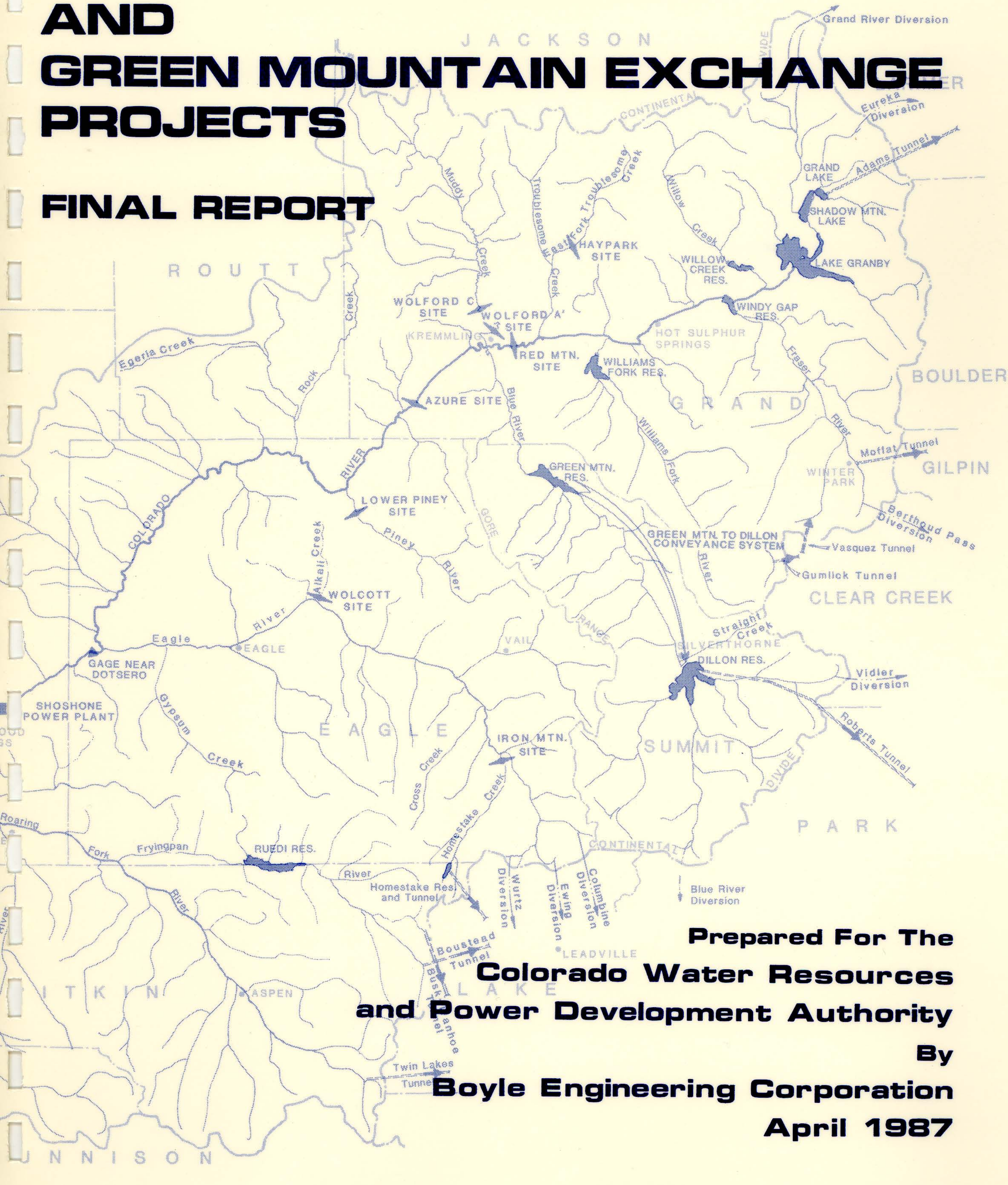
JOINT-USE RESERVOIR AND GREEN MOUNTAIN EXCHANGE PROJECTS

FINAL REPORT



Prepared For The
Colorado Water Resources and Power Development Authority
By
Boyle Engineering Corporation
April 1987

FINAL REPORT

JOINT-USE RESERVOIR
and
GREEN MOUNTAIN EXCHANGE PROJECTS

prepared for

COLORADO WATER RESOURCES
and
POWER DEVELOPMENT AUTHORITY

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Mr. Uli Kappus, P.E., Executive Director
Colorado Water Resources and Power Development Authority
1580 Logan Street, Suite 620
Denver, Colorado 80203

April 27, 1987

Final Report for the Joint-Use Reservoir and Green Mountain Exchange Projects

Dear Mr. Kappus:

We are pleased to submit this Final Report, as required by our contract dated June 7, 1985. A Summary Report is submitted under separate cover. These reports provide information and data on reconnaissance-level engineering and hydrology for two conceptual projects: the Joint-Use Reservoir Project and the Green Mountain Exchange Project. Preliminary hydrologic, geotechnical, engineering design and cost studies have been carried out to define project alternatives based on an approximately uniform level of investigation. The Study has progressed in the following sequence, each concluded with a task memorandum report:

- o Initial screening and pre-reconnaissance investigation
- o Hydrologic, water rights and operational analysis
- o Reconnaissance-level project designs and cost estimates
- o Comparative analysis and cost-yield relationships of alternative reservoirs and alternative combinations of water-exchange reservoirs and conveyance system.

Alternative projects have been defined along with water yields and estimated costs. This will provide consistent, comparative, technical and cost data to serve as a basis to evaluate these alternatives for possible future implementation.

It should be noted that no assessment has been included in the Study of the December, 1986 agreement between the Northern Colorado Water Conservancy District and its Municipal Subdistrict, the Colorado River Water Conservation District and the Denver Board of Water Commissioners nor the possible change in operation of the Shoshone Power Plant.

As you are aware, all of the work on this Study has been executed in Colorado. Thomas S. Maddock, the Principal-in-Charge of this contract for Boyle Engineering Corporation; Dan Boyd, Project Manager; Dale Diamond, Technical Director; Young Yoon, Principal Hydrologist; Chen & Associates, Geotechnical Subconsultant; and the entire Denver office staff; have greatly appreciated this opportunity to contribute to the evaluation of two such important projects within the State of Colorado. We also acknowledge the excellent support and guidance we have received from Dan Law, your Project Manager, and the project sponsors; the Colorado River Water Conservation District, represented by Eric Kuhn; and the Denver Water Board, represented by Bob Fischer; the Exchange Team Advisory Committee; and the Board of Directors of the Authority. We look forward to future opportunities to be of service.

Very Truly Yours,
BOYLE ENGINEERING CORPORATION

Daniel W. Boyd

Daniel W. Boyd, P.E.
Project Manager

DN-C10-100-12
DWB/cec

EXECUTIVE SUMMARY

The evaluation of the Joint-Use Reservoir and Green Mountain Exchange Projects (Study) was initiated by the Colorado Water Resources and Power Development Authority (Authority) at the request of the Colorado River Water Conservation District (CRWCD) and the Board of Water Commissioners for the City and County of Denver (DWB). The projects designated for investigation evolved from two concepts advanced to resolve the problems associated with increasing water demands in the Denver Metropolitan area and the concern over water supplies on the western slope of Colorado. The concepts are:

- Joint-Use Reservoir: A reservoir which would be capable of providing about 30,000 acre-feet (af) of water per year for east slope and west slope use, and

- Green Mountain Exchange: A water exchange project that would utilize Green Mountain Reservoir, a Replacement Reservoir, and a pump and pipeline system to increase the water supply to Roberts Tunnel.

Both concepts would require new reservoirs to regulate flows in the Upper Colorado River Basin. The Joint-Use Reservoir Project would retain snow-melt runoff to provide additional water for use in western Colorado and would partially relieve Dillon Reservoir of its obligation to release water to meet downstream water demands. The Green Mountain Exchange Project would replace the required releases from Green Mountain Reservoir with releases from a new Replacement Reservoir(s) and permit water stored in Green Mountain Reservoir to be pumped upstream to Dillon Reservoir. The increase in water in Dillon Reservoir due to either project could be diverted to the Denver Metropolitan area through the Roberts Tunnel.

PROJECT STUDY

Study of the Joint-Use Reservoirs and Green Mountain Exchange Projects began in June 1985, when Boyle Engineering Corporation (Boyle) entered into contract with the Colorado Water Resources and Power Development Authority. The purpose of this technical Study has been to estimate water yields for alternative Joint-Use Reservoirs and alternative Green Mountain Reservoir water exchange projects and to develop preliminary designs and cost estimates based on an approximately uniform level of investigation. This information is required prior to any decision on the next level of development of these projects.

This Study has been conducted in three phases. In the first phase, existing data was collected and evaluated for initial screening of nine reservoir sites identified by the Authority as candidates for Joint-Use or Replacement Reservoirs. The candidate reservoir sites for this study were selected based on the existence of a water right or claimed water right at the site and previous studies conducted by the Colorado River Water Conservation District, the Denver Water Board, the U.S. Bureau of Reclamation and others. The site selection was also based on geographic location within the Colorado River drainage. The reservoir sites are displayed on Figure 1.1.

In conjunction with reservoir site evaluations, three alternative conveyance routes from Green Mountain Reservoir to Dillon Reservoir were appraised to determine a "Green Mountain Pumpback System" configuration as part of the Green Mountain Exchange Project. The screening resulted in the selection of a pipeline route along Colorado Highway 9 for conveyance of water from Green Mountain Reservoir to Dillon Reservoir, and selection of the following six reservoir sites for further study.

- Wolford Mountain Site A' on Muddy Creek
- Wolford Mountain Site C on Muddy Creek
- Red Mountain Site on the Colorado River
- Azure Site on the Colorado River
- Wolcott Site on Alkali Creek
- Una Site on the Colorado River

The second phase consisted of hydrologic, operational and water rights analyses using a hydrologic simulation model. The third phase of the Study involved preliminary geotechnical investigations, core drilling at the Wolford Mountain Site A' and the Red Mountain Site, and subsurface investigation of the conveyance route. The third phase also included developing reconnaissance-level project designs, cost estimates and cost-yield relationships for comparisons of alternative reservoirs and reservoir combinations. The Study is concluded with this Final Report and Appendixes. A Summary Report, under separate cover, summarizes this Final Report. Recognizing that the Study has covered only a limited number of the facets involved in selection of projects for construction, no ranking or preference has been attempted. Instead, alternatives have been defined and water yields and estimated costs have been listed for consideration in the next level of development of these projects.

HYDROLOGIC ANALYSIS

The portion of the Upper Colorado River Basin analyzed in this Study extends over 200 miles from the Continental Divide to the Cameo stream gage upstream of Palisade, Colorado. The farthest downstream dam and reservoir site studied was the Una Site, between Parachute and DeBeque, some 35 miles east of Grand Junction.

In the upper reaches of the study area, the Blue River and Muddy Creek flow into the Colorado River near Kremmling, Colorado. Four dam sites are located in this vicinity. Alternative Wolford Mountain Sites A' and C are located to the north on Muddy Creek. The Red Mountain Site is 1 mile east of Kremmling on the Colorado River, and the Azure Site is 10 miles downstream to the southwest of Kremmling on the Colorado River in Lower Gore Canyon.

To the south of Kremmling, the existing Green Mountain Reservoir regulates the Blue River to allow for out-of-priority diversions by the Colorado-Big Thompson Project and to provide water for western Colorado's needs. Further up the Blue River, Dillon Reservoir stores flows which can be diverted through the Roberts Tunnel. West of this area, closer to the confluence of the Eagle and Colorado Rivers is the Wolcott Site. It is located on a minor tributary to the Eagle River just north of the town of Wolcott. It is an off-stream storage site that would be filled with water pumped from the Eagle River, and, alternatively could also receive flows pumped from the Colorado River.

Hydrologic conditions prevailing in the Upper Colorado River Basin were evaluated to estimate the yield from each reservoir site and to assess the ability of the proposed reservoirs to meet the objectives of a Joint-Use Reservoir or to function as Replacement Reservoirs for the Green Mountain Exchange Project. These hydrologic analyses included estimation of historic and natural streamflows in conjunction with evaluating historic water use. Water rights and other legal and institutional arrangements were examined that could affect river administration in the basin. In addition, various levels of water development in the basin were examined to estimate potential future water utilization. To effectively perform such hydrologic analyses, a review was made of several existing hydrologic simulation models for their potential applications. The Boyle

Engineering Stream Simulation Model (BESTSM) was selected because of its unique capabilities for handling various hydrologic conditions encountered in the study area. BESTSM accounts for monthly water volumes of inflows, diversions, return flows, river gains and losses, and outflows for each segment of the stream system and allocates water based on the Colorado water rights priority system and other legal and institutional arrangements.

Water Yield for Reservoirs and Reservoir Combinations- Firm annual yields for the six reservoirs selected and for the four representative combinations of reservoirs are presented in Table S.1. The firm annual yield estimates were based on a projected high future-level of water development in the basin with two different Green Mountain operating conditions. The first Green Mountain Reservoir operating condition assumes that the Reservoir would continue to operate as originally prescribed in Senate Document No. 80. The second condition assumes that the pumpback system from Green Mountain Reservoir to Dillon Reservoir and the associated water exchange arrangement would be in operation.

Some of the proposed reservoir sites are in effect alternative locations for regulating the same water. When more than one reservoir is added in the upper portion of the basin, the yield of the combination of reservoirs is less than the sum of the yields of individual reservoirs.

RESERVOIRS

Nine sites were initially designated as candidates for Joint-Use or Replacement Reservoirs. Screening analyses reduced these to six dam sites. Dimensions of the six dams and reservoirs considered in this reconnaissance-level study are presented in Table S.2. The study has involved review of previous reports and published data, site investigations, hydrological analyses, preliminary design and project cost estimates.

TABLE S.1

SUMMARY OF FIRM ANNUAL YIELD FOR SINGLE RESERVOIRS AND COMBINATIONS OF RESERVOIRS

RESERVOIR OR COMBINATIONS	RESERVOIR CAPACITY (af)	FIRM ANNUAL YIELD [1] WITHOUT PUMPBACK (af/yr)	FIRM ANNUAL YIELD [1] WITH PUMPBACK (af/yr)
Wolford Mountain A' [2]	120,000	40,000	39,000
w/Colo. diversion	120,000	49,000	43,000
Wolford Mountain C	60,000	25,000	23,000
Red Mountain	140,000	56,000	54,000
Azure	85,000	48,000	48,000
Wolcott w/diversion from:[3]			
Eagle & Colo.	350,000	138,000	135,000
Eagle only	160,000	69,000	65,000
Una	150,000	105,000	105,000
Red Mountain & Wolcott w/Eagle diversion	300,000	95,000	89,000
Azure & Wolcott w/Eagle diversion	245,000	120,000	114,000
Red Mountain& Una	290,000	161,000	159,000
Una & Wolford A' w/Colo. diversion	270,000	154,000	148,000

1) Firm annual yield is defined as the quantity of water that can be supplied every year without shortage during the study period of 1951 through 1983. A projected high future-level of water development was applied without and with pumpback to Dillon Reservoir from Green Mountain Reservoir.

2) Wolford Mountain A' would receive inflow only from Muddy Creek, or alternatively, supplement Muddy Creek storage with water diverted and pumped from the Colorado River.

3) Essentially all inflow to Wolcott would be pumped. For the 350,000 af High Wolcott, water would be diverted and pumped from both the Eagle and the Colorado Rivers. For the 160,000 af Low Wolcott, water would be diverted and pumped from the Eagle River only.

TABLE S.2

DESCRIPTIVE AND DIMENSIONAL DATA FOR ALTERNATIVE RESERVOIRS

	WOLFORD MOUNTAIN C	RED MOUNTAIN	AZURE	UNA
Dam Type	Embankment	RCC	Concrete Arch	RCC
Location	Muddy Creek	Colorado	Colorado	Colorado
Height of Dam	120 feet	85 feet	225 feet	130 feet
Dam Crest Length	1700 feet	1700 feet	500 feet	2550 feet
Reservoir Volume	60,000 af	140,000 af	85,000 af	150,000 af
Land Required	1900 acres	3300 acres	1150 acres	3800 acres
Hydropower Capacity	-	1300 kW	7000 kW	17,600 kW
Potential Generation	-	3.4 GWh/yr [1)]	36.2 GWh/yr [1)]	88.5 GWh/yr [1)]
Railroad Relocation	-	9.0 miles	2.8 miles	9.0 miles
Highway Relocation	0.9 miles	6.6 miles	-	7.0 miles

	WOLFORD MOUNTAIN A'	HIGH WOLCOTT	LOW WOLCOTT
Dam Type	Embankment	Embankment	Embankment
Location	Muddy Creek	Alkali Creek	Alkali Creek
Dam Height	140 feet	382 feet	288 feet
Dam Crest Length	3000 feet	4200 feet [3)]	2760 feet
Reservoir Volume	120,000 af	350,000 af	160,000 af
Land Required	2750 acres	2850 acres	1950 acres
Highway Relocation	0.8 miles	7.8 miles	7.8 miles
Pump Station	1 each [2)]	2 each [4)]	1 each [4)]
Off Site Source	Colorado	Eagle & Colorado	Eagle
Inlet Conduit length	3.9 miles	6.3 miles	0.8 miles

1) Annual hydropower generation from estimated flows in a year of average reservoir operation. One gigawatt-hour (GWh) equals one million kilowatt-hours (kW).

2) Pumping from the Colorado River is an alternative that would add about 10 percent to the yield of Wolford Mountain A'.

3) High Wolcott also requires two saddle dams with a total crest length of 4300 feet.

4) Essentially all flows stored at Wolcott would be pumped; in the case of High Wolcott water would be diverted and pumped from both Eagle River and Colorado River and for Low Wolcott, only from Eagle River.

CONVEYANCE SYSTEM

The conveyance system is the pumpback element of the Green Mountain Exchange Project. It is a pump and pipeline system that would pump water from Green Mountain Reservoir to Dillon Reservoir. Both reservoirs are located on the Blue River, a tributary to the Colorado River. Green Mountain Reservoir is located 26 miles downstream of Dillon Reservoir. The difference in water surface elevation between the two reservoirs is 1070 feet. The pipeline route would have an overall length of 140,900 feet and would follow the highway right-of-way where practical. Three pump stations would be required.

Two conveyance capacities have been studied: 8000 af per month and 12,000 af per month. These represent a range of the options available. To formulate Green Mountain Exchange Project alternatives in this Study, the estimated cost of the 8000 af per month conveyance system was combined with the lower capacity Replacement Reservoir(s) and the 12,000 af per month system was combined with the larger capacity Replacement Reservoir(s).

COST ESTIMATES

From the preliminary design drawings, reconnaissance-level estimates of construction quantities and unit construction prices have been made. Construction quantities, prices and allowances have been combined to obtain an estimated total construction cost. Costs are indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing costs have been added to obtain the total investment cost.

Comparison of alternative reservoirs and combinations of reservoirs for this Study is based on estimates of debt service and average annual operating costs. The costs are comprised of the yearly principal and interest payment (debt service) on assumed 30-year, 8 percent financing of the total investment cost and the estimated average annual cost of electrical pumping power, operating personnel, maintenance and repair costs. No price escalation was incorporated. No separate allowance was included for environmental mitigation. A summary of yearly debt service and average annual operating costs is presented in Table S.3. The relative costs among alternatives have been based on this average total cost per year.

TABLE S.3

SUMMARY OF YEARLY DEBT SERVICE AND AVERAGE ANNUAL OPERATING COSTS ($1,000,000)

FEATURE		DEBT SERVICE [1]	POWER COST	OTHER COSTS [3]	TOTAL COST PER YEAR [4]
RESERVOIR	CAPACITY (af)				
Wolford Mountain A'	120,000	$ 7.0	-	$0.1	$ 7.1
w/Colo. diversion	120,000	9.8	$0.4	0.2	10.4
Wolford Mountain C	60,000	3.8	-	0.1	3.9
Red Mountain	140,000	14.0	-	0.2	14.2
Azure	85,000	12.2	-	0.1	12.3
Wolcott w/diversion from:					
Eagle & Colo.	350,000	54.6	10.8	0.7	66.1
Eagle only	160,000	22.4	3.7	0.4	26.5
Una	150,000	25.8	-	0.2	26.0
CONVEYANCE SYSTEM	FLOW RATE [5] (af/yr)				
12,000 af/mo	119,000	$17.6	$ 9.8 [2]	$ 0.8	$ 28.2
	113,000	17.6	9.3 [2]	0.8	27.7
8,000 af/mo	87,000	14.7	7.1 [2]	0.7	22.5
	81,000	14.7	6.6 [2]	0.7	22.0

1) Annual principal and interest payment on 30-year, 8 percent financing in the amount of the total investment cost. Construction costs are indexed to January, 1986.

2) Compensation of $340,000 per year for lost revenue from Green Mountain hydropower included in average annual power costs for the conveyance system.

3) Other costs include operating and maintenance labor, supplies and repair costs.

4) Total cost per year is the sum of yearly debt service, average electrical power and other operating costs.

5) Flow rates shown match Average Conveyance System Flow of Table S.5.

ALTERNATIVE COMPARISONS

Single reservoirs of this Study were analyzed for their ability to meet Joint-Use Reservoir requirements. These single reservoirs and combinations of reservoirs were also analyzed for their ability to supply the greater yields necessary to meet Replacement Reservoir requirements based upon the yield of Green Mountain Reservoir. A reservoir could initially serve Joint-Use Reservoir requirements and later, in combination with additional constructed reservoir(s) also serve to meet water exchange project requirements.

Joint-Use Reservoir Project: Yields from Wolford Mountain A' and C, Red Mountain and Azure Reservoirs appear to be within range of the objective 30,000 af of annual yield for a Joint-Use Reservoir. Comparative unit costs of yield from Joint-Use Reservoirs are presented in Table S.4.

Green Mountain Exchange Project: For Replacement Reservoir alternatives, analyses were made of Wolcott Reservoir and combinations of reservoirs including Wolcott (Eagle) and Red Mountain, Wolcott (Eagle) and Azure, Red Mountain and Una, and Wolford Mountain A' and Una Reservoirs.

Two of the reservoir combinations, Red Mountain with Una, and Wolford Mountain A' with Una, are shown on Table S.1 as being capable of fully replacing the 144,000 af yield of Green Mountain Reservoir. Other reservoirs and reservoir combinations would only partially replace Green Mountain Reservoir. In these cases, a portion of the capacity of Green Mountain Reservoir would continue to serve its original function.

In order to compare alternatives in this study, it was assumed that the yield of Replacement Reservoir(s) could be exchanged for an equal quantity of yield from Green Mountain Reservoir. The actual quantities of exchange may differ from this Study depending upon the institutional arrangements between the interested parties.

TABLE S.4

JOINT-USE RESERVOIR ALTERNATIVES AND COMPARATIVE UNIT COST OF YIELD

JOINT-USE RESERVOIR	RESERVOIR CAPACITY (af)	FIRM ANNUAL YIELD [1] (af/yr)	TOTAL [2] COST PER YEAR ($1,000,000)	UNIT COST OF YIELD [3] ($/af)
Wolford Mountain A'	120,000	40,000	7.1	$180
w/Colo. diversion	120,000	49,000	10.4	210
Wolford Mountain C	60,000	25,000	3.9	160
Red Mountain	140,000	56,000	14.2	250
Azure	85,000	48,000	12.3	260

1) Firm annual yield is defined as the quantity of water that can be supplied every year without any shortage during the study period of 1951 through 1983. Demands on the Joint-Use Reservoirs are based on a high future-level of water development, without pumpback to Dillon Reservoir from Green Mountain Reservoir.

2) Total cost per year is the sum of yearly debt service, average electrical power and other operating costs from Table S.3.

3) Estimated cost per acre-foot of firm reservoir yield during a year of average operating costs.

Both the conveyance system cost and the costs of selected Replacement Reservoirs must be combined to indicate the total cost of additional water made available at Dillon Reservoir. This is presented in Table S.5. The Exchange Project Yield is the increased average annual yield to the Roberts Tunnel Collection System which results from operation of the pumpback with the Conveyance System and the Replacement Reservoir(s).

TABLE S.5

COST OF EXCHANGE PROJECT WATER
AVAILABLE TO ROBERTS TUNNEL

PROJECT [1] REPLACEMENT RESERVOIR(S)	AVERAGE [2] CONVEYANCE SYSTEM FLOW (1000 af/yr)	EXCHANGE PROJECT YIELD (1000 af/yr)	TOTAL [3] COST PER YEAR ($1,000,000)	UNIT [4] COST OF WATER ($/af)
Wolcott w/diversion from Eagle & Colo.	113	119	$ 93.8	$ 790
Red Mountain & Una	119	124	68.4	550
Una & Wolford Mountain A' w/Colo. diversion	119	124	64.7	520
Azure & Wolcott w/Eagle diversion	87	101	61.4	610
Red Mountain & Wolcott w/Eagle diversion	81	93	62.7	670

1) Diversion involves pumping from Colorado R. or Eagle R. or both rivers, as cited.

2) The 8000 af/mo conveyance system was combined with the Azure-Wolcott and Red Mountain-Wolcott replacement reservoirs. The 12,000 af/mo system was combined with the Wolcott/Eagle-Colo., Red Mountain-Una and Una-Wolford A' replacements reservoirs.

3) Total cost per year is the sum of yearly debt service, average electrical power and other operating costs from Table S.3 for the listed reservoir(s) and the corresponding conveyance system.

4) Estimated cost per acre-foot of project water delivered in a year of average operating costs.

In addition to supplying water to Dillon Reservoir by pumping from Green Mountain Reservoir, the water exchange concept would allow water to be retained in Dillon Reservoir that under current water rights priorities must be released to Green Mountain Reservoir. It should be noted that the estimated average annual project yields differ from the firm yields expected from the portion of the Green Mountain Reservoir storage made available for the exchange. This is due to a combined effect of the variation in the diversion requirement of Roberts Tunnel from year to year, the limited conveyance capacities, and the minimum flow release requirements (assumed to be 60 cfs in this Study) from Green Mountain Reservoir. The Unit Cost of Water in Table S.5 is the average Total Cost Per Year divided by the Exchange Project Yield. It provides a relative cost comparison of an acre-foot of water among the Exchange Project alternatives.

PROJECT DEVELOPMENT

This report summarizes the results of a 21-month study which has provided reconnaissance-level engineering and hydrology information on two conceptual projects: Joint-Use Reservoir and the Green Mountain Exchange. The development schedule for each of the alternatives addressed in this Study would require a series of additional steps including selection of preferred alternatives, feasibility and site-specific environmental studies, regulatory compliance, financing, design and construction and definition of institutional arrangements for project implementation. Neither the Colorado River Water Conservation District nor the Denver Water Board has made any decision with respect to the future of these projects.

A minimum of six additional years from the decision to proceed would be a reasonable projection of the time needed before any of the Joint-Use Reservoir projects would be completed. A minimum of 14 years is a reasonable projection for any of the Green Mountain Exchange Project alternatives. However, resolution of the various institutional constraints could substantially increase the time required. Recognizing that this Study has covered only a limited number of the facets involved in selection of projects for construction, no ranking or preference has been made. In accordance with the scope of work, water yields and estimated costs for alternatives have been derived and presented for consideration in the next level of implementation of these projects.

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1.0 INTRODUCTION

The evaluation of the Joint-Use Reservoir and Green Mountain Exchange Projects (Study) was initiated by the Colorado Water Resources and Power Development Authority (Authority) at the request of the Colorado River Water Conservation District (CRWCD) and the Board of Water Commissioners for the City and County of Denver (DWB). The projects designated for investigation evolved from two concepts advanced by the Metropolitan Water Roundtable. Organized by the Governor of the State of Colorado in 1981, the Roundtable discussed and suggested resolution of the problems associated with increasing water demands in the Denver Metropolitan area and the concern over water supplies on the western slope of Colorado. The concepts are:

- Joint-Use Reservoir: A reservoir which would be capable of providing about 30,000 acre-feet (af) of water per year for east slope and west slope use, and

- Green Mountain Exchange: A water exchange project that would utilize Green Mountain Reservoir, a Replacement Reservoir, and a pump and pipeline system to increase the water supply to Roberts Tunnel.

Both concepts would require new reservoirs to regulate flows in the Upper Colorado River Basin. The Joint-Use Reservoir would retain snow-melt runoff to provide additional water for use in western Colorado and would partially relieve Dillon Reservoir of its obligation to release water to meet downstream water demands. The Green Mountain Exchange Project would replace the required releases from Green Mountain Reservoir with releases from a new Replacement Reservoir(s) and permit water stored in Green Mountain Reservoir to be pumped upstream to Dillon Reservoir where it could be diverted to the Denver Metropolitan area through the Roberts Tunnel.

The purpose of the Study has been to estimate water yields for alternative Joint-Use Reservoirs and alternative Green Mountain Reservoir water exchange projects and to develop preliminary designs and cost estimates based on an approximately uniform level of investigation. This information is required prior to any decision on the next level of development of these projects.

1.1 BACKGROUND

Construction of Green Mountain Reservoir on the Blue River was initiated by the Bureau of Reclamation (USBR) in 1938 as a component of the Colorado-Big Thompson (CBT) Project. Completed in 1959, the CBT Project collects water from the upper reaches of the Colorado River Basin and delivers it through the Adams Tunnel to the Northern Colorado Water Conservancy District for some 125 water-user organizations in Northeastern Colorado. Of the total Green Mountain Reservoir capacity of about 154,000 af, 52,000 af is allocated for replacement of out-of-priority diversions by the CBT Project. This enables the CBT Project to divert water near Granby during periods when flows would otherwise have to be bypassed to satisfy senior water rights. The remaining capacity of Green Mountain Reservoir (approximately 100,000 af) is reserved for power generation and beneficial uses in Western Colorado.

The collection system for the Roberts Tunnel includes Dillon Reservoir, completed in 1963, on the Blue River upstream of Green Mountain Reservoir. Flows diverted from Dillon Reservoir through the Roberts Tunnel enter the North Fork of the South Platte River and then the Metropolitan Denver Water Supply. The water rights for the Roberts Tunnel Collection System are junior to those of Green Mountain Reservoir.

JOINT-USE RESERVOIR: The operational relationships between the Dillon Reservoir-Roberts Tunnel system and Green Mountain Reservoir have been prescribed in various federal court decrees. As a result of these decrees, Dillon Reservoir can store water out-of-priority with respect to Green Mountain Reservoir, subject to certain provisions. If Green Mountain Reservoir does not fill during the spring runoff, water is released from Dillon Reservoir to Green Mountain Reservoir. With permission of the USBR, however, this obligation can be satisfied through a water exchange. Water can be released from some other reservoir to replace water that otherwise would have to be released from Dillon Reservoir.

The Joint-Use Reservoir, one of two concepts investigated in this Study, would be dedicated toward meeting such obligations. This new reservoir would be capable of providing about 30,000 af of water per year, about half of which would be used to increase the water yield of Dillon Reservoir and the remainder would be used for western Colorado.

GREEN MOUNTAIN EXCHANGE: The concept of the Green Mountain Exchange Project recognizes Green Mountain Reservoir as a potential source of additional water supply to Dillon Reservoir. Water would be pumped from Green Mountain Reservoir through 26 miles of pipeline to Dillon Reservoir, which is about 1100 feet higher in elevation. In addition, a Replacement Reservoir(s) would be constructed to provide the replacement water for out-of-priority CBT Project diversions and to supplement natural flows thereby meeting in-basin irrigation and municipal demands. It (they) would also meet USBR water sales requirements that could otherwise have been supplied by the Green Mountain Reservoir storage made available for exchange. With the Replacement Reservoir(s) in operation, the water that would have been released from Green Mountain Reservoir could instead be pumped to Dillon Reservoir. This has been referred to as the "Green Mountain Pumpback" system.

1.2 PROJECT ORGANIZATION

Study of the Joint-Use Reservoirs and Green Mountain Exchange Projects began in June 1985, when Boyle Engineering Corporation (Boyle) entered into contract with the Colorado Water Resources and Power Development Authority. The Authority was created by the General Assembly of the State of Colorado for the primary purpose of aiding in the planning, design, financing and construction of water and hydroelectric power projects. There are nine members of the Authority, eight of whom represent the major drainage basins in Colorado, and the ninth represents the City and County of Denver. The Exchange Team, a subcommittee of the Metropolitan Water Roundtable, has served as an Advisory Committee to the Authority during the formulation and execution of the Study.

The Study has been conducted in three phases. The Phase 1 investigation was to collect and evaluate existing data on nine candidate reservoir sites and three alternative conveyance routes for initial screening purposes. The reservoir sites are displayed on Figure 1.1. Study phases and tasks are displayed in Figure 1.2. The results of the Phase I investigation are described in the Initial Screening Report, (Boyle, 1986). The screening resulted in selection of the highway route for conveyance of water from Green Mountain Reservoir to Dillon Reservoir, and selection of the following six reservoir sites for further study.

- o Red Mountain Site on the Colorado River
- o Wolford Mountain Site A' on Muddy Creek
- o Wolford Mountain Site C on Muddy Creek
- o Azure Site on the Colorado River
- o Wolcott Site on Alkali Creek
- o Una Site on the Colorado River

Phase 2 consisted of hydrologic, operational and water rights analyses using a hydrologic simulation model. The results of Phase 2 investigation are summarized in the report, Preliminary Hydrologic Analysis and included as Appendix A. Phase 2 analysis demonstrated that all six reservoir sites could produce the necessary yield to meet the Joint-Use Reservoir requirements (approximately 30,000 af), and that Wolcott Reservoir alone and several reservoir combinations could be used to replace the existing function of Green Mountain Reservoir as part of the Green Mountain Exchange Project.

Phase 3 is concluded with this Final Report which summarizes the complete Study. In addition to the initial screening and hydrological aspects it presents the reconnaissance-level designs and costs estimates of the six reservoir sites and conveyance system. It is supported by more detailed discussions in the following appendixes:

- o Appendix A - Hydrologic Analysis: hydrologic operational analysis and probable maximum flood analysis.

- o Appendix B - Preliminary Design of Reservoir and Conveyance System: geotechnical analysis, design considerations and detailed cost estimates.

- o Appendix C - Field Geotechnical Exploration of Red Mountain and Wolford Mountain A': drilling logs, field observations and laboratory tests.

This Final Report compares costs and yields of reservoirs, combinations of reservoirs and the conveyance system in a manner so that several of the elements can be combined to formulate alternative water exchange projects.

1.3 REPORT ORGANIZATION

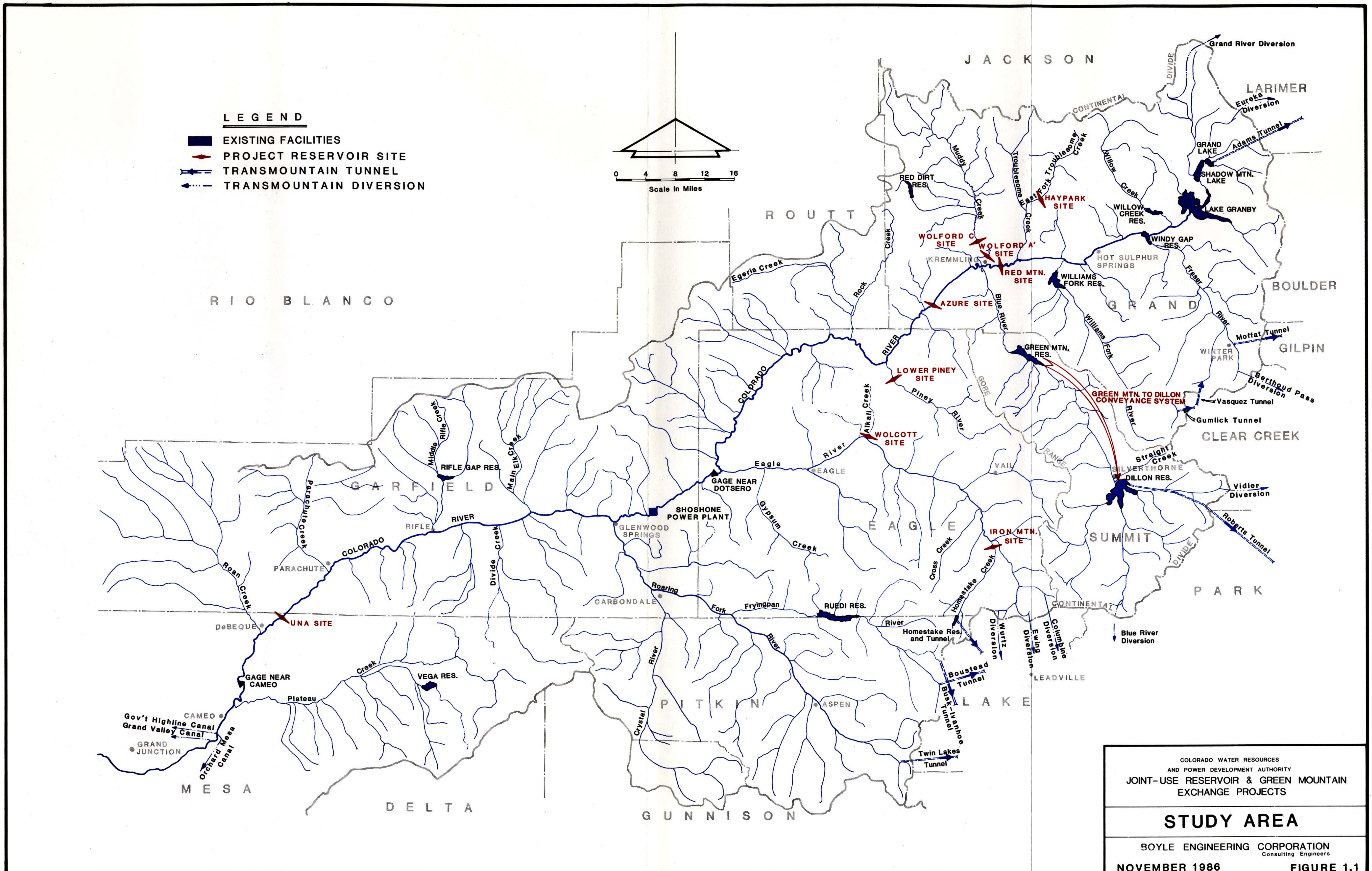
The final report is organized to present the three phases of the Study in the order in which they were concluded. This order has been:

- Initial Screening or pre-reconnaissance investigation
- Hydrologic, water rights and operational analysis
- Reconnaissance-level project designs and cost estimates and comparative analysis and cost-yield relationships of alternative reservoirs and potential reservoir combinations.

Recognizing that this study has covered only a limited number of the facets involved in selection of projects for construction, no ranking or preference has been attempted. Instead, efforts have been concentrated on developing reservoir water yields and project construction cost estimates based on an approximately uniform level of investigation.

An initial screening of previously published information about environmental concerns was conducted for each site with the objective of eliminating sites with "fatal flaws". No attempt has been made to suggest mitigation measures or evaluate the costs of complying with requirements that result from environmental concerns.

Reservoir sizes have been selected to represent the maximum size that conforms with the site topography, with some restrictions to limit the extent of reservoir-imposed relocations of existing improvements. Generally, maximum yields from individual reservoirs fall short of meeting Green Mountain Exchange Project objectives. Consequently, emphasis has been placed on portraying combinations of reservoirs and estimating firm yield from the sites rather than analyzing unit cost of yield for alternative dam heights at individual sites. Comparisons have been carried out for the cost of yield from alternative reservoirs and alternative combinations of reservoirs and for the cost of alternative conveyance system capacities.



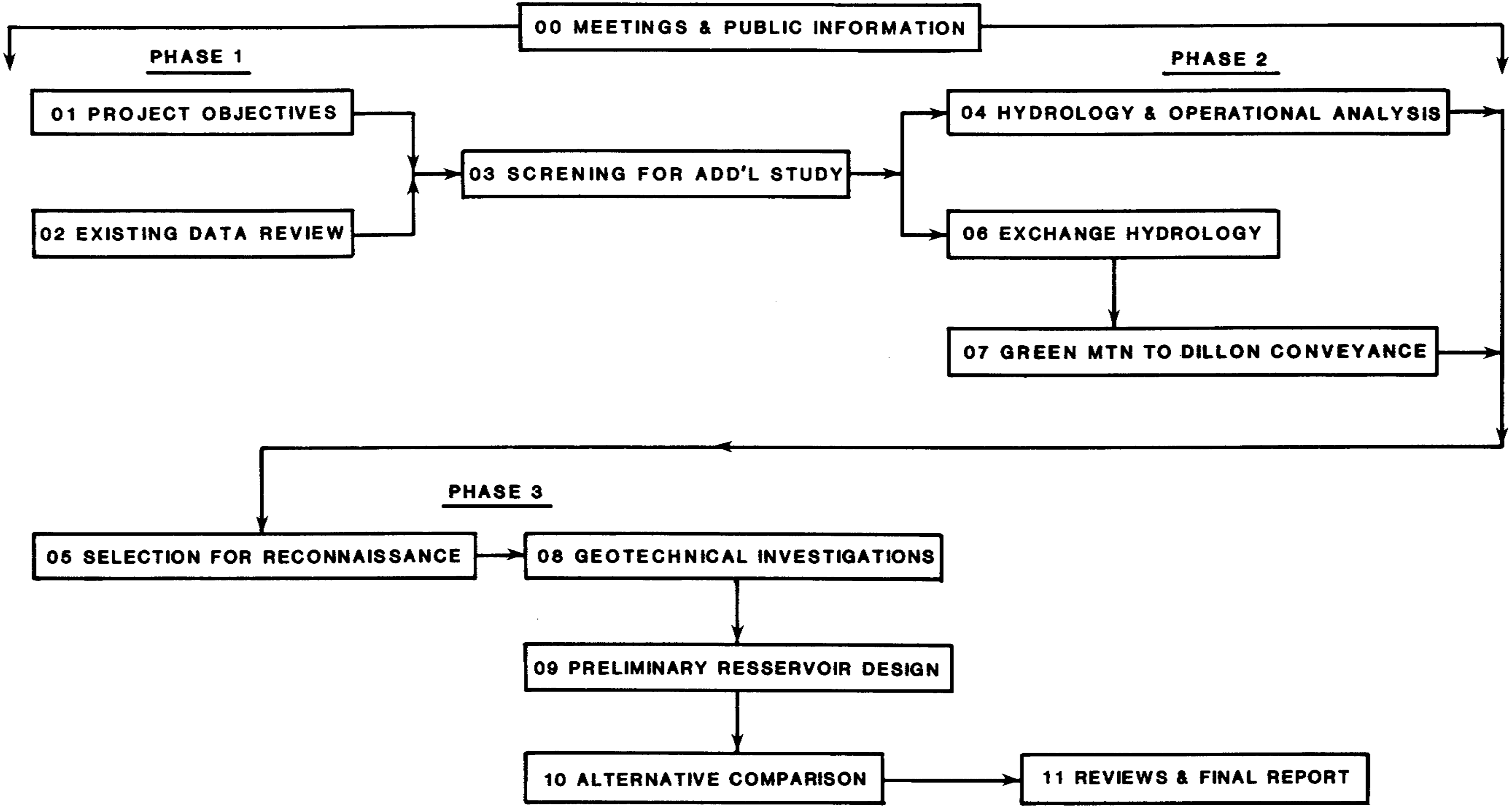
COLORADO WATER RESOURCES AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

STUDY AREA

BOYLE ENGINEERING CORPORATION
Consulting Engineers

NOVEMBER 1986 FIGURE 1.1



COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

STUDY PHASES & TASKS

BOYLE ENGINEERING CORPORATION
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FIGURE 1.2

2.0 ELEMENTS SELECTED FOR RECONNAISSANCE EVALUATION

Nine reservoir sites were initially identified by the Authority for consideration as Joint-Use Reservoirs or as Replacement Reservoirs for the Green Mountain Exchange Project. The Authority also identified the general requirements for the conveyance system. Selection of the candidate reservoir sites for this study was based on the existence of a water right or claimed water right at the site, geographic location within the Colorado River drainage, and previous studies conducted by the CRWCD, DWB, USBR and others. As a result of pre-reconnaissance evaluation and screening, six reservoir sites and one conveyance route were selected for reconnaissance evaluation. This chapter describes the nine reservoir sites and the results of the initial screening analysis.

2.1 RESERVOIR SITE DESCRIPTIONS

This section describes each of the reservoir sites which were considered during the pre-reconnaissance evaluation.

Red Mountain: The Red Mountain Dam Site is located on the Colorado River one mile east of Kremmling, Colorado. The dam would be located at a narrows, approximately 600 feet wide, formed by the base of Red Mountain to the north and a terrace extending from Junction Butte to the south. Upstream, a broad valley 4000 to 5000 feet wide would form the reservoir area. Reservoir capacities ranged from 84,000 af to 140,000 af.

Haypark: The Haypark Dam Site is located on the East Fork of Troublesome Creek, approximately 12 miles northeast of Kremmling, Colorado. Troublesome Creek flows south and enters the Colorado River 4 miles east of Kremmling. In the reservoir area, the East Fork of Troublesome Creek meanders through a 600-foot wide valley floor. Reservoir capacities of 20,100 af to 31,000 af were considered.

Wolford Mountain Sites A and A': Wolford Mountain Site A is located on Muddy Creek, 1 mile upstream from Kremmling, Colorado. Site A', a refinement due to geotechnical concerns, is located 3800 feet upstream from Site A. At Kremmling, an east-west trending ridge forms a 280-foot high bluff which overlooks the town. The north slope of this ridge would form the right abutment of the

Site A' dam. In addition to capturing flows from Muddy Creek, water could also be pumped into the reservoir from the Colorado River. This would require an intake below the Colorado-Blue River junction, 2 miles west of Kremmling, and a pump station to lift the flow about 160 feet to the average reservoir level. Reservoir capacities ranged from 80,000 af to 120,000 af.

<u>Wolford Mountain Site C</u>: Wolford Mountain Site C is located on Muddy Creek, 5 miles north of Kremmling and 3.8 miles upstream from Site A'. The dam site is a 250-foot wide canyon with sides that rise steeply to a height of approximately 80 feet. At that point, the left abutment is relatively flat for about 500 feet, continuing east to the base of Wolford Mountain. The right abutment slopes gently upwards towards Highway 40, located 0.7 miles west of the site.

The full reservoir water surface at Site C would be 10 feet higher than the water surface proposed for an alternative reservoir at Site A'. Considering that Site C is more distant from the Colorado River than Site A', supplemental storage of pumped water would be more expensive and was not included. Reservoir capacities ranged from 46,800 af to 80,000 af.

<u>Azure</u>: The Azure Dam Site is located on the Colorado River at the upper end of the Lower Gore Canyon, about 10 miles downstream from Kremmling. The Lower Gore Canyon is a narrow, deep gorge characterized by rugged, precipitous topography. While two dam heights were previously investigated at this site, only the higher dam, approximately 225 feet high was selected as retaining sufficient capacity for the purpose of this Study. Reservoir capacities ranged from 40,000 af to 85,000 af.

<u>Lower Piney</u>: The Lower Piney Site is located on the Piney River, about 1.5 miles upstream from the confluence with the Colorado River at State Bridge. In addition to capturing the flows of the Piney River, the site could serve as an off-stream storage facility for water pumped from the Colorado River. Reservoir capacities ranged from 80,000 af to 120,000 af.

<u>Iron Mountain</u>: The Iron Mountain Site is located on Homestake Creek, just upstream of its confluence with the Eagle River at Redcliff. The dam would be located 9 miles downstream from Homestake Reservoir. Besides the flows of Homestake Creek, the reservoir would receive water diverted from an upstream reach of the Eagle River by gravity flow. Reservoir capacities ranged from 68,000 af to 103,000 af.

Wolcott: The Wolcott Dam Site is 1 mile north of the town of Wolcott, Colorado, and 0.7 miles north of the Eagle River on Alkali Creek, a minor tributary of the Eagle River. The reservoir would serve as an off-stream storage site. Two reservoir sizes were considered in this Study. Low Wolcott would create a reservoir of 160,000 af capacity, supplied only with water pumped from the Eagle River, while High Wolcott, 350,000 af capacity, would be supplied by pumping from both the Eagle and Colorado Rivers. The Colorado River conveyance would include 6.3 miles of tunnel from State Bridge, Colorado, which is north of the reservoir site.

Una: The Una Dam Site is located on the Colorado River at the Mesa-Garfield county line. It lies between the towns of Parachute and DeBeque, Colorado, 35 miles northeast of Grand Junction, Colorado. Both dam abutments rise steeply from the 1100-foot wide valley floor. The reservoir would extend up the Colorado River valley to the town of Parachute. Reservoir capacities considered at this site ranged from 100,000 af to 196,000 af.

2.2 CONVEYANCE SYSTEM ALTERNATIVE DESCRIPTION

The conveyance system is the pumpback element of the Green Mountain Exchange Project. It is a pump and pipeline system that would pump water from Green Mountain Reservoir to Dillon Reservoir. Both reservoirs are located on the Blue River, a tributary to the Colorado River. Green Mountain Reservoir is located 26 miles downstream of Dillon Reservoir. The difference in water surface elevation between the two reservoirs is 1070 feet.

The Blue River flows from Dillon Reservoir to Green Mountain Reservoir, between the mountains of the Gore Range to the west and the Williams Fork Mountains to the east. The corridor along the river was judged to be the best general location for the conveyance pipeline because it provides the shortest route, ready access and crosses the gentlest terrain. Within this corridor, two alternative routes were considered, the River Valley Route and the Highway Route. In addition a gravity canal was conceptually evaluated as an alternative.

River Valley Route - The proposed river valley route parallels the west shore of Green Mountain Reservoir and follows the Blue River to Dillon Reservoir. Along this route, the Blue River is crossed five times. This alternative presents environmental impacts associated with the removal of riparian growth, impacts on fisheries and water quality during the construction of river crossings, and soil erosion. It would also present visual impacts due to the removal of vegetation near the river.

Highway Route - The proposed highway route follows the west shore of Green Mountain Reservoir and west side of Colorado Highway 9 to the town of Silverthorne. Using this route would allow the installation of the pipeline in the highway right-of-way where practical. It would require only one crossing of the Blue River. Impacts associated with this route include traffic delays and temporary detours, soil erosion, and riparian vegetation removal along side the highway.

Mountainside Gravity Route - The mountainside gravity route would require pumping water from Green Mountain Reservoir at elevation 7900 to approximately elevation 9200 to allow for gravity flow to Dillon Reservoir. To avoid the Eagle's Nest Wilderness Area, a pressure pipeline section, as in the highway route, would be installed from the reservoir to a pump station at Slate Creek. From Slate Creek the pipeline would climb the mountain to a point where water would flow by gravity to Dillon Reservoir. The covered canal section would generally follow the land contours to approximately elevation 9000, immediately north of Silverthorne, from which point a pressure pipeline would be employed to reach Dillon Reservoir. Along the route, inverted siphons would be used to cross under creeks.

Several factors associated with this route would create negative impacts. One is excavation on steep mountain slopes to create a bench for the installation of the pipeline and the access roadway for pipeline maintenance. Another is the loss of a wide band of vegetation within the heavily forested area along the east slope of the Gore Range which could create negative environmental and visual impacts.

2.3 INITIAL SCREENING

The objective of the initial screening was to eliminate at an early stage, those reservoir sites that would not be suitable as a Joint-Use or Replacement Reservoir. To do this, existing data on the nine identified reservoir sites was gathered and analyzed. A field examination of each site was also conducted. Historical monthly streamflows were estimated and preliminary yields were calculated for each site. Environmental and geotechnical factors which could preclude or seriously impede site development were also considered.

Environmental factors that were considered included known cultural resources, known threatened or endangered species, wildlife habitat and inundation of farm land. Based on the information evaluated, there appeared to be no environmental factors which would preclude further consideration of any of the reservoir sites.

Similarly, the alternative conveyance routes were evaluated to determine the more favorable route for future consideration. No environmental factors were discovered which would preclude further consideration of the conveyance routes.

Reservoir Sites Eliminated

Three reservoir sites were eliminated based on the findings of the initial screening and one site location was modified.

The Haypark Site was eliminated due to the small potential reservoir yield. Preliminary hydrologic analysis estimated a potential yield in the range of 10,000 to 12,000 af/yr.

The Lower Piney Site was eliminated because of the high potential for reservoir seepage due to the existence of permeable lenses within the bedrock in the reservoir area and at the dam axis.

The Iron Mountain Site was also eliminated due to potential reservoir seepage. The Homestake Shear Zone could allow significant seepage through the ridge to the adjacent Eagle River canyon.

The Wolford Mountain Site A location was modified due to questions about the stability of the right abutment. Site A', 3800 feet upstream, was substituted for Site A. These geotechnical questions are not a concern at Site A'.

Reservoir Sites Selected

The following reservoir sites were selected for further analysis: Red Mountain, Wolford Mountain A', Wolford Mountain C, Azure, Wolcott (both higher and lower dams) and Una. Table 2.1 presents the reservoir sites and capacities designated for reconnaissance-level evaluation. The minimum pool shown represents the lowest reservoir content after all usable water is released. It was based on either the assumed structural minimum (such as the elevation of the inlet to the outlet works) or the allowance reserved for sediment accumulation.

TABLE 2.1

RESERVOIR SITES AND CAPACITIES SELECTED
FOR RECONNAISSANCE INVESTIGATION

Reservoir	Total Storage (af)	Minimum Pool (af)
Wolford Mountain A'	120,000	6,500
Wolford Mountain C	60,000	6,500
Red Mountain	140,000	5,600
Azure	85,000	5,300
Wolcott		
High	350,000	14,000
Low	160,000	14,000
Una	150,000	45,000

Selected Conveyance Route

Using the criteria of reliability, maintainability, environmental disturbance, length and construction costs, the highway route appears to be the most favorable route. It is the shortest route and would have the easiest access for construction and maintenance. It requires the fewest river crossings and would create the least disturbance of the natural setting. The highway route was selected for use in the operation studies and cost analysis.

3.0 WATER RESOURCES

Hydrologic analyses were conducted to estimate the yield from each reservoir site and to estimate the capability of the proposed reservoirs to meet the requirements of a Joint-Use Reservoir or to function as Replacement Reservoirs for the Green Mountain Exchange Project. Legal and operational considerations involved hydrologic characteristics, administration of the Upper Colorado River Basin and projected water resources utilization. Specific hydrologic analyses were made of the following:

- A water rights analysis provided estimates of flows that are legally available for storage at the six proposed sites based on the monthly flows for the 1951 to 1983 hydrologic study period. The analysis was performed for existing conditions and moderate and high levels of future development of conditional water rights (see Section 3.3).

- Reservoir operation analyses were performed to estimate the firm annual yield of each reservoir site. In consultation with the Authority, the firm annual yield was defined as the volume of water that can be provided every year without any shortage.

- The Green Mountain Reservoir was analyzed in detail to estimate its yield based on the requirements of Senate Document No. 80, existing USBR operating policies and existing Colorado River administration.

- Operational analysis of the Upper Colorado River Basin with projected future water demands provided an estimate of the increased yield of Dillon Reservoir due to pumpback operation and provided estimates of the yields of single reservoirs and combinations of reservoirs serving as Replacement Reservoirs for Green Mountain Reservoir.

To facilitate the computations involved in hydrologic and water rights data management, and to perform the reservoir operation analyses, a monthly hydrologic simulation model was used. It incorporated the Colorado water rights priority system and other legal and institutional arrangements identified during the Study. The modeling area covered the Upper Colorado River Basin above the Cameo gage near Palisade. This chapter describes the physical, legal and operational considerations incorporated in the model. Reservoir yields are described in the following chapter and alternative yields and costs of the Green Mountain Exchange Project components are described in Chapter 13.

3.1 HYDROLOGIC CHARACTERISTICS

The Upper Colorado River watershed, which is the subject of this Study, extends from the Continental Divide at an elevation in excess of 10,000 feet, to the Cameo gage near Palisade, 210 miles downstream, at an elevation of about 4,800 feet (see Figure 1.1). The drainage area above the Cameo gage is approximately 8,000 square miles.

The major tributaries to the Colorado River in the study area are: the Fraser, Williams Fork, Blue, Piney, Eagle, and Roaring Fork Rivers. Smaller streams which also contribute to the Colorado River include Willow, Troublesome, Muddy, Rock, Divide, Elk, Rifle, Parachute, Roan, and Plateau Creeks. Principal reservoirs located in the Upper Colorado River Watershed include: Grand Lake, Shadow Mountain Lake, Lake Granby, Willow Creek Reservoir, and Green Mountain Reservoir, all operated by the USBR as part of the CBT; Williams Fork and Dillon Reservoirs owned by the DWB; Homestake Reservoir, jointly owned by the cities of Colorado Springs and Aurora; and Ruedi Reservoir operated by the USBR as part of the Fryingpan-Arkansas Project.

Precipitation varies dramatically within the study area. At the higher elevations, precipitation exceeds 30 inches per year, whereas in Garfield County, 30 miles east of Grand Junction, Colorado, annual precipitation is as low as 10 inches per year. Snowfall in the study area begins as early as October and ends as late as the end of April.

Average annual virgin flow of the Colorado River (based on 1951-1983 historical flows adjusted for major diversions and reservoirs as described in Chapter 4) ranges from about 0.5 million af at the headwaters near Hot Sulphur Springs to 3.1 million af at the Cameo gage. A wide variation in total annual virgin flow is characteristic of the river as illustrated by annual extremes at the Cameo gage of 1.7 million af in 1977 and 5.2 million af in 1983.

3.2 ADMINISTRATION OF THE UPPER COLORADO RIVER

The administration of the Upper Colorado River is largely affected by two major water rights: Cameo and Shoshone. The more senior of these and therefore higher priority, is the demand at Cameo. It consists of a number of senior water rights for the Grand Valley Canal and the Grand Valley Irrigation Project, some of which date to the late 1800's. The Grand Valley Irrigation Project includes the Government Highline Canal and the Orchard Mesa Canal rights which are diverted from the river at the Grand Valley Diversion Dam. The demand measured at the Cameo gage both for the Grand Valley Canal and the Grand Valley Irrigation Project during the summer is normally in excess of 2000 cubic feet per second (cfs). However, if a check structure located at the afterbay of the Orchard Mesa Power Plant is operated, which allows the power plant tailwater to be used by the Grand Valley Canal, the demand may be reduced to less than 2000 cfs.

The demand for water at Shoshone has a priority date of 1902 and a decreed diversion rate of 1250 cfs. It supplies the Public Service Company of Colorado's Shoshone hydroelectric plant in Glenwood Canyon and is a year round non-consumptive use. The plant also has a junior water right for 158 cfs with a priority date of 1929. In most years, when the 1902 right at Shoshone is satisfied, there is sufficient water to meet the summer demand at Cameo. Downstream of the Shoshone Power Plant, before reaching Cameo, the Colorado River flow is supplemented by tributary inflow largely from the Roaring Fork River.

When flow is insufficient, a senior water right holder can place a call upon the river to which junior rights must defer and reduce diversions. Many junior diversions in the Upper Colorado River Basin have been protected from the Shoshone or Cameo calls by the replacement function of several reservoirs. For example, the major function of the Williams Fork Reservoir is to allow for out-of-priority diversions by the Denver systems, and one of the functions of Ruedi Reservoir is to protect diversions by the Fryingpan-Arkansas Project. Green Mountain Reservoir also has a replacement function which warrants the following detailed description because of its significance to this project.

Operation of Green Mountain Reservoir

Green Mountain Reservoir, located on the Blue River, is a feature of the CBT West Slope Collection and Storage System constructed by the USBR. Construction of Green Mountain dam was completed in 1943.

The reservoir has a total storage capacity of 153,639 af, with a dead storage of 6,860 af. It has an original storage right of 154,645 af and a refill right of 6,316 af. The operating policy of Green Mountain Reservoir is set forth in Senate Document No. 80 (Act of August 9, 1937, 50 Stat. 564) and reaffirmed in subsequent court decrees and stipulations including:

- Consolidated Cases (Civil Actions) Nos. 2782, 5016, and 5017
- October 12, 1955 Stipulation and Decree
- April 16, 1964 Stipulation and Decree
- November 2, 1977 Memorandum Opinion and Order
- February 9, 1978 Supplemental Judgment and Decree

Senate Document No. 80 specifies that 52,000 af of storage in the Green Mountain Reservoir is to be reserved to supply replacement water to the Colorado River for out-of-priority CBT project diversions. The balance of the storage, about 100,000 af, is to be used primarily for power generation and for irrigation and domestic uses in western Colorado which are not satisfied by natural flows. Under Senate Document No. 80, one of the uses of the Reservoir in such circumstances, is to augment irrigation and domestic uses that existed in 1937 and, to the extent storage water is thereafter available for release, to augment similar needs which subsequently arise. To meet these needs, the Reservoir has been operated to maintain a flow of about 1250 cfs during the irrigation season at the Dotsero gage in Glenwood Canyon. Approximately 66,000 af of water was released from storage in 1977 to supplement natural flow shortages in western Colorado (USBR, 1963-1982).

The water rights for Green Mountain Reservoir have a priority date of 1935, and are senior to those of Roberts Tunnel and Dillon Reservoir, which are Denver's Blue River diversion. A combination of the 1955 and 1964 Stipulations and Decrees provided that, upon approval of the Secretary of Interior, Denver can store out-of-priority water in Dillon Reservoir during the spring snowmelt runoff season. This out-of-priority storage is permitted on the condition that if Green Mountain Reservoir does not fill, water would be released later to satisfy the fill requirement of Green Mountain Reservoir. Water can be released either from Dillon Reservoir to flow into Green Mountain Reservoir, or from Williams Fork Reservoir to meet the Green Mountain Reservoir release obligations. Another condition was that energy lost to the Green Mountain Power Plant because of reduced flow, would be replaced in kind.

On December 22, 1983, the USBR published an operating policy for Green Mountain Reservoir (Federal Register Vol. 48, No. 247) with a provision that releases from the 100,000 af power pool would be made available without charge to make up natural water shortages for those irrigation and municipal uses perfected by use prior to October 16, 1977. Releases for these purposes are not to exceed 66,000 af per year. The remaining water from the power pool would be made available for use on the western slope, through "water sales". The amount of water sales and the analysis of their impact are described in the Draft Water Marketing Program Environmental Statement issued by USBR in June 1985.

3.3 WATER RESOURCES UTILIZATION

Water availability for the various reservoirs analyzed as potential Joint-Use Reservoirs or components of the Green Mountain Exchange Project was evaluated under three levels of water utilization in the basin: the existing level of use and two future levels of use. Each of these three levels reflects increased development in the basin over present (1986) use.

In all three operating scenarios, the major projects to be operated and their average annual target demands were established by the Authority in consultation with the DWB and the CRWCD. Demands used in the operating simulations for all three development scenarios were furnished by the DWB for their Fraser River, Williams Fork and Blue River diversion systems for operation both with and without the Green Mountain exchange. These demands assumed construction of Two Forks Reservoir with a storage capacity of 1.1 million af. Demands for the proposed Rock Creek Reservoir, Indian Creek Reservoir, reformulated West Divide Project and Red Cliff Project were furnished by the CRWCD.

Existing-Level Use

The Existing-Level Use Scenario consists of operation of the Colorado River basin under its 1983 level of development with several exceptions. The exceptions, which consisted of expanded demands in all cases, fall into two general categories of use: 1) increased diversions by existing systems which are not presently operating at their full capacity; and 2) diversions by project features or proposed facilities which are not yet constructed. These diversions are:

1. <u>Expanded Operation of Existing Systems</u>
   - CBT/Windy Gap Project (Adams Tunnel)
   - Fryingpan-Arkansas Project (Boustead Tunnel)
   - Fraser River Diversion System (Moffat Tunnel)
   - Blue River Diversion Project (Roberts Tunnel)
   - Homestake Project (Homestake Tunnel)
   - Green Mountain Reservoir Water Sales Program

2. <u>New Facilities not Presently in Existence</u>
   - Williams Fork Collection System Extension (Gumlick Tunnel)
   - Straight Creek Diversion (Roberts Tunnel)
   - Two Forks Reservoir (1.1 million af storage)

Table 3.1 contains a comparison of the average levels of annual diversions for the major transmountain diversion systems as historically recorded and as operated in the Existing-Level Use Scenario with increased demands by those systems listed above. Green Mountain Reservoir Water Sales Program, which is not included in Table 3.1, is assumed in this Study to supply water ranging from 12,500 af/yr to 22,800 af/yr with an average of approximately 16,000 af/yr (RCI, 1985).

TABLE 3.1

AVERAGE ANNUAL TRANSMOUNTAIN DIVERSION DEMANDS (1000 af/yr)

Facility	Recorded Diversions[1)] 1951-83	Recorded Diversions[1)] 1973-83	Projected Demands of Existing-Use Scenario[2)] 1951-83
Adams Tunnel [3)]	219.7	230.6	288.2
Boustead Tunnel	16.2	45.7	52.5
Busk-Ivanhoe Tunnel	5.9	6.9	6.0
Columbine Ditch	1.5	1.7	1.6
Ewing Ditch	1.0	1.0	1.1
Grand River Ditch	17.0	17.1	17.0
Gumlick Tunnel	5.0	4.6	27.1
Homestake Tunnel	12.3	24.2	29.4
Hoosier Tunnel	7.7	7.9	8.2
Moffat Tunnel (excluding Gumlick Tunnel Diversion)	45.4	52.4	72.4
Roberts Tunnel	29.6	62.0	153.4 (278.4)
Twin Lakes Tunnel	42.8	43.7	42.8
Wurtz Ditch	2.5	2.9	2.5
TOTAL	406.6	500.7	702.2 (827.2) [4)]

1) Recorded diversions were taken from Annual Operating Plans (USBR, 1951-1983b), and personal communications with DWB personnel.
2) Existing-Use Scenario includes expansion of existing facilities.
3) Includes CBT and Windy Gap diversions.
4) Demands in ( ) are increased to include projected Green Mountain Exchange.

The in-basin demands for the present level of irrigation, municipal and industrial use in the basin were estimated based on evaluation of the historic diversion records for ditches and structures having entitlements of 5.0 cfs and larger. Recorded diversions for the entire 33-year study period were first compared with the associated decreed water rights. Major discrepancies between the listings were resolved. Where the diversion records were incomplete, the beginning years of diversion were assumed on the basis of the decreed dates of appropriation for the rights. Incomplete diversion records were extended or filled in by comparison with the available portions of the diversion record and by correlation with other diversions. Finally, to resolve inconsistencies, the aggregate of the adjusted diversion records for each basin was compared with the consumptive-use estimates prepared by the USBR (USBR, 1963-1982).

Table 3.2 shows the estimated present level of irrigation demands in the Colorado River Basin above Cameo for the major segments of the basin. Also shown on Table 3.2 are the average irrigation efficiencies. These efficiencies represent the amounts of crop consumptive use as a percent of the diversions. The average annual use of water for irrigation in the Colorado River Basin above Cameo has not changed significantly over the 33-year study period.

TABLE 3.2

ESTIMATED IRRIGATION DIVERSIONS [1)] FOR MAJOR WATER RIGHTS

UPPER COLORADO RIVER BASIN (1951 - 1983 Average Annual)

Water District	Geographic Location	Estimated Diversion (1000 af)	Estimated Irrigation Efficiency (%)
36	Blue River	17.7	50
50,51	Fraser River Muddy Creek Troublesome Creek Upper Colorado River near Kremmling	58.7	60
37	Eagle River	28.1	60
38	Roaring Fork River	83.6	45
52,53	Colorado River between Kremmling and Glenwood Springs	29.6	50
39,45,70	Colorado River between Glenwood Springs and Cameo	101.9	60
	TOTAL	319.6 [2)]	

1) Values are estimates based on available diversion records maintained by the State Engineer's Office.

2) Accounts for more than 90% of all irrigation diversions upstream of Cameo

It is estimated that municipal water use in the basin has doubled during the span of the 33-year study period, based on analysis of recorded diversions. "Municipal water use" in this analysis includes, as a minor component, some industrial water uses that are associated with mining. Table 3.3 indicates the estimated average annual municipal demands by five-year increments during the study period. A total aggregate demand of 38,700 af per year was operated in the Existing-Level Use Scenario for all years. Consumptive depletions of diversions for municipal uses varied from 20 to 100 percent depending on the location and nature of the demand.

TABLE 3.3

ESTIMATED ANNUAL IN-BASIN [1)]
MUNICIPAL DEMANDS

UPPER COLORADO RIVER BASIN

PERIOD	DEMANDS (1000 af/yr)
1951 - 1955	18.2
1956 - 1960	18.2
1961 - 1965	21.3
1966 - 1970	28.2
1971 - 1975	31.4
1976 - 1980	34.5
1981 - 1983	38.7
Existing Level	38.7

1) Values are estimated based on available diversion records maintained by the State Engineer's Office.

Future-Level Use

The two future level operating scenarios consisted of increasing the demands of certain existing projects and adding presently undeveloped conditional projects to the Existing-Level Use Scenario to produce target levels of water demand under projected moderate and high levels of future development in the basin. The projects to be expanded and developed in the future and the target levels of demand were established by the Authority in consultation with the DWB and the CRWCD. Specific projects included in Moderate and High Future-Level Use Scenarios are as follows:

Moderate-Future Scenario

All of those projects under the Existing-Use Scenario were utilized plus:

- Homestake Project (Phase II)
- Ruedi Reservoir Marketing (partial use)
- Rock Creek Reservoir
- Indian Creek Reservoir

High-Future Scenario

All of the projects included in the Existing-Use and Moderate Future-Use Scenarios were utilized plus:

- Eagle-Arkansas
- Continental-Hoosier
- Pueblo/Eagle
- Ruedi Reservoir Marketing (full use)
- West Divide Project
- Red Cliff Project
- Oil Shale Projects (above that available from Ruedi and Green Mountain Reservoirs)

Although not specifically identified by individual towns or ski area, the high projected demand of the Upper Fraser River, Upper Blue River, and the Eagle River areas were considered. It was felt that Green Mountain water sales, Indian Creek Reservoir, and Rock Creek Reservoir would accommodate the needs of these areas. It should be recognized that this Study was focused on the entire basin and was not intended to cover site-specific water supply problems or demand projections.

Table 3.4 lists the additional projects and expansion of existing projects operated in the two Future-Level Use Scenarios along with their additional average annual depletions above the Existing-Level Use Scenario. These demands were identified by reviewing the following documents: "Draft Supplement to the Final Environmental Statement for the Fryingpan-Arkansas Project, Water Marketing Program for Ruedi Reservoir", (Simons, et al., 1983); "Application for License for Project No. 2511, Redcliff Project", (Fleming, 1977); "1984 Development Work for the Redcliff Project", (Western, 1984); and supplemental information provided by DWB and CRWCD.

In the Moderate Future-Level Use Scenario, the water use in the basin was increased above the Existing Level by an annual average of 14,000 af of in-basin municipal and industrial depletions, 3000 af for oil-shale development and 21,000 af of transmountain diversions.

In the High Future-Level Use Scenario, the average annual water use was increased to provide an additional 64,000 af for in-basin irrigation, municipal and industrial uses other than oil shale; 133,000 af for oil-shale development; and 36,000 af for transmountain diversions.

An estimated total future demand averaging 136,000 af per year was used in the operating simulation for oil-shale development in the Colorado River Basin. This figure was based upon those projects for which Biological Consultations are filed with the U.S. Fish and Wildlife Service and upon quantities requested in applications to USBR under the Green Mountain Reservoir and Ruedi Water Sales Programs (Simons, et al., 1983; RCI, 1985). Of this total, 3,000 af was supplied from the Green Mountain Reservoir Water Marketing Sales in all three scenarios; and 3,000 af and 40,000 af were provided under the Ruedi Reservoir Water Marketing Sales in the Moderate and High Future-Level Use Scenarios, respectively. The balance of the 93,000 af annual demand for oil shale in the High Future-Level Use Scenario was assumed to be supplied from the Colorado River in the vicinity of Parachute and DeBeque and in the Main Elk Creek Basin.

TABLE 3.4

PROJECTS/FACILITIES INCLUDED IN
MODERATE AND HIGH-FUTURE-LEVEL USE SCENARIOS

PROJECT OR FACILITY	RIVER OR CREEK	AVE. DEPLETION (1000 af/yr) [1) MOD. FUT. LEVEL	HIGH FUT. LEVEL	ASSUMED ANNUAL DEMAND PATTERN
Homestake Project				
Homestake II	Eagle	21	21	Constant
Eagle-Arkansas	Eagle	0	6	Constant
Continental-Hoosier	Blue	0	6	Constant
Pueblo/Eagle Systems [2)]	Eagle	0	3	Constant
Ruedi Res. Marketing	Fryingpan	3	40	Variable
Rock Creek Reservoir	Rock	13	13	Constant
Indian Creek Res.	Eagle	1	1	Constant
West Divide Project	Divide	0	25	Variable
Red Cliff Project	Eagle	0	25	Constant
Oil Shale Projects [2)]	Colorado & Main Elk	0	93	Variable
TOTAL		38	233	

1) Numbers represent average annual depletions, not project yields.

2) Assumes that the increased demands by oil shale projects are 3000 af/yr under the Moderate Future-Use Scenario and are supplied by Ruedi Reservoir Marketing Program. Under the High Future-Use Scenario, increased oil shale demand would be 133,000 af/yr, of which 40,000 af/yr are supplied by the Ruedi Reservoir Marketing Program.

A comparison of the total water demands in the Colorado River basin above Cameo used in the simulations for the three levels of development is displayed in Table 3.5.

TABLE 3.5

DEMAND BY CATEGORY OF USE FOR DEVELOPMENT SCENARIOS

	Average Annual Demand - (1000 af)		
Category of Use	Existing [1)]	Moderate Future	High Future
Transmountain Diversions	702	723	738
Irrigation	320	320	336
Municipal	52	66	100
Oil Shale	3	6	136
TOTAL	1,077	1,155	1,310

1) The projected Green Mountain Water Sales Program is assumed to vary from 12,500 af/yr to 22,800 af/yr with an average of approximately 16,000 af/yr. Of this average, 13,000 af/yr is considered to be municipal use and the remaining 3,000 af/yr is considered to be oil-shale use.

4.0 RESERVOIR YIELDS

The Boyle Engineering Stream Simulation Model (BESTSM) was utilized to analyze the Upper Colorado River Basin and to estimate yields from the proposed reservoir sites under various water development conditions.

The conditions analyzed to estimate yields from the proposed reservoirs include the following:

- Three development scenarios
- Reservoir operation with and without pumpback
- Two alternative pumpback capacities
- Alternative capacities of proposed reservoirs
- Single reservoir and combinations of proposed reservoirs

This section briefly describes the hydrologic simulation model and its application, operating rules incorporated and reservoir yields under various conditions. Supplemental information is in Appendix A.

4.1 HYDROLOGIC SIMULATION MODEL

The hydrologic system analyzed under this Study is complex. The streamflows in the basin are affected by numerous transmountain and in-basin diversions and storage facilities. In addition, water rights and legal and institutional arrangements existing in the study area impact the amount of water available for development at a given location. To effectively perform such hydrologic analysis and reliably compare various water development alternatives, the application of a hydrologic simulation model was sought as part of this Study. A review was made of several existing hydrologic simulation models. Most of these models were developed for site-specific purposes or did not have the capability of handling the unique hydrologic and river operating conditions encountered in the study area. For these reasons, the hydrologic simulation model, BESTSM which appeared to be most suitable for the purpose of this Study, was selected with the approval of the Authority. BESTSM accounts for monthly water volumes of inflows, diversions, return flows, river gains and losses, and outflow for each segment of the stream system.

For reservoirs, a complete water accounting is also performed. Some of the factors considered include reservoir inflow, pumping from adjacent streams, bypass requirements to meet senior downstream rights and minimum instream flows, reservoir releases, spills, seepage, and evaporation. The model allocates water based on the Colorado water rights priority system and other legal and institutional arrangements in the Colorado River Basin which are discussed in previous sections.

The modeled area covers the Upper Colorado River Basin above the Cameo gage. The study system is divided into 56 segments. The model incorporates over 800 major diversion structures each having an aggregate water right greater than 5 cfs. This accounts for more than 90 percent of the total diversions in the basin. The total number of water rights associated with these structures is approximately 1600. The operations of all existing major reservoirs are simulated in the model. These include Lake Granby, Willow Creek, Green Mountain, Williams Fork, Dillon, Homestake, and Ruedi Reservoirs. The total storage and minimum pool for these existing reservoirs are summarized in Table 4.1.

Other reservoirs such as Shadow Mountain Reservoir and Grand Lake were excluded from the model simulations because of their relatively small impact on streamflows. Additional reservoirs are simulated under future development scenarios. These are: Rock Creek Reservoir, planned by the CRWCD; Iron Mountain Reservoir for the Red Cliff project; Kendig Reservoir for the West Divide Project; and Main Elk Reservoir for oil-shale development.

TABLE 4.1

EXISTING RESERVOIRS

RESERVOIR	TOTAL STORAGE (af)	MINIMUM POOL (af)
Lake Granby 1)	539,760	74,190
Willow Creek Reservoir 1)	10,553	6,675
Williams Fork Reservoir 2)	96,820	3,183
Green Mountain Reservoir 1)	153,639	6,860
Dillon Reservoir 2)	254,000	3,270
Homestake Reservoir 3)	43,500	0
Ruedi Reservoir 1)	102,369	1,089

1) Information from USBR, 1981.
2) Information from DWB, personal communications.
3) Information from City of Colorado Springs, personal communications.

4.2 SIMULATION MODEL RULES

Major assumptions incorporated as operating rules in BESTSM are listed under the following classifications:

Major Demands

- The Grand Valley demand measured at the Cameo gage was assumed to equal to 1650 cfs from April to October of each year and 800 cfs from November to March. (Note: the use of 1650 cfs as the Cameo demand in the model simulation reflects the operation of the river during low flow conditions as experienced in such dry years as 1977. This assumes that in dry years the "check" of the Orchard Mesa Irrigation District is operated (see Section 3.2). It is understood that the operation of the check is a local agreement involving the Grand Valley Water Users, Orchard Mesa Irrigation District, and the Grand Valley Canal to share the river flows among these users during the low flow conditions. There is no agreement as to operation of the check in relation to basin-wide river administration.)

- The Shoshone Hydroelectric Power Plant demand measured at Dotsero is equal to 1250 cfs all year long. (The additional 158 cfs right with a priority date of 1929 was not incorporated because the effect on the monthly flows of the Study would be insignificant).

Green Mountain Reservoir Operation

- The 52,000 af replacement pool of Green Mountain Reservoir has first priority for filling and is reserved to provide replacement water for out-of-priority CBT project diversions.

- Releases from the 100,000 af power pool of Green Mountain Reservoir are used to make up natural water shortages for those irrigation and domestic water rights perfected by use prior to October 16, 1977. For the purpose of this Study, an annual release of up to 66,000 af was utilized during the irrigation season (assumed to be April through October) to satisfy the Grand Valley demand and to relieve shortages to other diversions senior to October 16, 1977 that are caused by the Shoshone or Cameo call.

- The remaining water available from the 100,000 af power pool in Green Mountain Reservoir is used to meet the demands for water sales at a maximum level of 22,800 af/yr as projected in the Draft Water Marketing Program Environmental Statement (RCI, 1985).

- During the non-irrigation season (assumed to be November through April) releases from Green Mountain Reservoir are made for power production and replacement of CBT out-of-priority diversions. Winter releases are used to drawdown the reservoir to a level between 40,000 and 60,000 af by April depending on the anticipated inflows during the snowmelt season.

Effect of Williams Fork and Dillon on Green Mountain Reservoir

- o Williams Fork Reservoir provides replacement for out-of-priority diversions by the Fraser River Diversion Project, the Williams Fork Diversion Project, the Roberts Tunnel, and storage in Dillon Reservoir.

- o Dillon Reservoir is allowed to store out-of-priority ahead of Green Mountain Reservoir under the condition that it satisfy the one-fill requirement of Green Mountain Reservoir by the end of the water year. Calls by Green Mountain Reservoir are met first by releases from Williams Fork Reservoir up to its power plant capacity (assumed to be 300 cfs for this Study). When Williams Fork is not able to provide water, the call is transferred to Dillon Reservoir. These releases are credited against the Green Mountain fill requirement. Any unsatisfied amount owed to Green Mountain is transferred in September from Dillon Reservoir to Green Mountain Reservoir.

Rules for Other Reservoirs

- o Homestake Reservoir is operated to provide water to Aurora and Colorado Springs through the Homestake Tunnel.

- o Under the Existing-Level Use Scenario, Ruedi Reservoir is operated to provide replacement water for Fryingpan-Arkansas Project diversions made out-of-priority. For Future-Level Use Scenarios, the release requirement for water sales is also incorporated.

- o Windy Gap is operated as a transbasin diversion limited by a 600 cfs pumping capacity.

Instream Flow Requirement

- The instream flow requirements were based on estimates provided by the Colorado Water Conservation Board and the Authority in consultation with DWB and CRWCD. They were incorporated in the model and are presented in Table 4.2. For the proposed reservoirs, it is assumed that the releases made to meet downstream demands can be credited to instream flow requirements.

TABLE 4.2

INSTREAM FLOW REQUIREMENTS [1)]

LOCATION	REQUIREMENT
Below Granby Reservoir	20 cfs Oct - Apr
	75 cfs May - Jul
	40 cfs - Aug
	20 cfs - Sep
Below Willow Creek Reservoir	7 cfs Oct - Apr
Below Windy Gap	90 cfs Oct - Sep
Below Williams Fork Reservoir	15 cfs Oct - Sep
Below Dillon Reservoir	50 cfs Oct - Sep
Below Green Mountain Reservoir [2)]	60 cfs Oct - Sep
Below Homestake Reservoir	24 cfs Oct - Sep
Below Hunter Creek Diversion	21 cfs Oct - Sep
Below Ruedi Reservoir	39 cfs Nov - Apr
	110 cfs May - Oct
Below Azure Reservoir [2)]	150 cfs Oct - Sep
Below Red Mountain Reservoir	150 cfs Oct - Sep
Below Wolford Mountain Reservoir [2)]	10 cfs Oct - Sep
At Wolford Colo. R. pump site [2)]	150 cfs Oct - Sep
At Wolcott Eagle R. pump site	45 cfs Oct - Mar
	110 cfs Apr - Sep
At Wolcott Colo. R. pump site [2)]	150 cfs Oct - Sep

1) Information provided by Colorado Water Conservation Board (exceptions noted).
2) Based on estimated or proposed values provided by the Authority in consultation with the DWB and CRWCD.

Reservoir Water Rights

- Water rights decreed or filed for the proposed reservoir sites considered in this Study are summarized in Table 4.3. Although the location of Wolford Site A' is different from that of Site A as specified in the water right decree, for this analysis it was assumed that Site A' had the same priority date as Site A. There are other conditional rights in the vicinity which if transferred may increase the yield of the Muddy Creek sites.

TABLE 4.3

WATER RIGHTS SUMMARY FOR PROJECT RESERVOIRS

RESERVOIR	WATER RIGHT DECREED	WATER RIGHT PENDING	DATE OF RIGHT 1)	OWNER OR CLAIMANT 2)
Wolford Mtn. A'	119,600 af	-	1981	Grand Co.
Wolford Mtn. C.	-	119,600 af	1983	CRWCD, MPWCD
Diversion from Colorado River (Ice Water Pumping Plant and Gore Canyon Power Plant Conduit)	-	2,000 cfs	1983	& Grand Co.
Wolcott	350,000 cfs	-	1971	DWB
Eagle River Pumping Plant	2,500 cfs 4)	-	1971	DWB
State Bridge (Colo. R.) Pumping Plant	3,000 cfs 4)	-	1971	DWB
Una	195,984 af	-	1966	CRWCD
Red Mtn. 3)	-	149,000 af	1984	CRWCD
Azure, Original	25,584 af	-	1962	MPWCD
First Enlargement	63,804 af	-	1967	CRWCD
Total	89,388 af			

1) Year adjudicated or year filed for pending rights.
2) CRWCD - Colorado River Water Conservation District
MPWCD - Middle Park Water Conservancy District
DWB - Denver Water Board
3) Pending claim at same site is named Gabriel Reservoir.
4) Maximum combined diversion rates are not to exceed 3000 cfs.

4.3 MODEL VERIFICATION

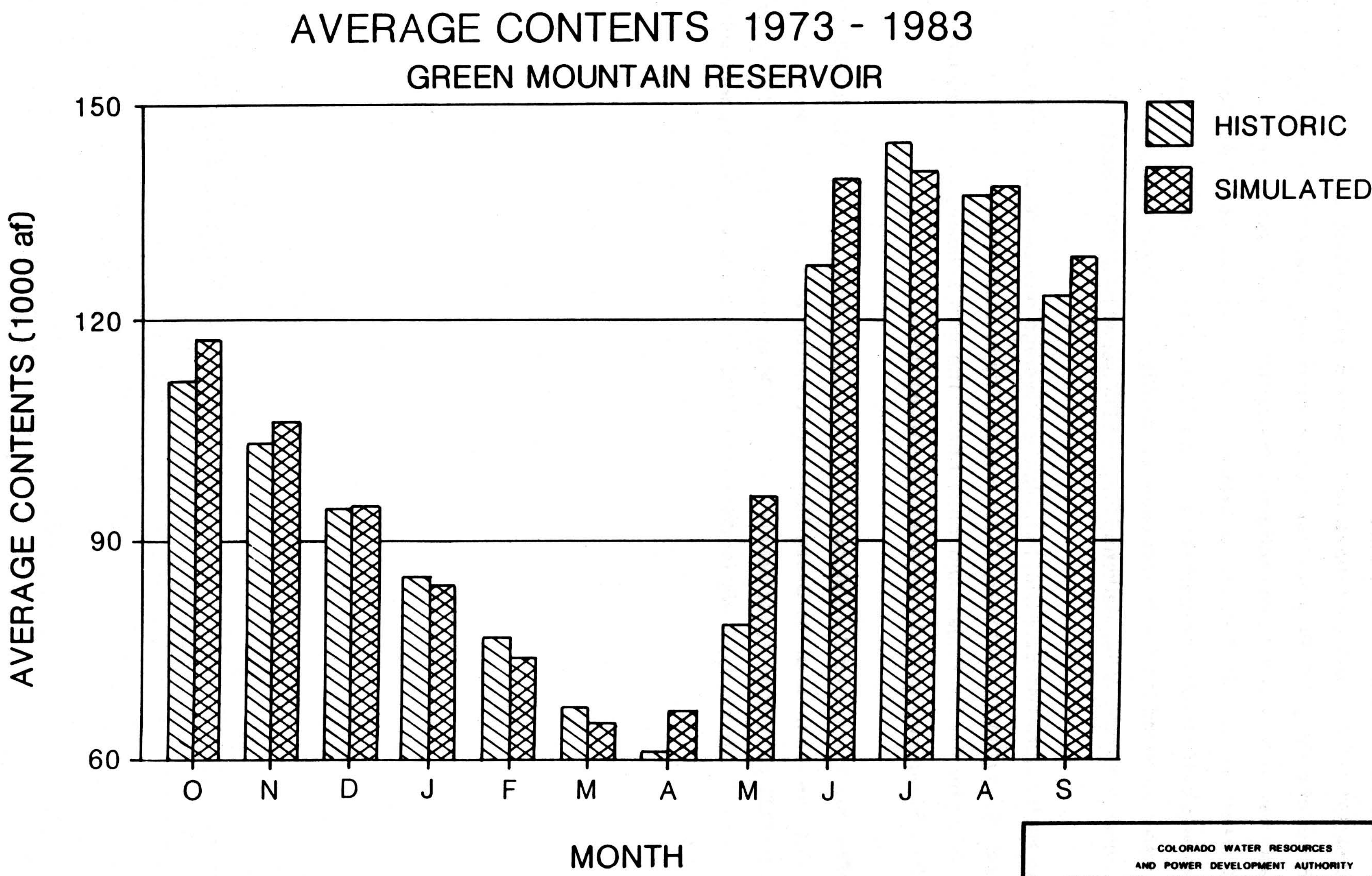
The model was initially applied to historic conditions using the appropriate operation rules for calibration purposes. The basic input data used were virgin flows (historic flows adjusted for depletions due to diversions and reservoir operations) and estimated or recorded diversion data discussed in Chapter 3. The recorded and simulated flows at several locations of the Colorado River were compared. They are presented in Table 4.4. The historic and simulated reservoir contents for Green Mountain Reservoir were also compared. They are shown in Figure 4.1. In both cases, the simulated results are in good agreement with recorded data. Some discrepancies in the Green Mountain Reservoir contents during April through June were due to difficulties in duplicating historical power release schedules which were highly variable during this period. This monthly difference has no impact on the total seasonal releases of the model.

TABLE 4.4

COMPARISON BETWEEN AVERAGE
ANNUAL RECORDED AND SIMULATED FLOWS
(1973 - 1983)

GAGE	RECORDED (1000 af)	SIMULATED (1000 af)
Colo. R. at Sulfur Springs	174	174
Fraser R. at Granby	101	101
Blue R. below Green Mtn. Reservoir	293	290
Colo. R. near Kremmling	728	712
Colo. R. near Dotsero	1,509	1,496
Colo. R. near DeBeque	2,647	2,642
Colo. R. near Cameo	2,727	2,724

In addition to the above comparison of average annual quantities, the comparison was also made between recorded and simulated flows on a monthly basis. The discrepancies were generally less than 5 percent.



COLORADO WATER RESOURCES AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

RESERVOIR CONTENTS FOR GREEN MOUNTAIN RESERVOIR

BOYLE ENGINEERING CORPORATION
Consulting Engineers

FEBRUARY 1987 **FIGURE 4.1**

4.4 GREEN MOUNTAIN RESERVOIR WATER SUPPLY

CBT Replacement Requirements and In-Basin Uses

As discussed in the previous sections, releases from Green Mountain Reservoir are to be used to satisfy out-of-priority CBT Project diversions, to supplement natural flow shortages thereby meeting in-basin use, and for water sales and power generation. In addition, the reservoir supplies supplemental water for the Silt Project, an irrigation project located near Silt, Colorado. An operation study was performed to estimate each of these Green Mountain Reservoir water uses. The demands for water sales and the Silt Project used for this Study are based on figures presented in the Green Mountain Reservoir Water Marketing Program EIS issued by USBR in September, 1985.

For the High Future-Use Scenario, low, high and average release requirements have been estimated. Table 4.5 presents a summary of the results. The in-basin water uses presented in the table include those for the Silt Project and water sales. The total indicates a combination of the replacement for CBT out-of-priority diversion and the in-basin uses.

The USBR has maintained the Colorado River Accounting Sheets since 1963 (USBR, 1963-1982). These include accounts of the Green Mountain Reservoir water uses. The average annual release made for CBT out-of-priority replacement was 21,600 af during the period of 1964 through 1982. The release estimated in this Study for the same period is 21,200 af which compares well with the recorded data.

Table 4.5 indicates that the Green Mountain releases to meet CBT out-of-priority replacement requirements and in-basin use demands are highly variable from year to year. In addition to meeting these requirements, Green Mountain Reservoir makes releases for power generation. Historically the reservoir has been drawn down to between 40,000 and 60,000 af during the winter months of each year. These releases are made for power generation purposes and to evacuate the reservoir in anticipation of spring runoff volumes.

TABLE 4.5

GREEN MOUNTAIN RESERVOIR
ESTIMATED ANNUAL RELEASE REQUIREMENT FOR CBT
REPLACEMENT AND IN-BASIN USES
(Water Years 1951 - 1983)
(1000 af/yr)

	LOW [1)]	HIGH [1)]	AVERAGE
CBT Replacement [2)]	8.9 (1965)	44.7 (1963)	24.0
In-Basin Uses [2)]	8.0 (1970)	92.9 (1956)	27.8
Total	20.4 (1971)	117.9 (1956)	51.8

1) Low and High total demands do not occur in the same year as the Low and High CBT Replacement and In-Basin demands occur. The figure in parenthesis indicates the year of occurrence.

2) Based on High Future-Use Scenario.

Green Mountain Reservoir Firm Yield and Exchange Project Yield

An analysis was performed to estimate the firm annual yield of Green Mountain Reservoir. In this analysis, the release requirements of the reservoir were assumed to be constant from year to year. The firm yield of the reservoir was estimated to be 144,000 af. With 52,000 af reserved as a CBT replacement pool, 92,000 af/yr is available as firm yield from the remaining power pool. These results are summarized in Table 4.6.

The potential Exchange Project yield was also estimated for two pumpback alternatives. One alternative assumed that the entire storage of Green Mountain Reservoir would be made available for pumpback to Dillon, while the other alternative assumed that the active storage of 52,000 af would be reserved in Green Mountain Reservoir for necessary releases to satisfy CBT out-of-priority diversions. The effective storage available for these two alternatives is approximately 147,000 and 95,000 af, respectively. For both alternatives, the minimum flow requirement downstream of Green Mountain Reservoir was assumed to be 60 cfs. In addition, both pumpback alternatives assumed the same level of demand as projected by the Denver Water Department.

For each pumpback alternative, two different conveyance system capacities were evaluated: an 8000 af per month (134 cfs) capacity system which would be conveyed in a 63-inch diameter pipeline and a 12,000 af per month (199 cfs) capacity system which would use a 75-inch pipeline (see Chapter 11 for more detail). The average annual amounts of water pumped to Dillon for the two alternative conveyance capacities are shown in Table 4.6. In addition, the table presents the increased yields to Roberts Tunnel which would result from exchange operation with alternative conveyance system capacities and Green Mountain Reservoir available storage combinations. The increased yields to Roberts Tunnel amount to increased diversions to the Metropolitan Denver Water Supply System. They differ from the potential firm annual yields from the respective Green Mountain Reservoir pools because of the effect of the following:

TABLE 4.6

GREEN MOUNTAIN EXCHANGE PROJECT
AVERAGE ANNUAL YIELD
(1000 af/yr)

	Green Mountain Reservoir Effective Storage [1] Reserved for Exchange (in 1000 af)	
	95 [2]	147 [3]
Potential Firm Annual Yield [4]	92	144
Exchange Project Yield with 8,000 af/mo Conveyance Capacity [5]		
Annual Pumpback Quantity	81	96
Total Increased Annual Yield to Roberts Tunnel	93	111
Exchange Project Yield with 12,000 af/mo Conveyance Capacity [5]		
Annual Pumpback Quantity	84	119
Total Increased Annual Yield to Roberts Tunnel	96	124

1) The total Green Mountain storage is approximately 153,600 af, of which 6,900 af is dead storage. Effective storage excludes dead storage.

2) This alternative assumes that 52,000 af would be reserved in Green Mountain Reservoir for necessary releases to satisfy CBT out-of-priority diversions, while the remaining storage would be made available for the pumpback concept.

3) This alternative assumes that the entire storage of Green Mountain Reservoir would be made available for the pumpback concept.

4) Firm Annual Yield is defined as the quantity of water that can be supplied constantly every year without shortage during the study period of 1951 through 1983.

5) Assumes design capacity of conveyance system from Green Mountain to Dillon Reservoir, 8000 af/mo or alternative 12,000 af/mo. The annual pumpback quantity and increased yield to Roberts Tunnel were estimated based on transmountain diversion demands for the Roberts Tunnel which varies from year to year.

- The minimum flow release requirements from Green Mountain Reservoir assumed for both pumpback alternatives tend to reduce the amount of water that can be pumped from Green Mountain Reservoir to Dillon Reservoir.

- The variation in the diversion requirement of Roberts Tunnel from year to year and the maximum conveyance capacities limit the Roberts Tunnel yield.

- The increased yield to Roberts Tunnel is greater than the pumpback quantities because additional water can be stored in Dillon Reservoir that otherwise would have been released to meet calls by Green Mountain Reservoir.

The conveyance system was operated under the assumption that water could be pumped to Dillon Reservoir as long as water is available in Green Mountain Reservoir and storage space is available in Dillon. As a result, the average end-of-month contents in Green Mountain are reduced under the with-pumpback alternatives when compared against those occurring under the without-pumpback alternatives. The average monthly contents during the study period of 1951 through 1983 in Green Mountain Reservoir varied between 67,000 af and 143,000 af without the pumpback and 17,000 af to 90,000 af under the pumpback alternative utilizing the entire Green Mountain Reservoir storage with the conveyance capacity of 12,000 af/mo. At Dillon, the average end-of-month contents increase slightly, with values ranging from 116,000 af to 198,000 af without the pumpback, to 116,000 af to 202,000 af with the pumpback. Although the conveyance system could be operated in a manner to minimize reservoir fluctuations for environmental and recreational considerations, this additional mode of operation was beyond the scope of this Study.

4.5 RESERVOIR YIELDS UNDER ALTERNATIVE CONDITIONS

The hydrologic simulation model was applied to estimate the firm annual yield of each proposed reservoir under the three development scenarios discussed in Chapter 3. The six proposed reservoir sites selected for reconnaissance investigation are shown in Table 2.1.

For the purpose of this Study, the firm annual yield is defined as the consistent quantity of water that can be supplied every year without any shortage during the study period of 1951 through 1983. It was assumed that the monthly release schedule would be uniform throughout each year.

The study period (1951-1983) includes the two significant drought periods of 1954-1956 and 1977. These become critical periods for determining the firm annual yield. Through drought frequency analysis, it was estimated that the 1954-1956 drought would occur in the study area once in 50 to 100 years while the 1977 drought would occur in the study area once in 30 to 50 years depending on the location of the stream. Accordingly, the firm annual yield presented in this report indicates the amount of water that can be supplied during these two drought periods.

Diversion Capacities

Wolford Mountain Site A' and Wolcott were analyzed with diversions from adjacent streams to augment natural flows at the sites. The Wolford Mountain Site A' analysis considered an alternative of diverting water from the Colorado River below the confluence with the Blue River to augment the storage of Muddy Creek flows. Wolcott Reservoir involves diverting water from the Eagle River and the Colorado River or, alternatively from the Eagle River only.

The Diversion capacities selected for this Study were:

Storage Site	Diversion Source	Pumping Capacity
Wolford A'	Colorado River	150 cfs
Wolcott	Eagle River	600 cfs
Wolcott	Colorado River	600 cfs

The pumping capacity of the diversion from the Colorado River for Wolcott Reservoir, 600 cfs, was based on optimization of capital and operation costs by Parsons et al., (1974). The other capacities were selected as a result of a similar optimization analyses of reservoir yields in relationship with construction and operation costs for diversion facilities.

Minimum Pool

Useful storage for the determination of firm yield is the reservoir storage capacity less the minimum pool. The minimum pool requirements for each reservoir utilized in this Study is the structural allowance or the sediment storage allowance, whichever is greater, as determined by previous investigations (see Chapters 5 through 10 describing the individual reservoirs). For reservoirs that lacked this information, it was assumed that a minimum pool equal to 4 percent of capacity would contain estimated sediment yields for a 50-year period. Minimum pool capacity at the six proposed reservoir sites is shown on Table 2.1.

Yields of Single Reservoirs and Combinations of Reservoirs

Under each development scenario, two different conditions were assumed for the operation of Green Mountain Reservoir. One condition, referred to as "Without Pumpback", assumes that Green Mountain Reservoir will continue to operate in accordance with Senate Document No. 80. The other condition assumes implementation of the pumpback alternative which utilizes the entire Green Mountain Reservoir storage. This second condition reflects one of the alternatives envisioned under the Green Mountain Exchange Project concept. Firm annual yields for the six reservoirs selected for reconnaissance investigation and for four combinations of reservoirs are presented in Table 4.7. Yields were also estimated for other capacities of these reservoirs and are reported in Appendix A.

Capability of Reservoirs for Joint-Use or Exchange

It is evident from the table that all six reservoir sites can produce the necessary yield to meet the Joint-Use Reservoir requirement (approximately 30,000 af). With the exception of the Wolcott Reservoir alternative with diversions from both Colorado and Eagle Rivers, estimated yields of single reservoirs are much less than the 144,000 af yield of Green Mountain Reservoir, and fall short of satisfying the replacement objective of the Green Mountain Exchange Project. To obtain greater yields, representative combinations of reservoirs were analyzed. Comparison of alternative reservoirs is discussed in Chapter 13.

TABLE 4.7
ESTIMATED RESERVOIR FIRM ANNUAL YIELD [1)]
UNDER VARIOUS SCENARIOS (1000 af/yr)

		PROJECTED EXISTING		MODERATE FUTURE		HIGH FUTURE	
RESERVOIR[2)]	CAPACITY (1000 af)	W/O PUMPBACK	W/ PUMPBACK[3)]	W/O PUMPBACK	W/ PUMPBACK[3)]	W/O PUMPBACK	W/ PUMPBACK[3)]
Wolford Mountain A'	120	41	41	40	40	40	39
w/Colo. diversion	120	54	47	49	44	49	43
Wolford Mountain C	60	26	24	25	24	25	23
Red Mountain	140	59	58	58	56	56	54
Wolcott w/diversion from:							
Eagle & Colo.	350	155	149	151	143	138	135
Eagle only	160	83	75	81	70	69	65
Azure	85	53	53	51	51	48	48
Una	150	105	105	105	105	105	105
Red Mountain & Wolcott w/Eagle diversion	300	98	90	97	90	95	89
Azure & Wolcott w/Eagle diversion	245	133	121	129	121	120	114
Red Mountain & Una	290	164	163	163	161	161	159
Una & Wolford A' w/Colo. diversion	270	159	152	154	149	154	148

1) Firm Annual Yield is defined as the quantity of water that can be supplied every year without shortage during the study period of 1951 through 1983. Quantity expressed in thousands of acre-feet per year.

2) Based on conveyance system pumpback to Dillon Reservoir drawing upon the entire Green Mountain Reservoir.

3) Diversion involves pumping from Colorado R. or Eagle R. or both rivers, as cited.

5.0 WOLFORD MOUNTAIN SITE A' DAM AND RESERVOIR

Wolford Mountain Site A' is located on Muddy Creek, 1.5 miles north of Kremmling. Figure 5.1 presents a plan view of the Wolford Mountain Site A' area.

Following is a tabulation of descriptive and dimensional data:

Dam type	Embankment
Height of dam	140 feet
Dam crest length	3000 feet
Reservoir volume	120,000 af
Land required	2750 acres
Highway relocation	0.8 miles

In addition to capturing flows from Muddy Creek, water could also be pumped into the reservoir from the Colorado River. Excess flows from the Colorado River would be conveyed in a buried conduit and pass through the ridge by tunnel into Wolford Mountain A' Reservoir. This would require an intake below the confluence of the Colorado and Blue Rivers, 2 miles west of Kremmling, and a pump station to lift the flow about 160 feet to the average reservoir level.

5.1 SITE TOPOGRAPHY

An east-west trending ridge forms a 280-foot high southward facing bluff overlooking the town of Kremmling. The north slope of this ridge would form the right abutment of the dam. From the dam site, Muddy Creek circles the east end of the bluff and then flows west between the bluff and the town to join the Colorado River 1 mile downstream from Kremmling.

5.2 STUDIES AND FIELD INVESTIGATIONS

The "Wolford Mountain Reservoir Project Feasibility Report", (Western, 1983), provided reconnaissance level investigation of four sites on Muddy Creek, designated A, B, C and D. Site A was located 3800 feet downstream of Site A'.

During the course of this Study, field and laboratory geotechnical investigations were conducted at Wolford Mountain Site A' by Chen & Associates. Three core borings and 15 auger borings were drilled to define the subsurface stratigraphy and physical characteristics of the geologic formations at the dam axis, the tunnel, a saddle dam location and potential construction material borrow areas. Exploratory boring locations, logs, test results and a geologic dam site profile are included in Appendix C.

5.3 GEOLOGICAL DESCRIPTION

Wolford Mountain Site A' is located near the western edge of the Middle Park Basin, a structural sag that formed as the Park Range was uplifted to the west and the Front Range was uplifted to the east. These major geologic structures, along with several north-trending thrust faults, were formed during the period of mountain building, known as the Laramide Orogeny, that occurred in the late Cretaceous to Eocene geologic epochs about 40 to 70 million years ago.

After the Laramide Orogeny, the Middle Park Formation, which is comprised largely of sandstone, conglomerate and shale, was deposited as basin fill derived from materials eroded from the adjacent uplifted formations. Under the Middle Park Formation are older, pre-Laramide sedimentary rocks. Underlying these sedimentary rocks are Precambrian crystalline rocks, over 600 million years old. Within the basin, the Precambrian rocks are exposed in places where they have been brought up by overthrusting along the Laramide thrust faults. This is displayed on Wolford Mountain, north of the site, where the Williams Range Thrust Fault has brought older, dark Precambrian granite over the younger, light tan shale.

The dam would be constructed across a 500-foot wide valley eroded into the sandstone and claystone shale members of the Pierre Shale. The sandstone members of the Pierre Shale have formed very steep slopes where the creek has incised its way through the rock. The claystone shale members are less resistant and have weathered to flatter slopes.

Alluvium covers the valley floor and terrace alluvium overlies bedrock above the valley floor. Colluvium, derived from upslope soils and rock and transported to its present location by slope wash processes, covers the Pierre Shale on many slopes above the present stream channel.

5.4 DESIGN CONSIDERATIONS

Preliminary evaluation indicated the availability of moderately impermeable earth materials and a large quantity of permeable materials. Little, if any, aggregate for concrete was observed. Spillway capacity requirement was not unusually large. This suggested that the site was most suited for an earth embankment dam. Design considerations addressed in this section include foundation conditions, earthquake hazard, spillway requirements and construction materials.

Surface Material

Based on borings in the valley bottom, it is anticipated that up to 30 feet of alluvial material could be encountered above the shale bedrock. The material consists of interbedded silts, silty sands, clayey sands with scattered gravels, and sandy clay. Loose, fine grained deposits were encountered in many locations. The unconsolidated materials encountered would not be suitable for support of a dam.

Rock Strength

The Pierre Shale bedrock at the site consists of dark gray shales with several fine grained, thinly bedded sandstone units 20 to 50 feet in thickness. The sandstone is generally moderately cemented. The results of unconfined compressive strength tests performed on intact specimens of rock core varied from 8400 to 13,300 pounds per square inch (psi). This information, along with data pertaining to the frequency, orientation and general nature of the discontinuities in the rock mass, indicated a relatively low rock mass strength. Low strength is associated with a potential for settling. In the area under the core zone the removal of several feet of weaker rock should be anticipated.

Foundation Permeability

In-place packer permeability tests were conducted in the shale and sandstone bedrock in two borings. Test results indicate that the maximum permeability varied from 0 to 361 feet per year, which is a relatively low value. Reported permeability values were relatively low for the upper 20 feet of the foundation rock where both the percent core recovery and rock quality designation were low. Though supported by very little data, it would appear that while the rock is fractured and broken in the upper 20 feet, the discontinuities are relatively tight.

Seismic Considerations

To develop dam stability during an earthquake, design criteria must consider the proximity of active faults as well as probable magnitude of the seismic event. Preliminary evaluation of the earthquake potential in Colorado classified several faults in the vicinity of the Kremmling area dam sites as potentially active faults (Kirkham and Rogers, 1981). These include the Gore Fault, the fault along the west side of the Williams Fork Valley, the Frontal Fault and the Antelope Pass Fault. A brief description of each fault is presented in Appendix B. The Gore and Frontal Faults are approximately 9 miles from Wolford Mountain Site A' at their closest points. The Williams Fork Valley Fault is approximately 6 miles and the Antelope Pass Fault is approximately one mile from the dam site. Without detailed investigations of the faults relevant to dam design, a conservative design approach should be used. For this Study, it was assumed that the nearby faults are potentially active.

For preliminary design, it is appropriate to use slopes on the faces of the dam that are less steep than might be recommended after thorough analysis. Allowance should be included in the cost estimates for removal of materials that might not support earthquake induced forces from the area under the embankment. For final design, additional geologic seismic investigations would be conducted and a probabilistic analysis carried out to refine the design earthquake magnitude and distance from the site. The design dam section can then be analyzed for dynamic loading using the selected accelerogram time histories for both horizontal and vertical earthquake-induced forces.

Spillway Requirement

The spillway size for the Wolford A' Dam was based on the probable maximum flood (PMF) of 85,500 cfs estimated by Morrison-Knudsen Engineers, Inc. (1986) in their work at Site C, 3.8 miles upstream. Attenuation of the peak inflow by routing through the reservoir resulted in a reduced outflow for spillway design of 24,000 cfs.

Construction Materials

Low permeability materials for use in an embankment may be obtained from the alluvial deposits in the valley bottom upstream of the dam or from the colluvial deposits on the side slopes of the reservoir. Semipermeable and permeable sands and gravels for the embankment may be obtained from terrace deposits on the slopes above the valley.

Terrace deposits of sand and gravel appear to be well suited for use as shell material, but should be expected to contain significant quantities of silt. Determination of the suitability of the local deposits for filter, drain and transition materials will be left to final design. It is prudent at this point to assume that the material will be unsuitable and continue preliminary planning on the assumption that the materials would be obtained from sources off site. On site material is expected to be contaminated by shale and not sufficiently durable to serve as concrete aggregate.

There is no tested source of riprap in the Kremmling area. It is possible that a quarry for riprap could be developed in the granitic rocks present on Wolford Mountain and on the east side of Red Mountain or Junction Butte. Haul distance to the site would be 3 to 8 miles.

5.5 PROPOSED DAM AND SPILLWAY

Based on the foundation conditions and the materials potentially available for dam construction, a modified homogeneous earthfill dam design has been selected for the Wolford Mountain Site A'. Depending on the quantities of material actually available, determined by more comprehensive field investigations, an alternative zoned embankment design with a core zone and granular shells might also be considered.

Embankment Section

The slopes for the embankment have been set at 2.5:1 for the downstream slope and 3:1 for the upstream slope. The downstream slope is governed by the strength properties of the embankment material and the foundation rock mass strength. The upstream slope is governed by these properties as well as the capacity of the embankment material to drain. If it remained saturated, instability might result under conditions of reservoir drawdown.

Filter, drain and transition zones have been incorporated into the preliminary design section. The filter provides protection against internal erosion of the core zone into the downstream shell zone and foundation under the action of seepage. The drain zone conducts the seepage that passes through the core and foundation out of the embankment in a controlled manner and isolates the phreatic surface from the downstream shell. The transition zone acts as a filter media between the upstream shell and core zones for drainage from the core during reservoir drawdown, and in the event that cracks develop in the core, material will be transported into these cracks from the transition zone thereby reducing the amount of seepage that would otherwise occur.

The upstream face of the embankment would be covered with riprap, consisting of cobble and boulder size rock, to protect the upstream shell zone against wave-action erosion. The downstream face should be planted with natural grasses to control surface erosion.

The stability of the upstream and downstream slopes were evaluated at sections where the orientation of the foundation bedding planes appeared to be most unfavorable. The results of the preliminary analyses yielded adequate safety factors for preliminary design, thus confirming that the selected embankment slopes are appropriate.

Spillway Description

The spillway is designed as an uncontrolled Ogee section. The sill elevation would be 7476 feet with a length of 120 feet. During the peak outflow of 24,000 cfs the depth of flow would be 13 feet. A deflector bucket would be provided at the downstream end of the spillway apron for energy dissipation.

Saddle Dam

A saddle dam would be needed 0.8 miles west of the main dam to contain the reservoir during the maximum flood event. Only about 2 feet high, it would be constructed as part of the access road.

Foundation

Standard penetration tests in the valley bottom indicated that the alluvial material is very loose in many locations. Due to the potential for collapse of loose deposits subsequent to construction of the dam and reservoir impoundment, as well as the potential for liquefaction of loose cohesionless materials when subject to seismic loading, the approach adopted at this level of study has been to excavate the alluvium and colluvium from beneath the embankment.

On the basis of the limited data available on subsurface conditions at the site, the permeability of the foundation rock appears to decrease relatively rapidly with depth. Nevertheless, in order to minimize the possibility of uncontrolled seepage through the foundation estimates have been made for a single line grout curtain beneath the core zone of the dam, and limited rim grouting in the abutments and at the saddle dam site.

5.6 DAM AND RESERVOIR APPURTENANCES

Proposed appurtenances include the outlet works and the pumping and conveyance facilities to supplement the yield from regulation of Muddy Creek flows with off-stream storage of snow-melt flows of the Colorado River. Also presented for consideration is a potential pumped-storage site for hydropower.

Outlet Works

The outlet works would be converted from the conduit used to divert flows during construction. It would consist of a conduit placed in a cut-and-cover trench excavated to bedrock under the embankment. A conduit diameter of 10 feet is projected to serve for construction diversion. It would be completed as an outlet works by inserting a 6.5-foot diameter outlet pipe, and leaving an access way along side to the guard valve. The intake structure would be built and, at the downstream termination, a 60 inch Howell-Bunger valve would be installed to control the design discharge of 1630 cfs.

Colorado River Diversion for Off-Stream Storage

A pump station on the Colorado River would divert snow-melt runoff flows to Wolford Mountain Site A' Reservoir to supplement the yields obtained from Muddy Creek. The intake and pump station would be located at the entrance to Upper Gore Canyon. From the pump station, north of the Denver and Rio Grande Grande Western Railroad, an inlet conduit would pass under the tracks to an intake structure in the river. Two 2500-horsepower pump units could pump the projected flow of 150 cfs to the reservoir against a maximum dynamic head estimated at 230 feet.

The length of pipeline to the reservoir would be 20,600 feet which includes 5700 feet of tunnel. The tunnel affords a shorter route and reduced pumping lift. Required replacement flows would be released through the tunnel and pipeline to the pump station and then to the

Colorado River at a lower rate of flow over a period of approximately six months. Pumps would be capable of reverse flow for power generation upon reservoir releases. Power revenues could offset approximately 20 percent of pumping costs.

Pumped-Storage Hydropower Potential

For pumped-storage hydroelectric power generation, two reservoirs are needed, located a short distance apart but separated in elevation. Water is released from the upper reservoir to generate electricity during the work day and early evening when electrical power demand is at its peak. When electrical demand is low, at night and weekends, water is pumped back to the upper reservoir from the lower reservoir to replenish the water supply. Pumped-storage hydropower is financially attractive in many instances because the peak-demand energy is much more valuable than the off-peak energy. The shorter the distance between reservoirs in relation to the elevation difference, the greater the potential financial return.

Exceptional topography at Wolford Mountain Site A' may provide pumped-storage hydroelectric potential. A possible upper reservoir site on Wolford Mountain has been identified 1.4 miles east of an upper reach of the Site A' reservoir and 1300 feet higher in elevation. This distance to elevation-difference ratio of 5.5:1 falls within the generally accepted topographic criteria of reservoir sites which have been found financially attractive. The site topography indicates a potential plant capacity in the 500 megawatt (MW) range. No field investigation has been conducted. A reconnaissance-level investigation may be justified to indicate the geologic suitability, the potential market for peaking power and a financial analysis of the potential for pumped-storage hydropower development at this site. Reserving water in Wolford Mountain Site A' Reservoir for pumped-storage use would accordingly reduce the reservoir yield as a Joint-Use or Replacement Reservoir.

5.7 HIGHWAY RELOCATION

Sections of Highway U.S. 40 cross small arms of the proposed reservoir. New highway sections would be constructed on fill over the present roadway to an elevation above the future water level. The length of the highway involved is approximately 4200 feet.

5.8 SUMMARY OF ESTIMATED OPERATION AND CONSTRUCTION COSTS

An embankment dam at Wolford Mountain Site A', with a dam height of 143 feet and crest length of 3000 feet, would impound a reservoir with a capacity of 120,000 af. Reconnaissance-level cost estimates have been developed for both operation and construction costs. The detailed cost estimates and their derivation are described in Appendix B.

Dam Operation and Pumping Cost Estimate

Dam operation and upkeep would be carried out by a damkeeper, an annual technical inspection team and services of specialty contractors. The estimated average annual cost of dam operation and upkeep at the proposed Wolford Mountain Site A' Dam and Reservoir is $110,000. Recreation-type services are assumed to be offset by user charges and are not included in this estimate.

The system to divert water from the Colorado River would require full time operators during the approximately two months of pump operation. One operator with supervision would handle the reservoir release operations during the remainder of the year. Specialty services are required for pump station and pipeline repairs. Energy consumption for pumping is estimated to be partially offset by energy generation from release flows. Annual operation costs are summarized in Table 5.1. Details of annual pumping and dam operation costs are in Appendix B.

TABLE 5.1

WOLFORD MOUNTAIN SITE A'
SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Dam Operation and Upkeep		$ 110,000
Pumping Labor and Maintenance		130,000
Operators and Station Labor	78,500	
Equipment and Supplies	9,000	
Maintenance and Repair	42,500	
Pumping Energy		510,000
Credit for Generation		(100,000)
Total Estimated Annual Operation Cost		$ 650,000

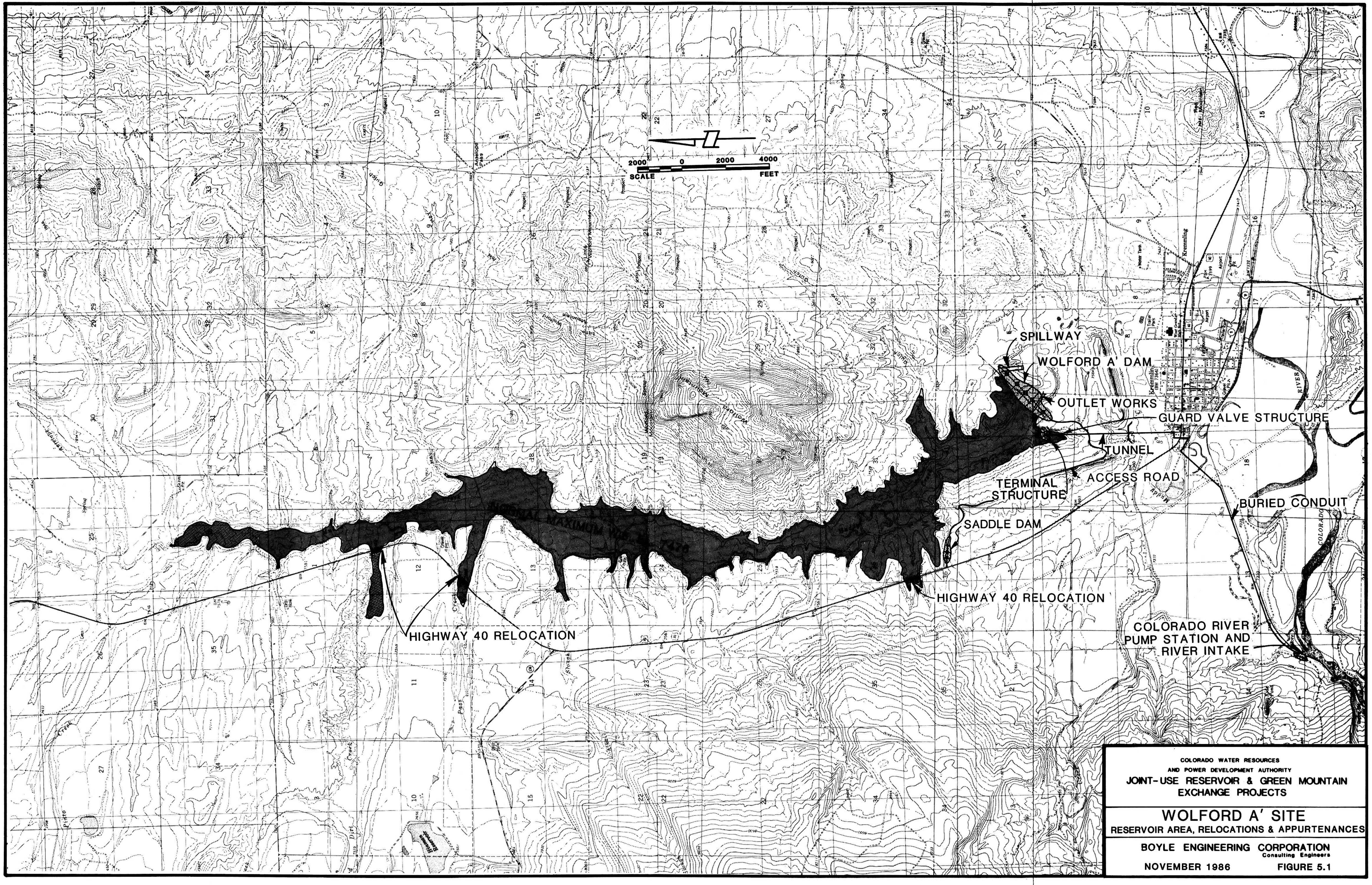
Estimated Construction Quantities and Costs

Estimated construction quantities, prices, and allowances have been combined to obtain a total construction cost. The costs are indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing have been added to obtain the total investment cost. (Section 13.3). Estimated construction quantities and estimated costs are summarized in Table 5.2

TABLE 5.2

WOLFORD MOUNTAIN SITE A' DAM AND RESERVOIR
SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	2,750	acres	$ 2,350,000
River Diversion			300,000
Dam and Spillway			35,460,000
Shell Volume	1,990,000	cubic yards	
Core Volume	1,613,200	cubic yards	
Drain Volume	108,000	cubic yards	
Filter Volume	112,800	cubic yards	
Transition Volume	108,000	cubic yards	
Excavation	2,362,000	cubic yards	
Structural Concrete Volume	19,700	cubic yards	
Mass Concrete	4,300	cubic yards	
Outlet Works			2,890,000
Relocations			
State Highway	0.8	miles	1,200,000
Transmission Line	0.6	miles	200,000
Pumping System			17,100,000
Pumps and Motors	2	each	
Structure Concrete	3,060	cubic yards	
6-foot diameter pipe	5,710	feet	
4.5-foot diameter pipe	14,900	feet	
Excavation	129,700	cubic yards	
Subtotal of Direct Costs			$ 59,500,000
Contingencies (25%)			14,900,000
TOTAL DIRECT COST			$ 74,400,000
Engineering, Legal, Administration (20%)			14,900,000
TOTAL CONSTRUCTION COST			$ 89,300,000
Interest During Construction (2 yrs.)			7,100,000
TOTAL CAPITAL COSTS			$ 96,400,000
Financing and Reserve			13,200,000
TOTAL INVESTMENT COST			$ 109,600,000



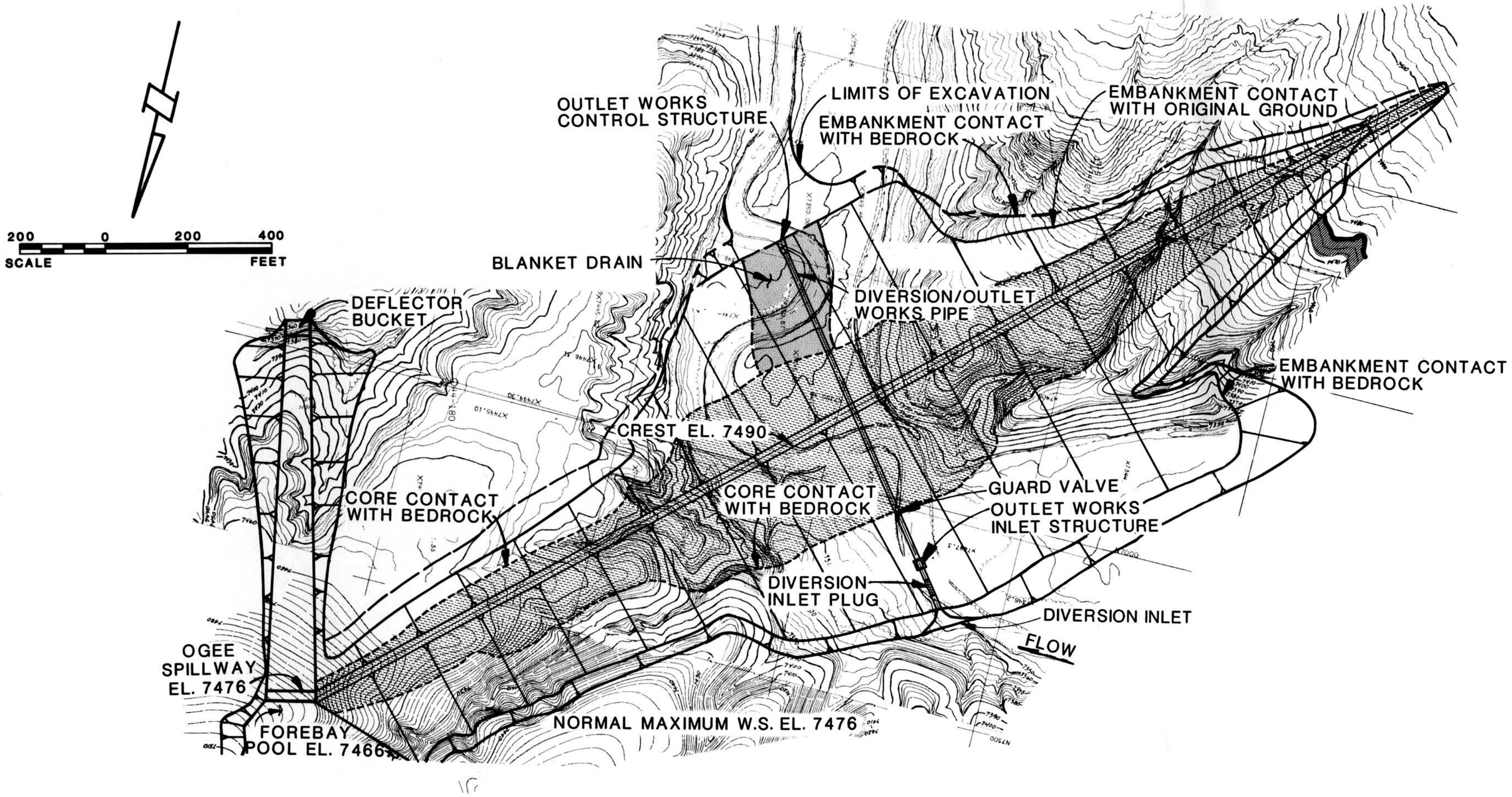
COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

WOLFORD A' SITE
RESERVOIR AREA, RELOCATIONS & APPURTENANCES

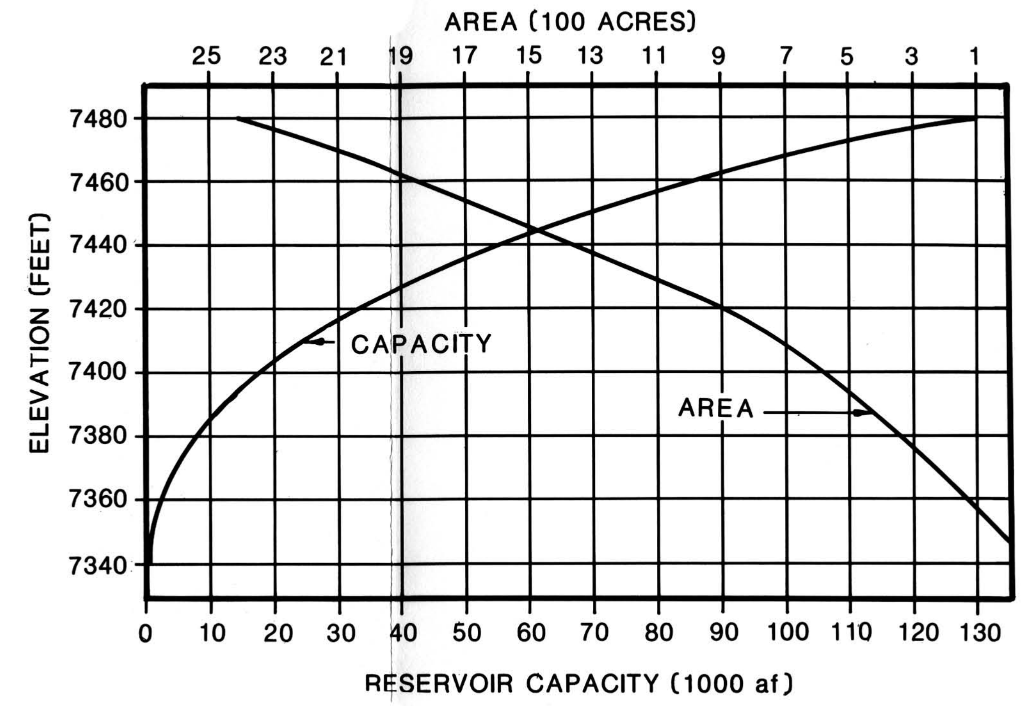
BOYLE ENGINEERING CORPORATION
Consulting Engineers

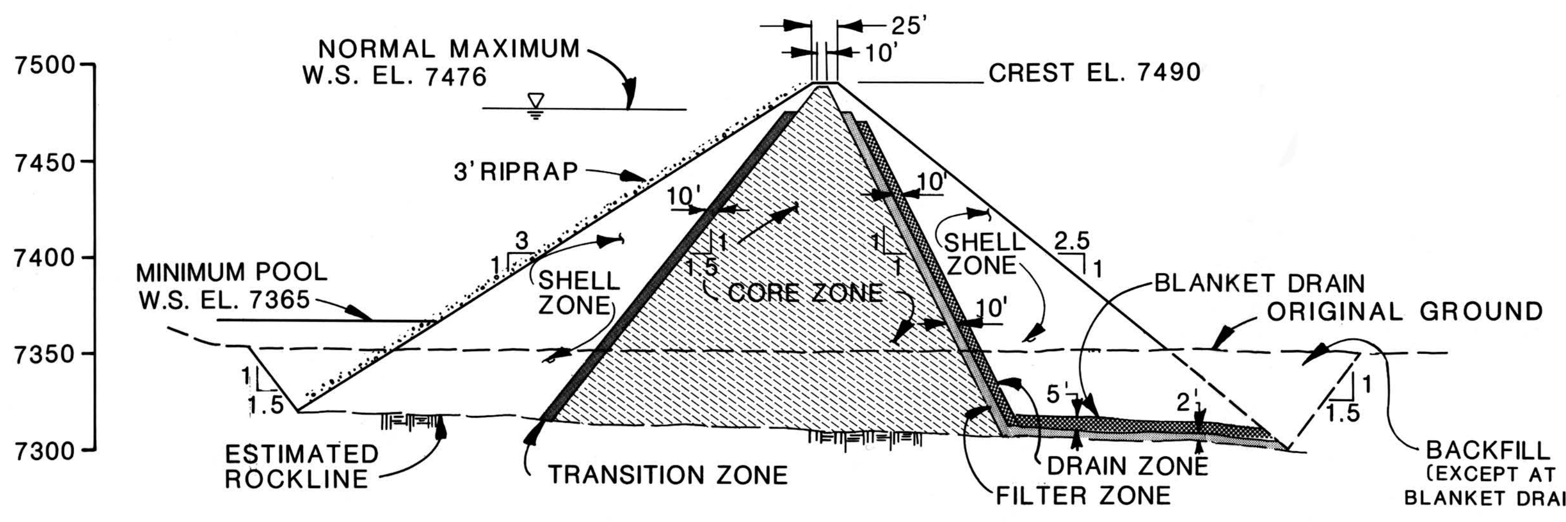
NOVEMBER 1986

FIGURE 5.1



DAM LAYOUT





TYPICAL DAM SECTION

100 0 100 200
SCALE FEET

COLORADO WATER RESOURCES AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

WOLFORD A' DAM

LAYOUT AND SECTION

BOYLE ENGINEERING CORPORATION
Consulting Engineers

FEBRUARY 1987

FIGURE 5.2

6.0 WOLFORD MOUNTAIN SITE C DAM AND RESERVOIR

Wolford Mountain Site C is located on Muddy Creek, 5 miles north of Kremmling, and 3.8 miles upstream from Site A'. Figure 6.1 presents a plan view of the Wolford Mountain Site C area.

Following is a tabulation of descriptive and dimensional data:

Dam type	Embankment
Height of dam	120 feet
Dam crest length	1700 feet
Reservoir volume	60,000 af
Land required	1900 acres
Highway relocation	0.9 miles

6.1 SITE TOPOGRAPHY

The dam site is a 250-foot wide canyon with sides that rise steeply to a height of approximately 80 feet. At that point, the left abutment is relatively flat for about 500 feet, continuing east to the base of Wolford Mountain. The right abutment slopes gently upwards towards Highway U.S. 40, located 0.7 miles west of the site. The full reservoir water surface elevation of 7485 feet at Site C is 10 feet higher than the water surface proposed for an alternative reservoir at Site A'.

6.2 STUDIES AND FIELD INVESTIGATIONS

A report, "Rock Creek Dam Project", prepared by Morrison-Knudsen Engineers, Inc. (1986) for the Colorado River Water Conservation District (CRWCD) investigated the feasibility of a reservoir of 46,800 af capacity at Site C on Muddy Creek.

The report titled "Seismotectonic Hazard Evaluation, Rock Creek Project Near Kremmling, Grand and Routt Counties, Colorado", was prepared by Michael West and Associates (1986) for Morrison-Knudsen Engineers as part of their study for the CRWCD.

A comparative review of capacities and costs of a dam at Site C and the proposed Rock Creek Dam, located 13 miles to the west, was conducted for the Municipal Subdistrict of the Northern Colorado Water Conservancy District (Swaisgood, 1983).

The "Wolford Mountain Reservoir Project Feasibility Report", prepared by Western (1983), provided reconnaissance-level geological, hydrological and construction cost information about the original Site C and three other sites downstream. Because of an old landslide on the left abutment of the original site, the "C" site was moved 1500 feet upstream to the present location.

As part of the study of Wolford Mountain Site C by Morrison-Knudsen Engineers (1986), field and laboratory investigations were performed. These included a geophysical survey, three core borings along the proposed dam axis, and four test pits in potential embankment material borrow areas. Exploratory boring locations, logs and test results are included in the report, Rock Creek Dam Project.

During the course of this Study the existing geological information was reviewed and correlated with field observations during two site visits. The results of this review are incorporated into this report.

6.3 GEOLOGICAL DESCRIPTION

Wolford Mountain Site C is located near the western edge of the Middle Park Basin. The regional geology is identical with Site A' and is described in Section 5.3.

The Site C Dam would be constructed across a 250-foot wide valley, with nearly vertical shale cliffs. Valley alluvium covers bedrock at the present location of Muddy Creek. Terrace alluvium overlies bedrock above the valley floor. Lower units of the Pierre Shale bedrock are exposed in the cliffs on the valley sides, but in most places the valley sides are overlain by colluvial deposits washed from upslope rock.

6.4 DESIGN CONSIDERATIONS

Preliminary evaluation indicated that impermeable earth materials suitable for the core of an embankment dam are available. The topography is suitable for an abutment spillway location. These are factors favorable to selection of an earth embankment design. A roller compacted concrete (RCC) design was also considered, however, laboratory tests of on-site aggregates indicated marginal durability (Morrison-Knudsen Engineers,1986). Design considerations addressed in this section include foundation conditions, earthquake design, spillway requirements and construction materials.

Surface Material

The alluvium encountered in the valley floor was 9 to 12 feet thick and consisted of clayey sand, medium grained sand, and well rounded granite and basalt cobbles. A bore hole indicated that the terrace material on the right abutment is about 35 feet thick, overlying a sound dark gray mudstone bedrock. A geophysical survey near the proposed spillway alignment on the left abutment and a left abutment borehole, indicated that approximately 25 feet of terrace material overlies about 15 to 35 feet of slightly weathered but jointed shale.

Rock Strength

The Pierre Shale is a hard gray mudstone or shale which has weathered to a friable, flaky, slivery material with separations along bedding planes where it is exposed. Drilling indicated that the upper 8 to 10 feet of bedrock in the floor of the valley was slightly to moderately weathered. Below that depth the bedrock was massive, dark gray, sound mudstone or shale.

The top 5 feet of the bedrock of the right abutment was found to be moderately to highly jointed along bedding planes. The top of the bedrock on the left abutment, unlike the right abutment, was highly weathered to 6 feet and moderately jointed to a depth of about 25 feet below the contact with the gravel. Below the weathered zone, the borehole revealed the rock to be sound, massive mudstone, but more highly jointed than that found under the right abutment and in the canyon floor.

The results of unconfined compressive strength tests performed on specimens of rock core varied from 3,153 to 10,667 psi. Based on the information available, for the purpose of preliminary stability analyses, a relatively low rock mass strength was estimated for the upper 20 to 30 feet of the foundation. Low strength is associated with a potential for settling. In the area under the core zone the removal of several feet of weaker rock should be anticipated.

Foundation Permeability

In-place packer tests conducted below a depth of 14 feet in the bedrock of the valley floor showed the foundation to be impervious. Tests in the bedrock of the right abutment indicated the foundation was impermeable from 5 to 15 feet below the top of rock and only slightly permeable below that. Tests performed in the boring on the left abutment reflected the higher degree of fracturing in the rock through uniformly high water takes.

Design Earthquake

To develop dam stability during an earthquake, design criteria must consider the proximity of active faults as well as probable magnitude of the seismic event. A preliminary evaluation of the earthquake potential in Colorado classified several faults in the vicinity of the Kremmling area dam sites as potentially active faults (Kirkham and Rogers, 1981). These include the Antelope Pass Fault, the Williams Fork Valley Fault, the Frontal Fault, the Gore Pass Fault, and the Gore and Steamboat Springs Fault Zones which bound the western flank of the Park Range uplift. The Frontal Fault and the Gore and Steamboat Fault Zones are approximately 15 miles from the Wolford Mountain Site C at their closest points. The Williams Fork Valley Fault is approximately 12 miles, the Gore Pass Fault is approximately 4 miles and the Antelope Valley Fault is approximately 2 miles from the dam site.

It has been concluded by Michael West and Associates (1986) that a conservative assessment of potential earthquake hazard for the Wolford Site C would be represented by a hypothetical floating earthquake of a Richter magnitude 5.5, at a hypocentral distance of 10 kilometers (6.2 miles) from the site.

For final design, it is recommended that additional geologic seismic investigations be conducted and a probabilistic analysis be carried out to confirm the design earthquake magnitude and distance from the site. The design dam section can then be analyzed for dynamic loading using the selected accelerogram time histories for both horizontal and vertical earthquake-induced forces.

Spillway Requirements

The spillway size for Wolford C Dam is based on the PMF of 85,400 cfs estimated by Morrison-Knudsen Engineering, Inc., (1986). Attenuation of the peak flow by routing through the reservoir resulted in a reduced outflow for spillway design of 40,900 cfs.

Construction Materials

Field investigations conducted by Morrison-Knudsen (1986), indicate that low permeability materials, suitable for the core zone of an embankment dam, appear on the right side of the broad valley about one half mile upstream of the dam axis. Material excavated at this level should not require extensive drying to prepare it for compaction. A sample tested in the laboratory showed the material to be more than 90 percent silt and clay.

A deposit of material located downstream of the proposed left abutment was investigated as a potential source of material for the shell zones of the embankment. Index property tests on samples obtained from this deposit indicated that the samples were reasonably well graded sands and gravels, with a maximum particle size of 6 inches, containing up to 30 percent fines.

The terrace deposit on the right abutment at the dam axis was evaluated for use as material for roller compacted concrete aggregate. Tests performed on the samples cast doubt on the durability of the material. Additional tests must be conducted to determine whether local materials are suitable for use as aggregate, filter and drain material. For cost estimates, hauling from sources along the Colorado River was assumed.

There is no identified source of riprap available in the Kremmling area. It is possible that a quarry for riprap could be developed in the granitic rocks on Wolford Mountain and on Red Mountain and Junction Butte. A haul distance of 12 to 15 miles was assumed for cost estimates.

6.5 PROPOSED DAM AND APPURTENANCES

Based on the foundation conditions and the materials potentially available for dam construction, a zoned embankment dam has been proposed for the Wolford Mountain Site C with an interior sloping core zone. The granular shells would constitute the majority of the volume.

Embankment Section

A moderately sloping core zone has been selected to conserve core material because quantities at the site may be relatively limited. Such a design would also minimize the possibility for adverse deformations in the core resulting from consolidation during and subsequent to construction. The external slopes of the embankment have been set at 2.5:1 for the downstream slope and 3:1 for the upstream slope. The downstream slope would be governed by the strength properties of the embankment material and the foundation rock mass strength. The upstream slope, would be governed by these properties as well the capacity of the embankment material to drain. If the material remained saturated instability might result upon reservoir drawdown.

Filter, drain and transition zones have been incorporated into the preliminary design section. The upstream face of the embankment would be covered with riprap, consisting of cobble and boulder size rock, to protect the upstream shell zone from wave-action erosion. The downstream face should be planted with natural grasses to control surficial erosion.

Preliminary stability analyses were performed by Boyle Engineering on potentially critical embankment sections using Bishop's Simplified Method of Slices (Bishop, 1955) applied to circular failure surfaces and Spencer's Method (Spencer, 1967) applied to noncircular surfaces. The results of the preliminary analyses yielded adequate safety factors for preliminary design, thus confirming that the selected embankment slopes were appropriate.

Spillway Description

A concrete-lined service spillway and an unlined auxiliary spillway would be provided. Each is designed as an uncontrolled Ogee section. The service spillway sill would be at elevation 7485 feet with a length of 35 feet. During the peak outflow of 40,900 cfs, the depth of flow at the service spillway would be 14 feet and the depth at the auxiliary spillway would be 11.5 feet. A deflector bucket at the downstream end of the service spillway apron would provide energy dissipation. The auxiliary spillway would be founded in shale rock on the right abutment.

During infrequent maximum flows considerable erosion would be expected in the lower portion of the auxiliary spillway. Such erosion should be no threat to the security of the dam. Incidences of required erosion damage repair would be expected to be many years apart.

Foundation

Due to the potential for collapse of the loose deposits subsequent to construction of the dam and reservoir impoundment, as well as the potential for liquefaction of loose cohesionless materials when subject to seismic loading, the approach adopted at this level of study has been to plan to excavate the alluvium from beneath the embankment.

In order to prevent uncontrolled seepage through the terrace deposit on the right abutment, a positive cutoff to competent rock will be required. This will require extending the embankment section beyond the auxiliary spillway to intercept the competent rock at the elevation of the top of the core zone. The fractured rock of the left abutment would be removed from under the core. Concrete under the service spillway crest would extend to competent rock below and to the east, and tie into the core zone to the west. Concrete under the auxiliary spillway crest would extend to competent rock below and into the core zones on the east and west. Estimates have been made for a single line grout curtain beneath the core zone of the dam, and limited rim grouting in the abutments, to seal off the major joints and discontinuities in the foundation rock.

Outlet Works

The outlet works would be converted from the conduit used for diversion during construction. It would consist of a conduit placed in a cut-and-cover trench excavated to bedrock along the right abutment. A conduit diameter of 10 feet is projected to serve for construction diversion. It would be completed by inserting the 4-foot diameter outlet pipe and leaving an access way along side to the guard valve. The intake structure would be built and, at the downstream termination, a 42-inch Howell-Bunger valve would be installed to control the estimated discharge of 620 cfs.

Pumped-Storage Hydropower Potential

A potential upper reservoir site on Wolford Mountain, described in Section 5.2.5 of Wolford Mountain Site A' is also applicable to Site C.

6.6 HIGHWAY RELOCATION

Sections of Highway U.S. 40 cross small arms of the proposed reservoir at elevations of 12 to 23 feet below the maximum water level. New highway sections would be constructed on fill over the present roadway to an elevation above the future water level. The length of the highway involved is approximately 5000 feet.

6.7 SUMMARY OF ESTIMATED CONSTRUCTION AND OPERATION COSTS

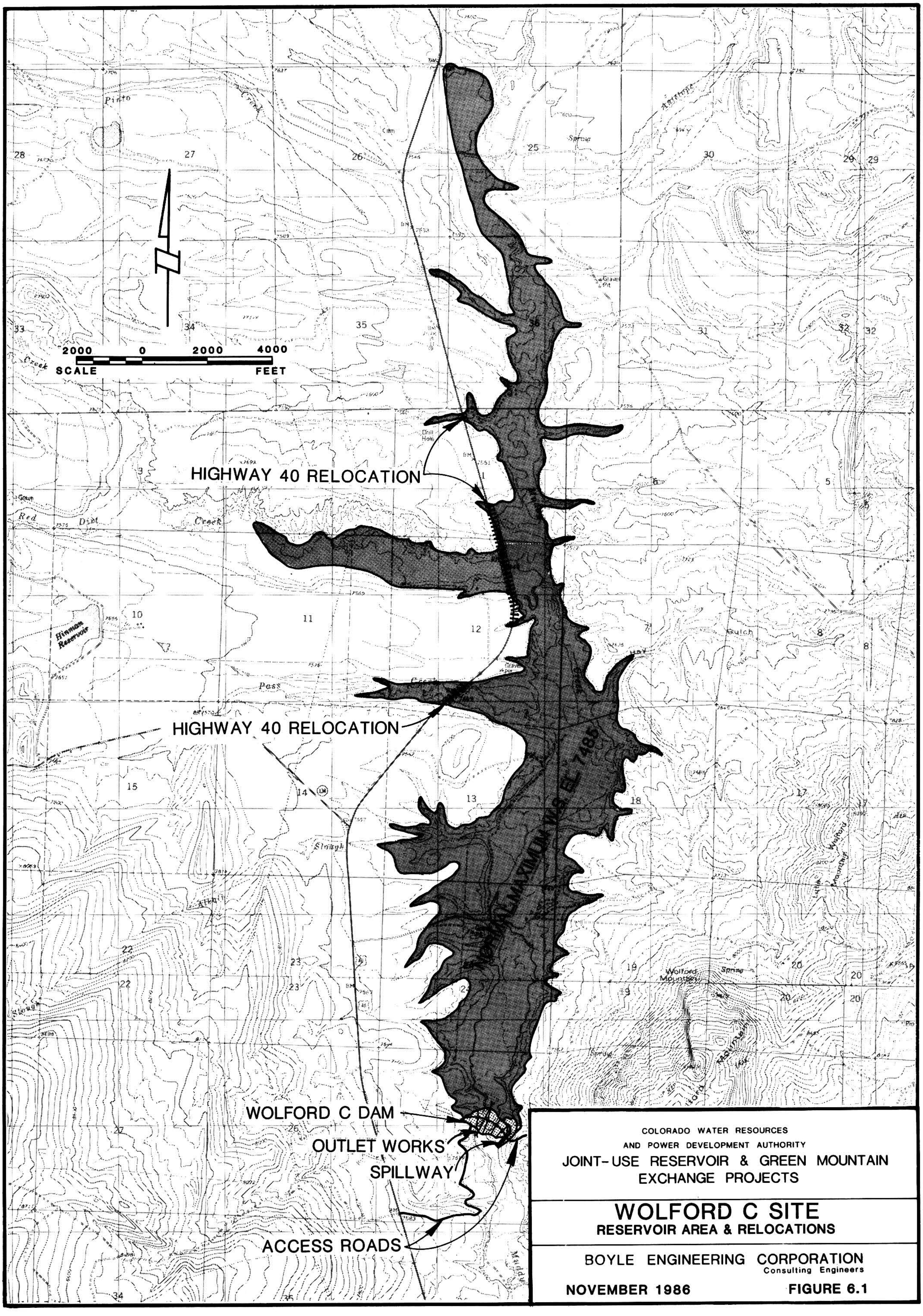
An embankment dam at Wolford Site C, with a dam height of 145 feet and crest length of 1700 feet, would impound a reservoir with 60,000 af capacity. Reconnaissance-level cost estimates have been developed for both operation and construction costs. The detailed cost estimates and the basis of their derivation are described in Appendix B. Estimated construction quantities, prices, and allowances have been combined to obtain a total construction cost. The costs are indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing have been added to obtain the total investment cost. Estimated construction quantities and estimated costs are summarized in Table 6.1. A detailed description of estimated construction quantities and estimated costs can be found in Chapter B.9.

Dam operation and upkeep would be carried out by a damkeeper, an annual technical inspection team and services of specialty contractors. The estimated average annual cost of dam operation and upkeep at the proposed Wolford Mountain Site C Dam and Reservoir is $100,000. Recreation-type services are assumed to be offset by user charges and are not included in this estimate. Details of dam operation annual costs are in Appendix B.

TABLE 6.1

WOLFORD MOUNTAIN SITE C DAM AND RESERVOIR
SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	1,900	acres	$ 1,630,000
River Diversion			250,000
Dam and Spillway			17,300,000
Shell Volume	828,100	cubic yards	
Core Volume	162,650	cubic yards	
Drain Volume	54,000	cubic yards	
Filter Volume	58,800	cubic yards	
Transition Volume	54,000	cubic yards	
Excavation	1,092,700	cubic yards	
Structural Concrete Volume	9,880	cubic yards	
Mass Concrete	12,300	cubic yards	
Outlet Works			2,170,000
Relocations			
State Highway	0.9	miles	1,800,000
Transmission Line	0.6	miles	300,000
Subtotal of Direct Costs			$ 23,450,000
Contingencies (25%)			5,900,000
TOTAL DIRECT COST			$ 29,350,000
Engineering, Legal, Administration (20%)			5,900,000
TOTAL CONSTRUCTION COST			$ 35,250,000
Interest During Construction (2 yrs.)			2,750,000
TOTAL CAPITAL COSTS			$ 38,000,000
Financing and Reserve			5,100,000
TOTAL INVESTMENT COST			$ 43,100,000



COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN
EXCHANGE PROJECTS

WOLFORD C SITE
RESERVOIR AREA & RELOCATIONS

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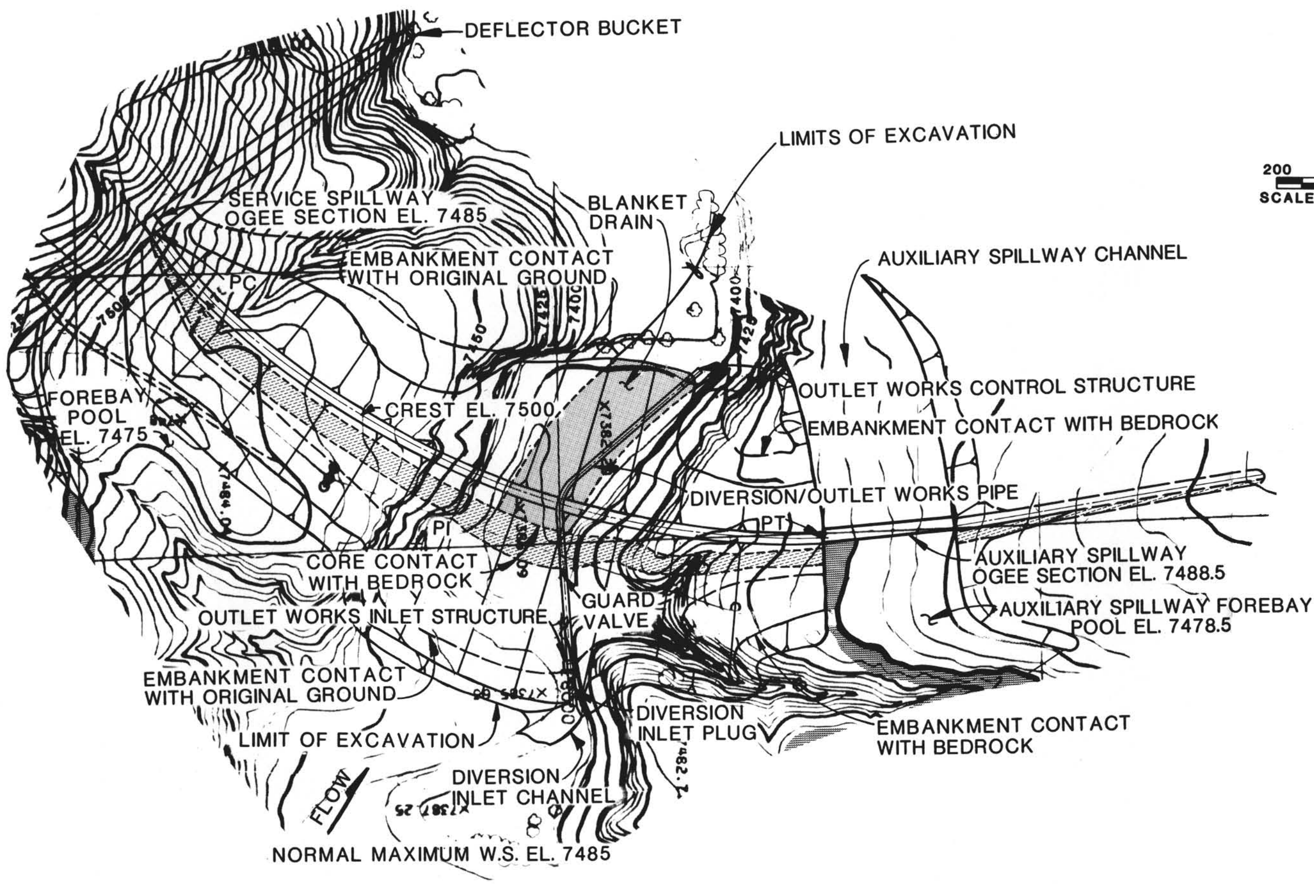
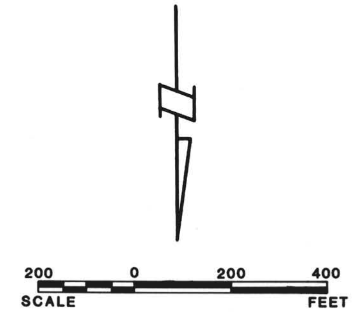
NOVEMBER 1986 FIGURE 6.1



DAM LAYOUT



CREST CURVE DATA

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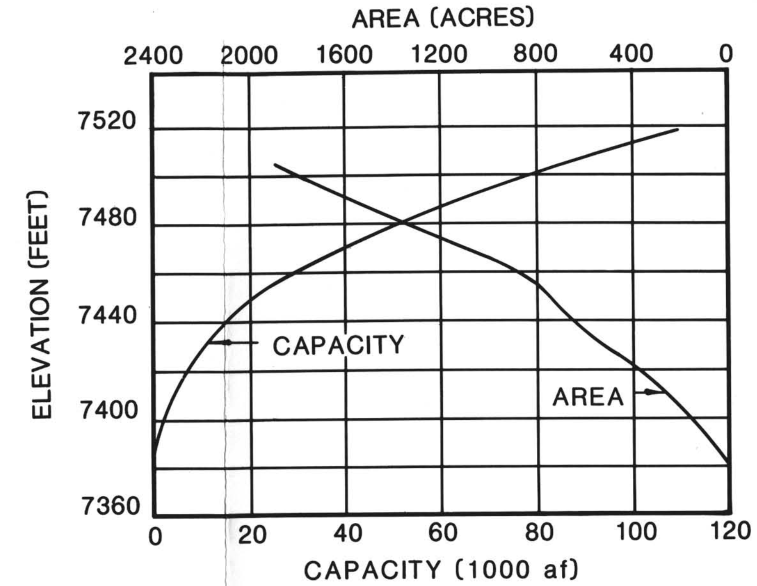
L = 1577.56'
T = 843.48'
R = 1800'
PC = 0+00
PI = 8+43.48
PT = 15+77.56



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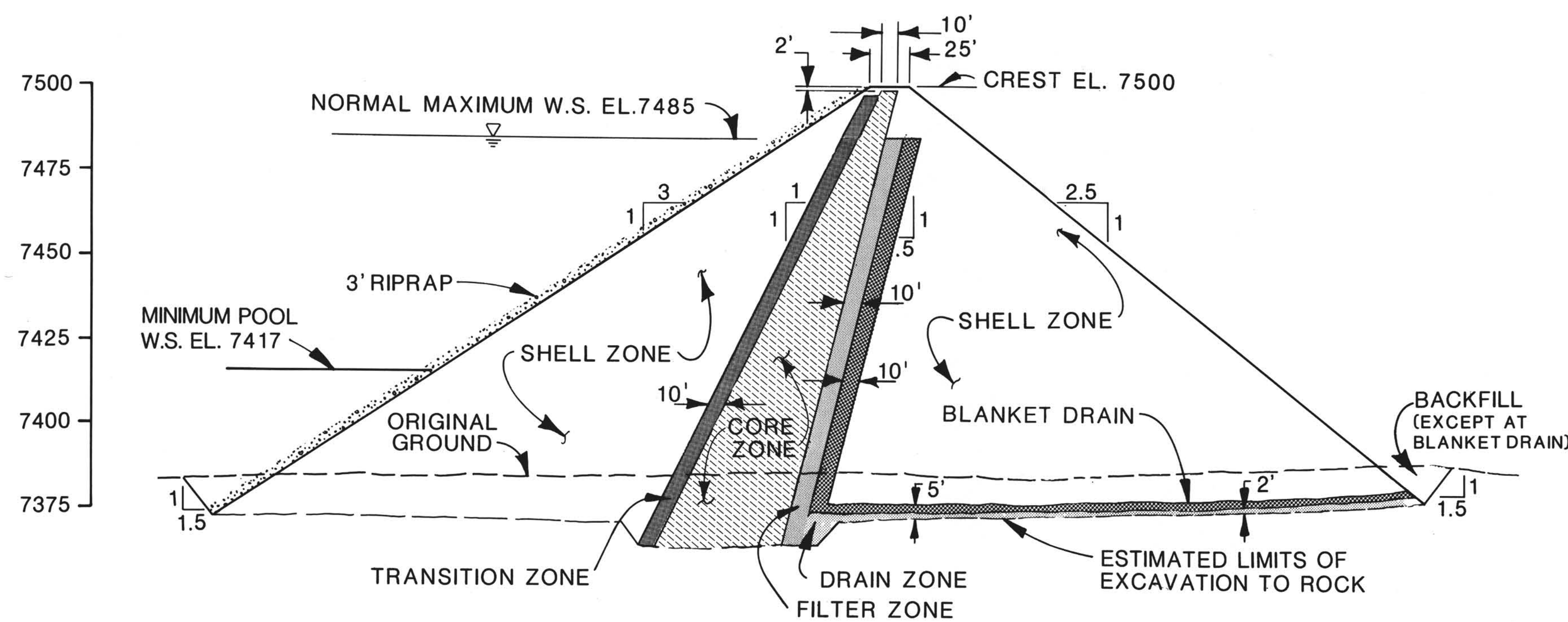

TYPICAL DAM SECTION

COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN
EXCHANGE PROJECTS

WOLFOLD C DAM
LAYOUT AND SECTION

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FEBRUARY 1987

FIGURE 6.2

7.0 RED MOUNTAIN DAM AND RESERVOIR

The Red Mountain Dam Site is located on the Colorado River 1 mile east of Kremmling. Figure 7.1 presents a plan view of the Red Mountain Reservoir Area.

Following is a tabulation of descriptive and dimensional data:

Dam type	RCC
Height of dam	85 feet
Dam crest length	1700 feet
Reservoir volume	140,000 af
Hydropower potential	3.4 GWh/yr
Land required	3300 acres
Railroad relocation	9.0 miles
Highway relocation	6.6 miles

7.1 SITE TOPOGRAPHY

The dam would be located at a narrows, approximately 600 feet wide, formed by the base of Red Mountain on the north side of the Colorado River and a terrace extending from Junction Butte on the south. Upstream, a broad valley 4000 to 5000 feet wide forms the reservoir area.

River-bed elevation at the dam site is 7325 feet. The left abutment terrace terminates at the river in a 55-foot high escarpment. The right abutment rises steeply from the river bed to elevation 7415, then slopes moderately to Red Mountain. The Denver and Rio Grande Western (D&RGW) Railroad runs along the base of the right abutment. The railroad would be relocated to pass north of the dam in an open cut above the proposed dam crest elevation of 7410 feet. The reservoir would extend 5 miles upstream with a large arm extending north of the Colorado River up Troublesome Creek.

7.2 STUDIES AND FIELD INVESTIGATIONS

A dam and reservoir was considered for the Red Mountain Site during the reconnaissance-level USBR Blue River-South Platte Investigation. A brief geological description was prepared, but the site was not incorporated into the final report, (USBR, 1945).

During the course of this Study, field and laboratory geotechnical investigations were conducted at the Red Mountain Site by Chen & Associates. Six core borings and 13 auger borings were drilled to define the subsurface stratigraphy and physical characteristics of the geologic formations of the dam area. Exploratory boring locations, logs, test results and a geologic dam site map are included in Appendix C.

7.3 GEOLOGICAL DESCRIPTION

The Red Mountain Dam Site is located near the western edge of the Middle Park Basin. The regional geology is identical to that of Wolford Mountain Site A' and is described in Section 5.3.

The Red Mountain Dam Site is located in the upper, eastern block of the Williams Range Thrust Fault. The surface trace of the fault is located about 500 to 1000 feet west of the dam axis and dips approximately 65^o to 70^o towards the east. Projected below the dam, the fault would lie at a depth greater than 1000 feet.

The dam site is underlain by Precambrian-aged biotite gneiss, hornblende gneiss and pegmatite dikes. The foundation rock is highly fractured and sheared as a result of faulting on the Williams Range Thrust Fault. Many of the fractures are healed; iron oxides, clays and other minerals have cemented the fractures which give the rock mass some coherence.

7.4 DESIGN CONSIDERATIONS

Preliminary evaluation of the large spillway capacity requirement and the strength and relative impermeability of the foundation suggested that this site is most suited for a concrete dam. Design considerations addressed in this section include foundation conditions, earthquake design, spillway requirements and construction materials.

Surface Material

The material encountered in the borings on the left abutment terrace include a layer of terrace sand and gravel 5 to 19 feet thick overlying fractured gneiss bedrock. A layer of colluvium 5 feet thick was encountered at the toe of the left abutment.

In the valley bottom, alluvial sandy silt and sand and gravel were encountered to a depth of 47 feet. A 6-foot thick layer of sandy silt under the alluvium and overlying the bedrock was encountered in a hole on the south margin of the present river channel. Although encountered in only one hole, the deep silt should be expected to reoccur intermittently in the valley alluvium. This material would not provide the necessary support for a dam.

The limited number of borings may not have encountered the greatest depth to bedrock. It is possible that a deeper buried alluvial channel may be present at the dam site.

Rock Strength

The gneiss rock which underlies the overburden at the site, is moderately to highly fractured. Results of the unconfined compressive strength tests on intact cores indicate uniaxial strengths ranging from 1500 psi to 5000 psi. These relatively low strengths resulted from the many healed fractures in the core which failed during the test. Based on these strengths and core properties, a rock mass strength envelope was estimated with a friction angle of 35^{o} and a cohesion intercept of 20 psi. These values would require an elongated dam base compared with designs for higher rock mass strengths.

Foundation Permeability

In-place packer permeability tests were conducted in the gneiss bedrock in four holes along the dam axis. Results of these tests indicate variable permeabilities. On the right abutment, the permeability was low and ranged from 1 to 14 feet per year. In the valley bottom the permeability increased with depth. Above a depth of 100 feet the permeability averaged 25 feet per year whereas below 100 feet the permeability averaged 85 feet per year.

Seismic Considerations

To develop dam stability during an earthquake, design criteria must consider the proximity of active faults as well as probable magnitude of the seismic event. Preliminary evaluation of the earthquake potential in Colorado classified several faults in the vicinity of the Kremmling area dam sites as potentially active faults (Kirkham and Rogers, 1981). These include the Gore Fault, the fault along the west side of the Williams Fork Valley, the Frontal Fault and the Antelope Pass Fault. A description of each fault is presented in Appendix B. The Gore and Frontal Faults are approximately 8 miles from the Red Mountain Site at their closest points. The

Williams Fork Valley Fault is approximately 4 miles and the Antelope Pass Fault is approximately 1 mile from the dam site. Without detailed investigations of the faults relevant to dam design, a conservative design approach should be used. For this Study, it was assumed that the nearby faults are potentially active.

For preliminary design, it is appropriate to use slopes on the downstream face of the dam that are less steep than might be recommended after thorough analysis. This forms a thicker structure, which results in lower tensile stresses and extends the rock contact at the base of the dam to improve safety against sliding. For final design, additional geologic seismic investigations would be conducted and a probabilistic analysis carried out to refine the design earthquake magnitude and distance from the site. The design dam section can then be analyzed for dynamic loading using the selected accelerogram time histories for both horizontal and vertical earthquake-induced forces.

<u>Spillway Requirement</u>

The spillway size for the Red Mountain Dam is based on the PMF which has a peak inflow of 240,000 cfs. The inflow hydrology was developed using Hydrometeorology Report No. 49 (NOAA, 1984) as a basis for the Probable Maximum Precipitation and the Corps of Engineers Hydrologic Engineering Center's HEC-1 mathematical modeling of the watershed for the inflow hydrograph (COE, 1981). Routing the inflow hydrograph through the reservoir results in a reduced outflow hydrograph. The spillway would be required to pass the peak outflow of 238,000 cfs with a depth of flow 14 feet above the spillway sill.

<u>Construction Materials</u>

Concrete aggregate material may be obtained from the alluvium within the valley bottom or the terrace deposits on the valley slopes. Screening and processing, including washing, would be necessary to produce suitable material. In addition, a rock quarry could be located below the terrace materials on the left abutment. The suitability of the material can be determined by the results of abrasion, durability and potential reactivity testing. This type of testing was not conducted for this study, but it was assumed for the cost estimate that the material would be suitable. Concrete aggregate and concrete are also available from a supplier in Kremmling who can produce and deliver the materials to the job site.

7.5 PROPOSED DAM AND APPURTENANCES

Both an earth embankment dam and a roller compacted concrete (RCC) gravity dam were considered for the Red Mountain Site. The controlling design parameter was the large spillway capacity required to pass the PMF.

The RCC gravity dam design allows most of the crest length of the dam to be used for spillway. An alternative embankment dam would require an extensive excavation in one of the abutments to provide a concrete lined spillway. The total dam and spillway length would be much greater. The RCC gravity dam was selected because it meets the hydrological requirements and fewer changes in physical features of the Red Mountain Site would be involved.

Dam and Overflow Description

The interior of the dam would be constructed of RCC with conventional concrete on both faces and the overflow section. A section is shown in Figure 7.2. The downstream face, with a slope of 0.7:1, would meet internal stress requirements at all levels of the dam. The downstream facing of the Ogee overflow would be stepped for ease of construction and to provide energy dissipation during spillway operation. Extension of the concrete apron downstream from the spillway increases sliding stability to 4 times the static loading.

The lengths of the two overflow sections were selected based on the two predominant rock depths at the site, and the PMF requirement. The training wall between the two sections would be incorporated into the diversion scheme.

Foundation Excavation

Surface deposits would be removed from the dam foundation and spillway aprons. Approximately 5 feet of bedrock beneath the dam would be excavated to allow the dam to be keyed into the foundation. The bedrock would be excavated beneath the aprons to allow placement on firm material. Dowels placed into the bedrock would secure the apron from lifting.

During the foundation excavation and construction, dewatering of the valley bottom would be required. Based on the preliminary drilling, cutoff trenches to bedrock would be necessary upstream and downstream of the dam to block seepage flows.

Outlet Works

The outlet works would be located in the right abutment. Design discharge would be 1765 cfs to meet the Colorado State Engineer's requirement of drawing the reservoir down 5 feet in 5 days.

Hydropower Plant

The power plant would be located downstream of the right abutment. The 4.5 foot diameter penstock would branch from the outlet works. Maximum power generation would be 2600 kilowatts. Two Francis turbines operating on flows by-passed or released to meet downstream requirements and flows which would be spilled, would produce an average of 3.4 million kilowatt-hours (kWh) per year. Evaluation of hydropower at this dam is discussed in Section 12.2.

7.6 RELOCATIONS

Both railroad and highway relocations would be required. Approximately 15 dwellings and ranch structure groups would be purchased when acquiring land for the reservoir.

Railroad Relocation

The D&RGW Railroad would be relocated to pass along the north side of the Red Mountain Dam and Reservoir. A new track elevation of 7420 feet would be well above the spillway elevation. At the dam site, the new track would be about 80 feet higher than the existing track. To reach this elevation, realignment would begin southwest of Kremmling at a grade of 0.8 percent, parallel with and south of the existing track. This grade would cross over Colorado Highway 9, leaving the existing rail line and bridge to serve local industry. Besides the highway over-crossing, two single-rail bridges are proposed: a 1000-foot bridge crossing Troublesome Creek and a 500-foot bridge crossing Sulphur Gulch. Minimum radius curves for this 70 mph design stretch would be 4000 feet and the maximum grade, 1 percent.

Highway Relocation

A 6.6-mile portion of Highway U.S. 40 would be relocated to accommodate the reservoir and railroad relocation. From the eastern town limit of Kremmling the highway would traverse the saddle between Red Mountain and Horse Gulch, then continue to the east, parallel with the proposed railroad realignment. It would rejoin the original Highway U.S. 40 approximately 4 miles west of Parshall. Three bridges would be required: a 1000-foot bridge crossing Troublesome Creek, a 500-foot bridge over Sulphur Gulch and a 150-foot railroad overpass.

Approximately 7 miles of the county roadway on the south side of the proposed reservoir would be relocated. A 200-foot bridge crossing the Colorado River and a causeway at Barger Gulch would be required.

7.7 SUMMARY OF ESTIMATED CONSTRUCTION AND OPERATION COSTS

An RCC dam at the Red Mountain Site, with a crest length of 1700 feet and dam height of 85 feet, would create a reservoir with 140,000 af of capacity. Reconnaissance-level cost estimates have been developed for both construction and operation costs. The detailed cost estimates and the basis of their derivation are described in Appendix B. Estimated construction quantities, prices, and allowances have been combined to obtain a total construction cost. Costs are indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing have been added to obtain the total investment cost. Estimated construction quantities and estimated costs are summarized in Table 7.1.

Dam operation and upkeep would be carried out by two damkeepers, an annual technical inspection team and services of specialty contractors. The estimated annual cost of dam operation and upkeep at the proposed Red Mountain Dam and Reservoir is $150,000. Recreation-type services, fees and income are not included. Details of annual costs are in Appendix B.

The cost of adding hydropower generating facilities is not included in the dam and reservoir cost estimate, but is listed separately in Table 7.2. Operation of the hydropower has been estimated to require the part-time services of one employee, such as one of the damkeepers. Specialty services would be employed for equipment repair and maintenance not performed by operators. Estimated annual costs are summarized in Table 7.3.

TABLE 7.1

RED MOUNTAIN DAM AND RESERVOIR

SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	3,300	acres	$ 4,880,000
River Diversion			2,500,000
Dam and Spillway			30,350,000
RCC Volume	172,000	cubic yards	
Structural Concrete Volume	36,000	cubic yards	
Mass Concrete	86,000	cubic yards	
Excavation	526,000	cubic yards	
Outlet Works			870,000
Relocations			
Railroad	9.0	miles	26,800,00
State Highway	6.6	miles	9,100,000
Other Roads	9.3	miles	8,400,000
Subtotal of Direct Costs			$ 82,900,000
Contingencies (25%)			20,700,000
TOTAL DIRECT COST			$ 103,600,000
Engineering, Legal, Administration (20%)			20,700,000
TOTAL CONSTRUCTION COST			$ 124,300,000
Interest During Construction (3 yrs.)			15,000,000
TOTAL CAPITAL COSTS			$ 139,300,000
Financing and Reserve			18,700,000
TOTAL INVESTMENT COST			$ 158,000,000

TABLE 7.2

HYDROPOWER ADDITION AT RED MOUNTAIN DAM

SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

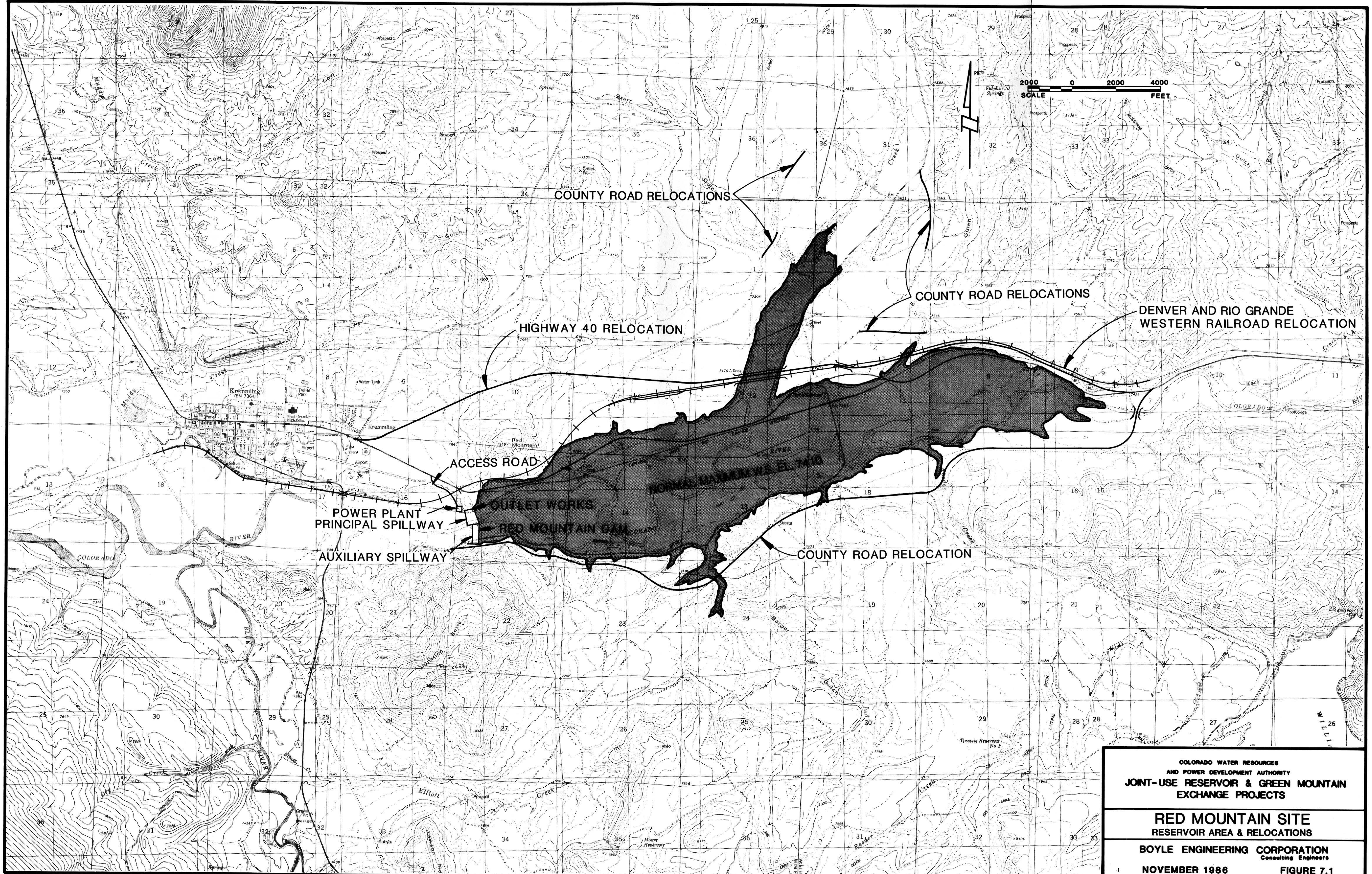
ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Power Plant			$ 2,430,000
Turbine Generator	2	each	
Penstock	260	feet	
Rock Excavation	5,500	cubic yards	
Concrete	270	cubic yards	
Transmission and Connection	0.4	miles	160,000
Subtotal of Direct Costs			$ 2,590,000
Contingencies (25%)			650,000
TOTAL DIRECT COST			$ 3,240,000
Engineering, Legal, Administration (20%)			650,000
TOTAL CONSTRUCTION COST			$ 3,890,000
Interest During Construction (3 yrs.)			310,000
TOTAL CAPITAL COSTS			$ 4,200,000
Financing and Reserve			600,000
TOTAL INVESTMENT COST			$ 4,800,000

TABLE 7.3

RED MOUNTAIN DAM AND RESERVOIR

SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Total Dam Operation and Upkeep	$150,000
Hydropower Plant Operation	
Operators, Supervision	$ 44,000
Equipment and Supplies	4,000
Repair and Replacement	34,000
Total Hydropower Plant Operation	$ 82,000



COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

RED MOUNTAIN SITE

RESERVOIR AREA & RELOCATIONS

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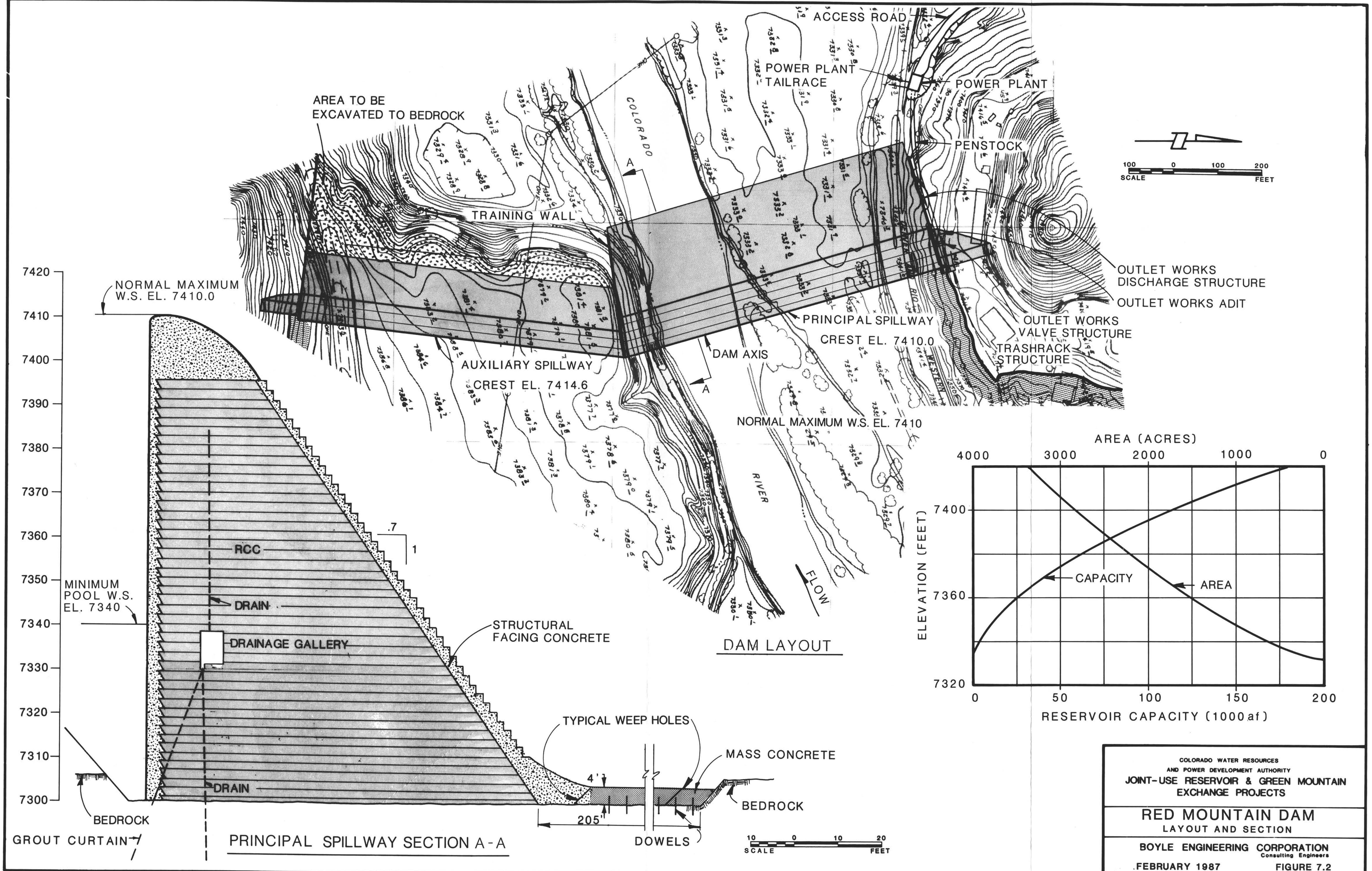
NOVEMBER 1986 FIGURE 7.1



COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

RED MOUNTAIN DAM
LAYOUT AND SECTION

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FIGURE 7.2

8.0 AZURE DAM AND RESERVOIR

The Azure Dam Site is located on the Colorado River at the upper end of the Lower Gore Canyon, about 10 miles downstream from Kremmling. Figure 8.1 presents a plan view of the Azure Dam and Reservoir area.

Following is a tabulation of descriptive and dimensional data:

Dam type	Concrete arch
Height of dam	225 feet
Dam crest length	500 feet
Reservoir volume	85,000 af
Land required	1150 acres
Railroad relocation	2.8 miles

8.1 SITE TOPOGRAPHY

The Lower Gore Canyon is a narrow, deep gorge characterized by rugged, precipitous topography. Elevation is approximately 6910 feet at the river and approximately 7400 feet at the top of the canyon. The Moffat route of the D&RGW railroad is cut into the north side of the canyon and would be relocated into a tunnel and open cut to avoid the dam and reservoir area. The maximum reservoir level was limited to remain below the railroad grade in Upper Gore Canyon, three miles upstream from the dam site.

8.2 STUDIES AND FIELD INVESTIGATIONS

The most recent study of the Azure Dam and Reservoir was prepared for the Municipal Subdistrict of the Northern Colorado Water Conservancy District by International Engineering Corporation, Inc. (IECO), now Morrison-Knudsen Engineers Inc., dated September, 1983. The feasibility report considered two dam heights and recommended the lower dam, sized to avoid railroad relocation. The project included a pumped storage hydroelectric facility involving a reservoir in Trough Valley, 1200 feet above and 1.8 miles southeast of Azure Reservoir. Study of the project was continued with the "Environmental Report for the Azure Hydroelectric Project", by Tom Pitts and Associates (1984).

Studies of nearby projects which also contain information pertinent to Azure include: "Seismotectonic Hazard Evaluation, Rock Creek Project Grand and Routt Counties, Colorado", (Michael W. West and Associates, Inc., 1986) and "Eagle-Piney/Eagle-Colorado Water Study", (Parsons, Brinckerhoff, Quade & Douglas, Inc. and Forrest & Cotton, Inc., 1974).

Fourteen core borings were drilled at the present Azure Dam Site during the investigations by IECO (1983). Construction materials were investigated with 37 auger borings, 13 tricone borings and 23 test pits. Exploratory boring locations, logs, geologic maps and sections and test results are reported in Volumes 2, 3 and 4 which form Appendices to the "Azure Project Definite Project Report" (IECO, 1983).

During the course of this Study, published geological information was reviewed and correlated with field observation during two site visits. This information is incorporated into this report.

8.3 GEOLOGICAL DESCRIPTION

The project site is located on the Colorado River at the western edge of the Gore Range. The geology of the area is influenced principally by the formation of the Gore Range. This range is the result of the uplifting of the basement rock above the level of the younger sedimentary rocks which border it to the east and west. The central part of the Gore Range consists of Precambrian crystalline rocks which have been uplifted and faulted repeatedly during geologic time and subsequently eroded leaving high peaks and steep-walled valleys. Flanking the range are sedimentary rocks of sandstone, shale, and limestone which have been folded, faulted and are locally intruded or covered by igneous rocks (Parsons, et al., 1974).

The Azure Dam Site, in the upstream portion of Lower Gore Canyon, is situated in uplifted Precambrian hornblend-biotite gneiss. The reservoir area, between Lower Gore Canyon and Upper Gore Canyon is a down-faulted basin of sedimentary rock which forms a relatively flat and broad valley. The valley has gentle to moderate slopes of recent alluvial deposits and older gravel terraces bordering the river.

The railroad relocation route, north of Lower Gore Canyon, would tunnel through the gneiss from the south-western portal then pass through a series of sedimentary formations to the north-eastern portal, along side the reservoir area.

8.4 DESIGN CONSIDERATIONS

The foundation investigation, conducted by IECO (1983), indicates that the hard, crystalline rock which form the abutments of the narrow gorge at the Azure Site would provide the lateral bearing required for a thin-arch concrete dam. The strong rock foundation and availability of rock and aggregates also suggest suitability for a concrete gravity or concrete faced rock fill dam. Preliminary evaluation selected the thin-arch design as requiring less concrete than a gravity dam alternative and permitting a spillway over the crest as compared with a spillway tunnel which would be required with a rock fill alternative.

The following statements on foundation conditions and other geotechnical observations are all from the IECO (1983) report.

Overburden

The main river channel rests in an accumulation of talus and alluvium. On the southeast bank, bedrock is overlain by 25 feet of talus and 18 feet of alluvium. The thickness of the talus and alluvium under the center of the channel is estimated to be 20 to 30 feet. A layer of very coarse talus and colluvium also covers most of the northwest side with depths of 13 to 23 feet.

Rock Strength

Examination of the rock cores and outcrops exposed on both sides of the river indicates that the gneiss is hard and only slightly weathered at the surface. Distinct systems of joints and faults were discovered in surface exposures on each side of the river and found in drill holes beneath the river channel. The frequency of fracturing and shearing is noticeably higher on the northwest side and beneath the river channel than on the southeast side of the river. The orientation and in-filling materials of these shear zones beneath the channel on the northwest side do not appear to be hazardous. They should present no stability problems in the foundation after they have been excavated, backfill with dental concrete and grouted.

Ten core samples from the Azure Site were selected for laboratory tests. Average compressive strength was 21,775 psi. Mean specific gravity was 2.76. The rock properties determined in laboratory tests were modified to reflect site conditions. The effective deformation modulus for the abutments, as modified in the IECO (1983) studies, is given as 3×10^6 psi and the modified Poisson's ratio as 0.20.

Foundation Permeability

The borings were water pressure tested to determine the permeability of the bedrock. These tests show that permeability decreases with depth, and the overall permeability is 10^{-3} cm/sec or lower. These results indicate no significant leakage will occur after normal foundation treatment.

Design Earthquake

To develop dam stability during an earthquake, design criteria must consider the proximity of active faults as well as probable magnitude of the seismic event. The Gore Range is bounded along its west flank by the Gore Fault and along its east flank by the Frontal Fault. The study, "Earthquake Potential in Colorado", (Kirkham and Rogers, 1981) considers both the Gore and Frontal Faults to be potentially active earthquake faults.

A seismicity report prepared by IECO (1983) for the Azure area recommended ground motion for dynamic analyses of the lower dam at the Azure Site. It was based on the predominant period and duration of shaking selected from empirically derived curves. The following parameters characterize the Maximum Credible Earthquakes:

	Local	Regional
Magnitude:	6.0	7.0
Distance:	10 km	40 km
Peak horizontal acceleration:	0.23 g	0.12 g
Predominant period:	0.25 sec	0.32 sec
Duration:	8 sec	21 sec

IECO (1983) also calculated a probabilistic ground motion such as might be used for pseudo-static analysis for the design of appurtenant structures of the project. This is a means of determining a horizontal force which, when applied to simple structures, approximates seismic resistant designs. A horizontal acceleration of 0.12 g (0.12 multiplied by the force of gravity) indicated a return period of about 25 years and 0.23 g indicated a return period of about 700 years.

Final design would involve selection of accelerogram time histories for both horizontal and vertical ground motion which would be applied for dynamic analysis of earthquake-induced forces. Preliminary dimensions used in this study for the cost estimate of the higher dam at the Azure Site were based on a composite of previously constructed dams.

Spillway Requirement

The PMF was estimated by IECO (1983) as 262,000 cfs. Attenuation of the PMF by routing through the reservoir resulted in peak outflow for the spillway design of 256,000 cfs.

Construction Materials

Laboratory tests indicated good quality aggregate within the reservoir area. The deposits contain rounded cobbles, gravels, and occasional boulder-size fragments of gneiss and minor quantities of sandstone and schist overlain by silty clay and clayey sand. Processing would be necessary to reduce the amount of fines in the finished aggregate and to remove oversize materials. The quantity of in-place sandy gravel was estimated at 505,000 cubic yards (IECO, 1983).

8.5 PROPOSED DAM AND SPILLWAY

The shape of the dam, its thicknesses, and orientation in the site are influenced by predicted loadings, foundation conditions, canyon shape and the canyon width at the crest elevation. The various factors considered for preliminary design of the thin-arch concrete dam were the normal reservoir water surface of elevation 7096, the usual operating temperatures of the concrete, the ice loading, and the Maximum Credible Earthquake.

Dam Section

The horizontal curvature of the dam was selected to transfer as much as possible of the applied loading laterally to the abutments. Site and practical considerations never present ideal conditions for lateral transfer of all the load, which results in transfer of a portion of the load vertically to the foundation. For the most economical use of concrete for vertical load transfer, curvature was provided in that direction as well.

Spillway Description

The spillway was designed as an uncontrolled Ogee section. The sill elevation would be 7096 feet with a length of 440 feet. During the peak outflow of 256,000 cfs the depth of flow would be 28 feet. Training walls would be provided to protect the abutments on each end of the spillway section. A stilling pool at the base of the dam would provide energy dissipation. It would be maintained at elevation 6920 feet by means of a mass concrete dam which could also serve as a downstream cofferdam during construction.

Foundation

Removal of the overburden which ranges in depth from zero to about 30 feet, and some shaping of slopes would be required. Average stripping depth into rock is not expected to exceed 5 feet. Some dental concrete work and rock bolting will be required. Such zones are not expected to exceed five percent of the foundation area (IECO, 1983).

The permeability of the rock tested in eight core holes along the proposed dam axis was low and decreased with depth. This indicates that leakage would be expected to be controlled with normal curtain grouting. A single grout line drilled to a depth equal to the proposed head of water in the reservoir would be placed on 10 foot centers along the upstream edge of the foundation. A consolidation grout curtain would be placed in three lines on 10-foot centers along the dam axis. Holes would be approximately 30 feet deep.

Natural Hazards

Rock falls should be anticipated from the nearly vertical slopes of the canyon walls. The rock is closely fractured creating unstable wedges which are subject to gravity and probably some frost wedging. Greater than normal construction accident risk may exist. Access roads and other surface facilities may require special protection and additional maintenance because of rock falls.

8.6 APPURTENANCES TO THE DAM

Appurtenances include outlet works and power plant. Consideration has been given to pumped storage for hydropower. The northern side of the canyon was selected for the diversion tunnel and power plant for relative ease of access along the railroad grade after relocation of the rail line. Extensive rock excavation up the canyon walls would be involved to construct an access road to the alternative south side location.

Outlet Works

The outlet works would be constructed within the tunnel required for river diversion during construction. Design discharge for the outlet works of 1370 cfs was estimated to satisfy the Colorado State Engineer's requirement of reservoir drawdown of 5 feet in 5 days. A bifurication structure could route outlet flow to the power plant.

Hydropower Plant

The proposed location of the hydropower plant is within a cavern cut into the rock downstream of the right abutment. The cavern provides protection from flooding and from rock falls and affords a shorter penstock than an alternative location 2000 feet downstream where the wider canyon might permit an exposed structure. Access to the power plant would be by elevator from what is now the railroad bed. Two Francis turbines would produce a total of 13,700 kilowatts, and generate an average of 36.2 million kWh per year. Evaluation of hydropower at this dam is discussed in Section 12.2.

Pumped-Storage Hydropower Potential

An upper reservoir site, located in Trough Valley near the divide between the Colorado and Blue Rivers, was investigated for pumped storage in a previous study (IECO, 1983). The operating scheme analyzed by IECO involved a 400-megawatt plant with a generating flow of 4400 cfs at an average head of 1150 ft. Off-peak pumping energy was 1250 gigawatt-hour (GWh) per year and weekday generation of 830 GWh per year. About 3000 af of water would be circulated. Reserving this quantity in Azure Reservoir for pumped-storage use would accordingly reduce the reservoir yield as a Joint-Use or Replacement Reservoir.

Access Road

The permanent access road to the dam and power plant would be constructed on fill over the present railroad bed. It would cross over the west portal of the railroad relocation tunnel and continue to join the existing county road that crosses the Colorado River at Radium. A new bridge at the Colorado River and at Sheepshead Creek would be constructed. Preliminary design is based on maximum grades of 6 percent and minimum curves of 900 feet radius.

8.7 RAILROAD RELOCATION

The Moffat route of the D&RGW Railroad is cut into the north side of the canyon 140 feet above the river and 85 feet below the proposed dam crest. It would be relocated into a tunnel and open cut to bypass the dam and reservoir area. The maximum reservoir level was selected to remain below the railroad grade in Upper Gore Canyon, 3 miles upstream from the dam site. The proposed realignment is shown on Figure 8.1 and relocation details are in Appendix B. The maximum grade would be 1 percent. The relocation requires approximately 11,600 feet of tunnel, a 5400 foot siding replacement and parallel 5400 feet of regraded main line track.

Power output of diesel locomotives is sharply reduced by oxygen diminution in high altitude tunnels. While the proposed tunnel at Azure may seem to be oriented to receive natural ventilation, in fact no data has been gathered on site air movement. For this contingency an allowance approximating the cost of a fan and adit ventilation system has been included in the cost estimate.

8.8 SUMMARY OF ESTIMATED CONSTRUCTION AND OPERATION COSTS

A concrete arch dam at Azure, with a dam height of 225 feet and crest length of 500 feet, would impound a reservoir with 85,000 af capacity. Reconnaissance-level cost estimates have been developed for both construction and operation costs. The detailed cost estimates and the basis of their derivation are described in Appendix B. Estimated construction quantities, prices, and allowances have been combined to obtain a total construction cost. Costs were indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing have been added to obtain the total investment cost. Estimated construction quantities and estimated costs are summarized in Table 8.1 for Azure Dam and Reservoir and in Table 8.2 for hydropower addition to Azure Dam and Reservoir.

TABLE 8.1

AZURE DAM AND RESERVOIR

SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	1,100	acres	$ 790,000
River Diversion			3,800,000
Dam and Spillway			14,080,000
Excavation	42,625	cubic yards	
Mass Concrete	57,000	cubic yards	
Structural Concrete	1,930	cubic yards	
Outlet Works			1,560,000
Access Road	0.8	miles	1,590,000
Railroad Relocation			43,280,000
Tunneling	2.2	miles	
Total Railroad	3.2	miles	
Subtotal of Direct Costs			$ 65,100,000
Contingencies (25%)			16,300,000
TOTAL DIRECT COST			$ 81,400,000
Engineering, Legal, Administration (20%)			16,300,000
TOTAL CONSTRUCTION COST			$ 97,700,000
Interest During Construction (5 yrs.)			23,500,000
TOTAL CAPITAL COSTS			$ 121,200,000
Financing and Reserve			16,800,000
TOTAL INVESTMENT COST			$ 138,000,000

TABLE 8.2

HYDROPOWER ADDITION AT AZURE DAM AND RESERVOIR

SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Power Plant			$ 8,190,000
Turbine Generator	2	each	
Penstock	175	feet	
Rock Excavation	8,200	cubic yards	
Concrete	2,000	cubic yards	
Transmission and Connection	7.4	miles	1,390,000
Subtotal of Direct Costs			$ 9,580,000
Contingencies (25%)			2,400,000
TOTAL DIRECT COST			$ 11,980,000
Engineering, Legal, Administration (20%)			2,400,000
TOTAL CONSTRUCTION COST			$ 14,380,000
Interest During Construction (4 yrs.)			2,300,000
TOTAL CAPITAL COSTS			$ 16,680,000
Financing and Reserve			2,320,000
TOTAL INVESTMENT COST			$ 19,000,000

Dam operation and upkeep would be carried out by a damkeeper, an annual technical inspection team and services of specialty contractors. The estimated average annual cost of dam operation and upkeep at the proposed Azure Dam and Reservoir is $80,000. Recreation-type services are assumed to be offset by user charges and are not included in this estimate.

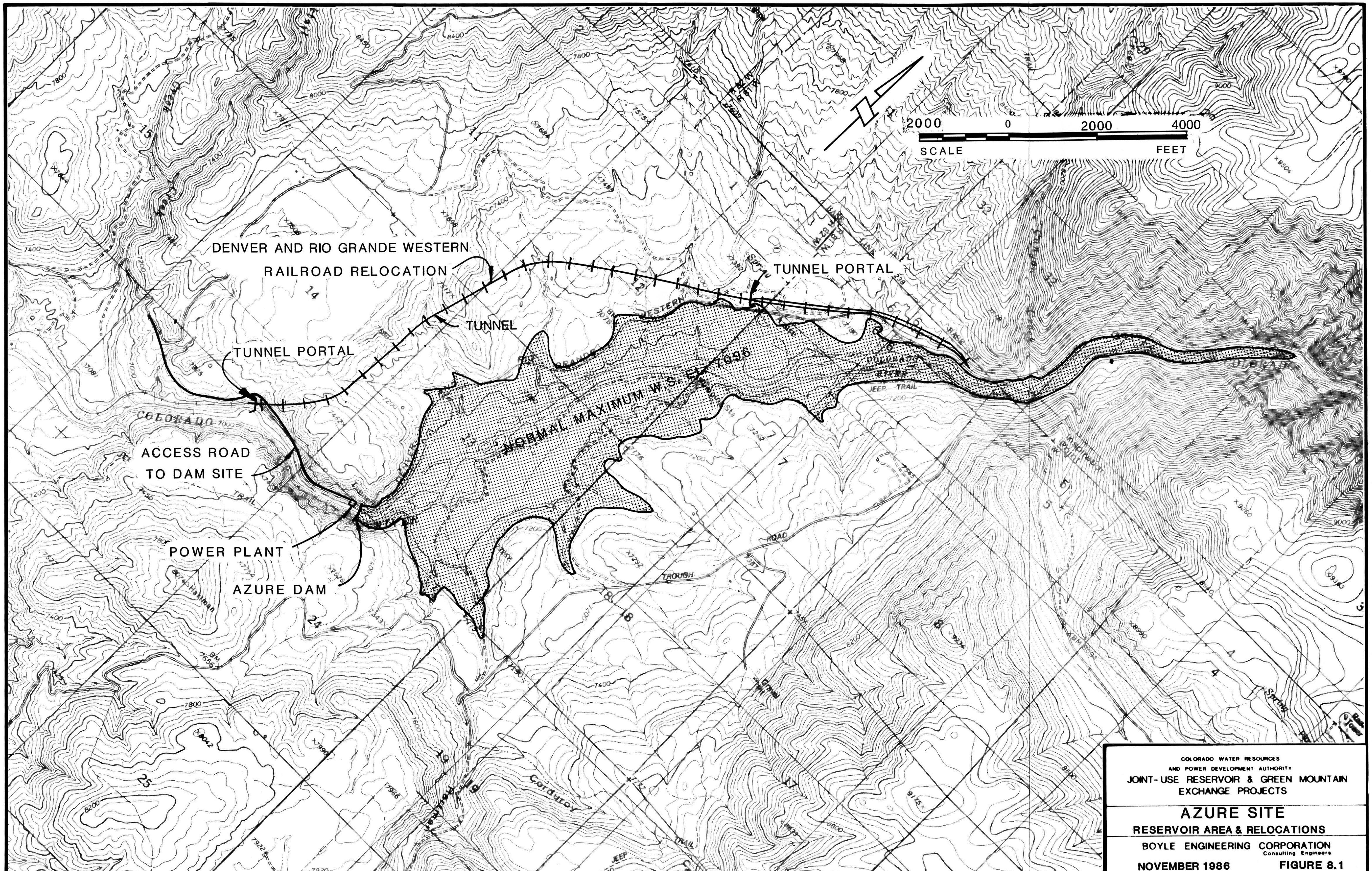
Operation of the hydropower plant has been estimated to require a day-time operator seven days per week. Specialty services would be employed for equipment repair and maintenance not performed by operators. Estimated annual costs are summarized in Table 8.3. Details of annual dam and hydropower operation costs are in Appendix B.

TABLE 8.3

AZURE DAM AND RESERVOIR

SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Total Dam Operation and Upkeep		$ 80,000
Hydropower Plant Operation		
Operators, Supervision	$ 114,000	
Equipment and Supplies	16,000	
Repair and Replacement	70,000	
Total Hydropower Plant Operation		$200,000



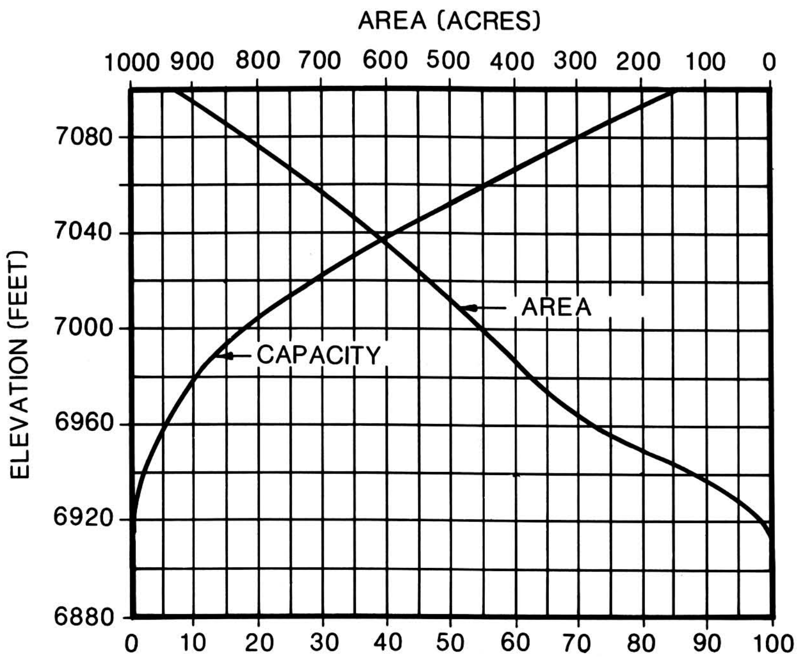
COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

AZURE SITE

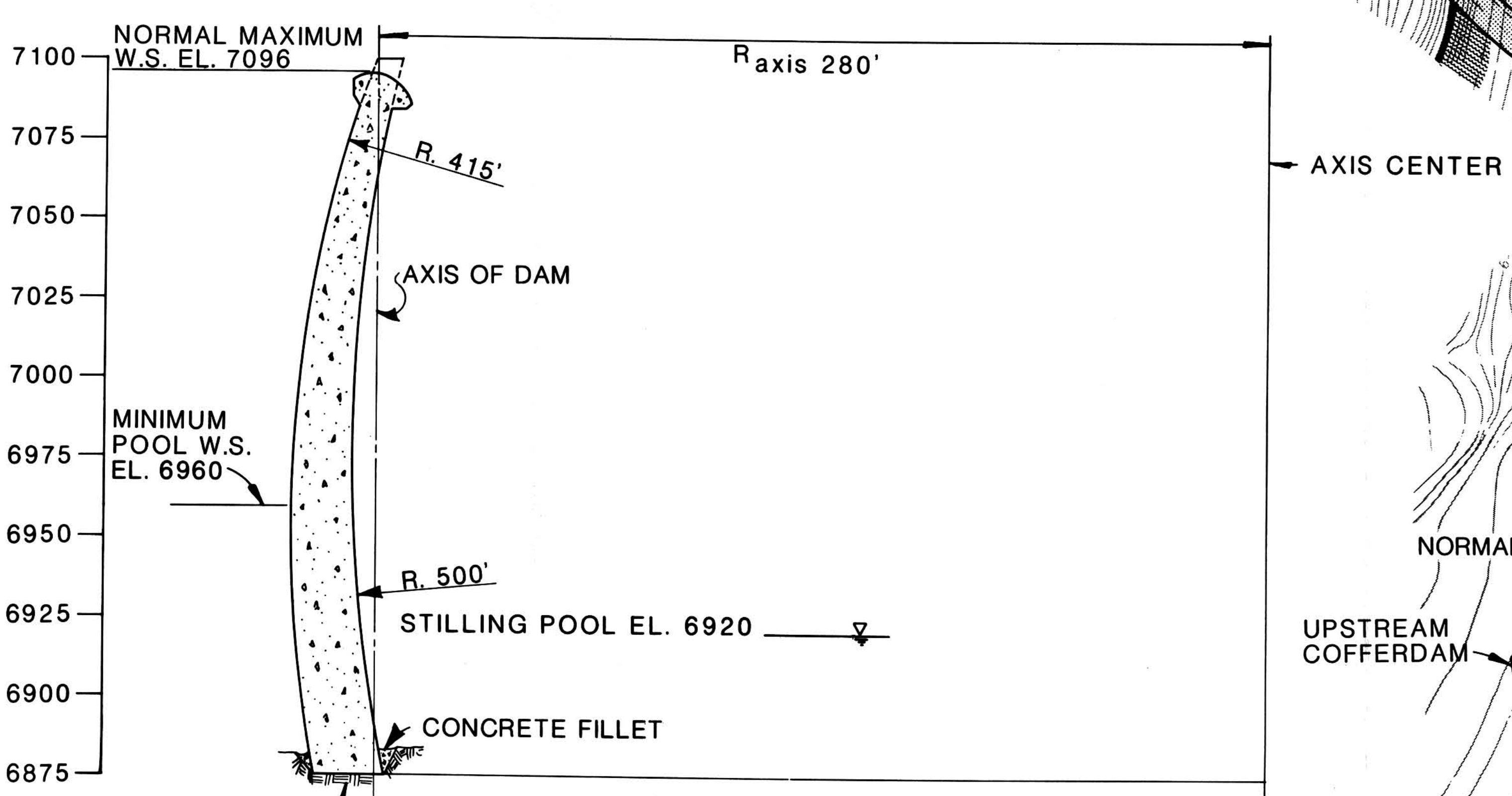
RESERVOIR AREA & RELOCATIONS

BOYLE ENGINEERING CORPORATION
Consulting Engineers

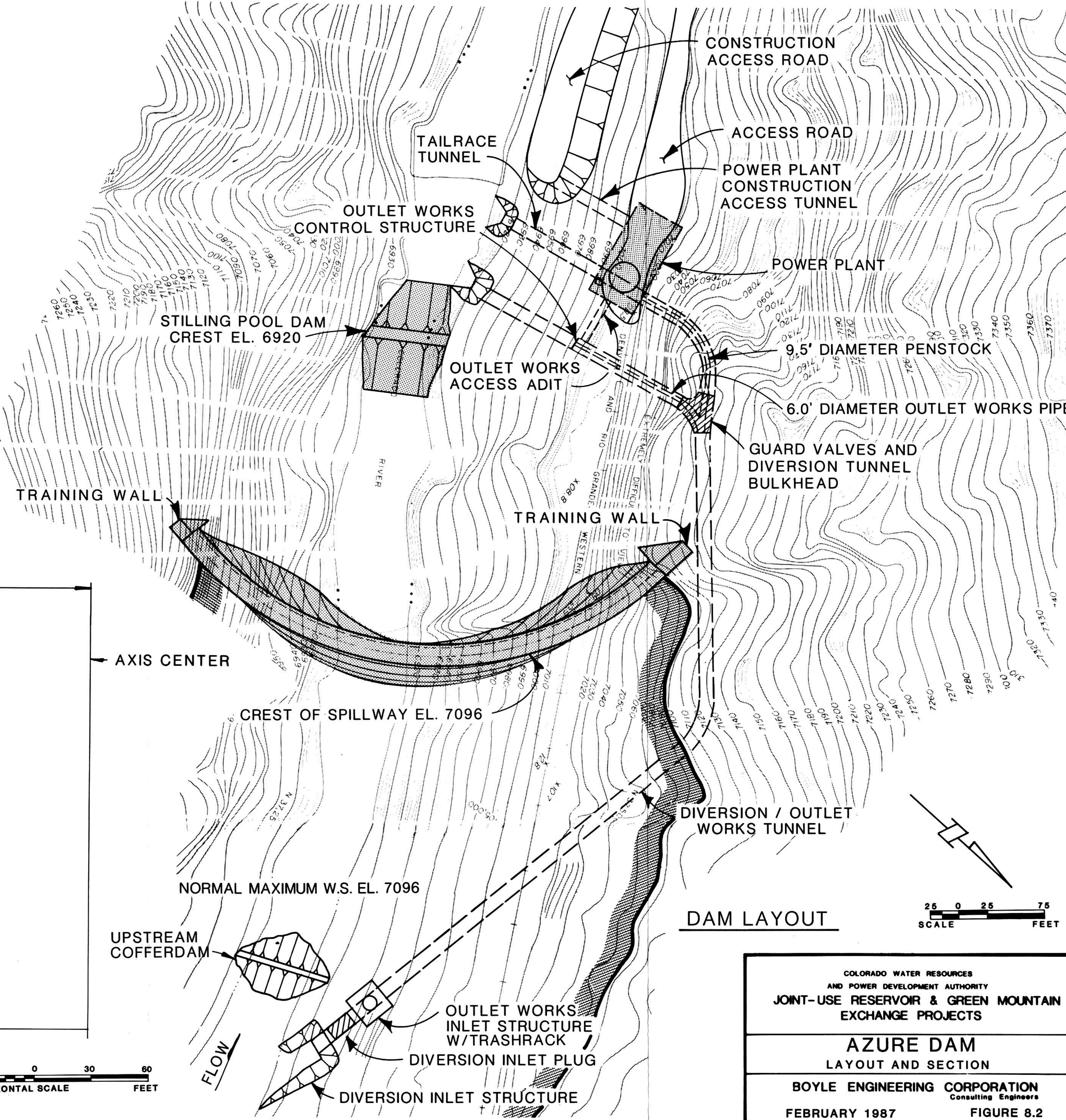
NOVEMBER 1986 **FIGURE 8.1**



RESERVOIR CAPACITY (1000 af)
REDRAFTED FROM IECO, 1983.



SECTION ALONG REFERENCE PLANE



DAM LAYOUT

COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN
EXCHANGE PROJECTS

AZURE DAM
LAYOUT AND SECTION

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FEBRUARY 1987

FIGURE 8.2

9.0 WOLCOTT DAM AND RESERVOIR

The Wolcott Dam Site is 1 mile north of the town of Wolcott, and 0.7 miles north of the Eagle River on Alkali Creek, a minor tributary of the Eagle River. The reservoir would serve as an off-stream storage site. Two reservoir sizes are considered in this study. High Wolcott would be supplied with water by pumping from both the Eagle River and the Colorado River while Low Wolcott would receive water pumped from only the Eagle River. Figure 9.1 presents a plan view of the Wolcott Dam and Reservoir area.

Following is a tabulation of descriptive and dimensional data:

	High Wolcott	Low Wolcott
Dam type	Embankment	Embankment
Height of dam	382 feet	288 feet
Dam crest length		
Main dam	4200 feet	2760 feet
Saddle dam (2)	4300 feet	Not required
Reservoir volume	350,000 af	160,000 af
Water source river	Eagle & Colorado	Eagle
Pump stations	2 each	1 each
Inlet tunnel length	6.3 miles	0.8 miles
Land required	2850 acres	1950 acres
Highway relocation	7.8 miles	7.8 miles

9.1 SITE TOPOGRAPHY

The valley bottom is approximately 500 feet wide at the proposed dam axis and the streambed elevation is 7030 feet. The right abutment is a moderately steep ridge rising about 500 feet above the valley bottom. The left abutment is lower, provided by the hills which form the east rim of the reservoir. Along the east reservoir rim, two saddle dams would be necessary to create the higher reservoir with 350,000 af of capacity.

9.2 STUDIES AND FIELD INVESTIGATIONS

A study of the Wolcott Dam and Reservoir was prepared for the DWB as part of the "Roberts Tunnel Collection System Study", by Parsons, Brinckerhoff, Quade & Douglas, Inc., and Forrest & Cotton, Inc., (1974). In that study, the Wolcott Dam and Reservoir Project was referred to as the Eagle-Colorado Dam and Reservoir. The study evaluated the feasibility of diverting water from both the Eagle and Colorado Rivers, providing regulatory storage at the Wolcott Site and pumping to Dillon Reservoir and the Roberts Tunnel.

Investigations included developing the subsurface stratigraphy in the area of the proposed dam by means of three borings (Parsons, et al. 1974). Nine auger borings were drilled in the valley area to delineate materials available for embankment construction. Geophysical investigations were carried out in the area of the dam and reservoir site and in the vicinity of the saddle dams and at an emergency spillway site west of the reservoir area. Exploratory boring locations, logs, geologic maps and test results are included in "Preliminary Geologic and Engineering Investigations Report" (Robinson & Associates, 1972) which was prepared as an appendix to the report of Parsons, et al. (1974).

During the course of this Study, published geological information was reviewed and correlated with field observations during two site visits. This information is incorporated into this report.

9.3 GEOLOGICAL DESCRIPTION

The geology of the region is influenced principally by the formation of the Gore Range to the east of the project site. This is described in Section 8.3 of the Azure Site.

The bedrock formations of the region include three major groups. Crystalline rocks are exposed in the core of the Gore Range. Sedimentary rocks, chiefly sandstone and shale with thin limestone beds, are exposed on the west side of the Gore Range. In the area of the proposed Wolcott Dam and Reservoir, volcanic intrusive and extrusive igneous rocks are exposed on the divide between the dam site and the Colorado River. Surface materials consist of colluvial deposits, glacial deposits and alluvial deposits.

The dam site and reservoir of the Wolcott Site are in an area of gentle northeast dipping sedimentary rocks. Most of the bedrock consists of shale and limy shale, with interbedded limestone, siltstone, and sandstone. The slopes above the stream valleys are covered by colluvium. The formations that would be involved in the development of the site are, in increasing order of age, the Pierre Shale, Niobrara Formation, Benton Shale, Dakota Sandstone and Morrison Formation.

9.4 DESIGN CONSIDERATIONS

Preliminary evaluation of the shale bedrock with its discontinuities and clay seams indicates that a broad dam base, as provided by an embankment dam, would be preferable to spread the loading and reduce stress concentration. Also, no aggregates for concrete have been encountered in the proposed construction area. These factors indicated the site is most suited for an embankment dam. Design considerations addressed in this section include foundation conditions, earthquake design, spillway requirements and construction materials.

Surface Material

At the dam axis, the borings indicate that the alluvium in the valley is approximately 20 to 25 feet thick, and the colluvium on the valley slopes is from 10 to 42 feet deep. The Parsons study (1974) recommended that the alluvium and weathered bedrock should be excavated to form a suitable foundation for the dam. That study estimated that the depths of these material could be up to 70 feet in several areas of the valley.

Rock Strength

The dam would be founded on the Benton Shale and Niobrara Formation. A boring at the dam site indicated that there are approximately 40 feet of weathered shale under about 20 to 24 feet of alluvium. The abutments of the dam would be keyed to the Benton Shale or Niobrara Formation. Clay seams were encountered in the Benton Shale. These layers form potential slide planes and will require further detailed evaluations at the final design level.

Foundation Permeability

Dakota Sandstone is the water bearing formation in this area of Colorado. It was encountered about 75 feet below the dam site and is exposed on the bluff along the Eagle River. The formation dips away from the Eagle River so there are no major springs along the river. Because the dip of the formation in general is opposite to the topographic slope, it should not present a problem for water retention in the reservoir.

Design Earthquakes

To develop dam stability during an earthquake, design criteria must consider the proximity of active faults as well as the probable magnitude of the seismic event. A preliminary evaluation of the earthquake potential in Colorado classified several faults within about 25 miles of the Wolcott Site as potentially active faults (Kirkham and Rogers, 1981). These include the Gypsum Fault, Gore Fault, Burns Fault, North Gore Range Fault, Blue River Valley Fault and the Frontal Fault.

A preliminary seismic assessment was made for the site in which maximum potential earthquake magnitudes were estimated on the basis of fault lengths. On the basis of this analysis, the maximum credible earthquake would be of Richter magnitude 6.7 occurring on the Gypsum Fault, located about 5 miles from the site.

An earthquake of a lesser magnitude than the maximum credible earthquake, with a more definite recurrence interval, is typically selected as the design basis earthquake. For preliminary design, the embankment geometry which has been chosen is considered appropriate in view of the potential dynamic loadings that can be anticipated at the site. It is possible that the alluvial sands and silts in the valley could have a low relative density which would indicate a potential for liquefaction. A conservative approach at this level of study would be to plan for the removal of the alluvium from beneath the area to be occupied by the embankment.

For final design, additional geologic seismic investigations would be conducted and a probabilistic analysis carried out to refine the design earthquake magnitude and distance from the site. The design dam section can be analyzed for dynamic loading using the selected accelerogram time histories for both horizontal and vertical earthquake induced forces.

Reservoir-Induced Seismicity

Reservoir-induced seismicity has been associated with the operation of several large reservoirs world-wide. Dam heights at these reservoirs ranged from 207 to 541 feet and reservoir capacities from 320,000 af and 130 million af. High Wolcott, with a height of 382 feet and a capacity of 350,000 af, would fall within this range. Reservoir-induced seismicity should not affect the safe operation of the reservoir where the dam is designed for strong ground motion. Further investigation would appear advisable.

Spillway Requirement

An analysis of the PMF conducted by Parsons, et al. (1974), concluded that the peak rate of inflow would be 21,600 cfs and the total inflow from the maximum precipitation event would amount to a volume of 12,000 af. Spillway design could be based on attenuation of the maximum rate of inflow or on temporary storage of the total volume of storm inflow or a combination of both.

Construction Materials

Locally available materials range from clays and silty clays to sands, gravels and boulders. Sands and gravels suitable for use in the filter, drain, and transition zones were not encountered in large quantities in the valley sediments but could be produced by off-site borrow sources, or by processing of alluvial deposits in the Eagle Valley.

Relatively flat external slopes of the dam are required to provide a broad dam base for overall stability of the dam and foundation. The flat slopes place little demand on the strength properties of the construction materials. Therefore, a wide variety of materials can be used in the construction of the embankment.

The total quantity of materials required to construct the high dam is estimated to be on the order of 46,000,000 cubic yards, based on 15 percent shrinkage. In addition a contingency of 50 to 100 percent is normally desired for construction materials sources with only preliminary subsurface investigation. When construction materials that are accessible to earth moving equipment in the valley bottom are exhausted, it will be necessary to develop rock quarries or haul from more distant borrow areas, possibly beyond the boundaries of the reservoir. A preliminary estimate of the quantity of easily obtainable materials is 20,000,000 cubic yards. In the cost estimates, an allowance has been included for the additional cost of materials in excess of this quantity.

9.5 PROPOSED DAM AND SPILLWAY

This section describes the selected dam type and the embankment and spillway sections for the dam at the Wolcott Site. The nature of the bedrock, orientation of the discontinuities and presence of slickensides and clay seams makes the stability of the foundation and abutments the controlling factor at the site. Under these conditions the embankment design is governed primarily by the stability requirements for the foundation.

Embankment Description

The external configuration of the embankment proposed in this report is based on preliminary stability analyses performed as part of the study by Parsons, et al., (1974) using parameters considered appropriate to the embankment materials and foundation conditions. The external slopes of the embankment have been set at 4:1 for both the upstream and downstream slopes. Weight berms, 200 and 400 feet wide on the downstream slope and 300 feet wide on the upstream slope, have been incorporated into the section in order to stabilize the foundation.

A moderately sloping core zone has been selected since core material quantities at the site may be relatively limited. Such a geometry will minimize the possibility for adverse deformations in the core resulting from consolidation of the zone during and subsequent to construction. The core-zone geometry was selected to provide a core-thickness-to-reservoir-head ratio on the order of 0.3 to 0.5 at the bottom of the core zone.

Filter, drain and transition zones have been incorporated into the preliminary design section. The upstream face of the embankment would be covered with riprap, consisting of cobble and boulder size rock, to protect the upstream shell zone against wave-action erosion. For the cost estimate of the present study the use of riprap with a long haul distance was assumed. The downstream face should be planted with natural grasses to control erosion.

The critical plane of failure may be along the clay seams in the foundation. The extent of these seams and the residual strengths of the clay must be verified prior to final design. It is anticipated that large diameter inspection shafts may be required to visually determine the extent of the thin clay seams.

Spillway Description

High Wolcott (350,000 af) - The proposed design involves the temporary retention of peak flood flows in the reservoir which would then be released over several days through the outlet works. A remote uncontrolled Ogee section spillway would be provided as a safety feature in the event that a malfunction occurs at the Colorado or Eagle River Pumping Plants. The spillway capacity would exceed the combined discharge from both the pumping plants.

Low Wolcott (160,000 af) - The spillway is proposed as a Morning Glory section with design capacity of 2350 cfs. The sill elevation would be 7347 feet with a radius of 18 feet. Safety over the function of this type of spillway is provided by the available storage of 12,000 af between the spillway crest and the crest of the dam.

Foundation Excavation

In general, the embankment should be founded on materials with strength and compressibility characteristics that equal or exceed the embankment materials. While there were apparently no standard penetration tests performed during the limited subsurface investigations carried out as part of the study by Parsons, it is reasonable to anticipate that the alluvial sands and silts in the valley will have a low relative density, due to the nature of unconsolidated alluvial deposits. Due to the potential for collapse of the structure in the loose deposits subsequent to construction of the dam and impoundment of the reservoir, as well as the potential for liquefaction of loose cohesionless materials when subject to seismic loading, the approach adopted at this level of study has been to plan for the removal of the alluvium from beneath the embankment. This will require the excavation of approximately 1.6 million cubic yards of material.

The actual amount of materials that may have to be excavated is a matter that would be determined during final design with the benefit of more comprehensive site investigations and stability analyses. Unless subsequent investigations identify the presence of significant deposits of materials that could jeopardize the structural integrity of the dam, consideration should be given to leaving acceptable portions of the alluvium and weathered bedrock in place. In order to reduce the risk of cracks forming in the core from differential settling, considerable rock excavation should be anticipated at the core zone contact with bedrock in locations where sharp breaks in slope exist.

Seepage Control

It is considered prudent to construct at least a single line grout curtain in the foundation of major earth structures in order to minimize the possibility of uncontrolled seepage through the foundation. Estimates have been made for a single line grout curtain beneath the core zone of the dam, and limited rim grouting in the abutments, for the purpose of sealing off the major joints and discontinuities in the foundation rock. The actual requirements of the grout curtain are a matter that must be established on the basis of a more extensive subsurface investigation and testing program.

Saddle Dams

For water levels greater than about elevation 7350, two saddle dams would be needed on the divides separating Alkali Creek from Ute Creek. The top width of the saddle dams will accommodate the proposed relocation of Highway 131.

The geology at the saddle dam sites consists of weathered rock and overburden. The bedrock is shale and thinly bedded limestone, which dips to the northeast or general downstream direction at each of the saddle dam sites. These planes can adversely affect the stability of the dams, as well as limit the grouting pressures that could be applied to the foundation. The combination of these two factors could limit the heights of these structures to about 100 feet. Further investigations during the final design phase are necessary to confirm this preliminary conclusion.

9.6 RIVER DIVERSION PUMPING SYSTEM

As an off-stream storage reservoir, Wolcott would be supplied with water by pumping from both the Eagle and the Colorado Rivers or from only the Eagle River.

Eagle River Diversion to Wolcott Reservoir

To divert water from the Eagle River into Wolcott Reservoir would require a pump station with diversion dam, intake works, conduit and tunnel. The diversion dam would be located on the Eagle River approximately 1500 feet downstream of the confluence with Alkali Creek. The dam would create a pool with a water surface elevation of 6907 feet. This would provide about 30 feet of submergence over the intake at the pump station. The intake to the Eagle River Pump Station would contain a sediment trap which could be flushed through the desilting pipe to a river section downstream of the diversion dam.

The Eagle River Pump Station would be equipped with five pumps, providing a total flow capacity of 600 cfs with about 460 feet total dynamic head. Each pump would operate with an 8000 horsepower electrical motor. Discharge from the Eagle River Pump Station would flow through an 8.5-foot diameter pipe to cross the Eagle River under the Diversion Dam and extend for 4500 feet in a tunnel to the terminal structure within Wolcott Reservoir.

Water released from the reservoir would flow by gravity back through the tunnel to the Eagle River Pump Station. At the pump station, the water would run the pumps in reverse, generating electricity. Power revenues could offset approximately 25 percent of pumping costs.

Colorado River Diversion to Wolcott Reservoir

To divert water from the Colorado River into Wolcott Reservoir would require a pump station with a diversion dam, intake works, tunnel. Also, channel improvements would be required to convey water from the tunnel outlet to the reservoir.

From the intake on the south bank of the Colorado River downstream from State Bridge, water would be pumped through approximately 31,000 feet of tunnel, discharge into Alkali Creek and flow into Wolcott Reservoir. Reservoir releases would pass through the Eagle River diversion facility.

The Colorado River Pump Station would be equipped with five, 15,000-horsepower pump units, each rated to pump at 120 cfs against a total dynamic head estimated at 835 feet. Total capacity of the pump station would be 600 cfs.

9.7 HIGHWAY RELOCATION

Relocation of approximately 6.9 miles of Colorado Highway 131 would be necessary. The re-alignment would begin approximately 1 mile northwest of the town of Wolcott and ascend the downstream side of the left abutment. It would follow the crest of the saddle dams and parallel the reservoir's east shoreline.

9.8 SUMMARY OF ESTIMATED CONSTRUCTION AND OPERATION COSTS

The high dam alternative at Wolcott would create a reservoir with 350,000 af of capacity and have a dam height of 382 feet. The low dam alternative at Wolcott would create a reservoir with 160,000 af of capacity and have a dam height of 288 feet.

Construction Quantities and Costs

Reconnaissance-level construction and operation cost estimates have been developed for both the high and low alternatives. The detailed cost estimates and the basis of their derivation are described in Appendix B. Estimated construction quantities, prices, and allowances have been combined to obtain a total construction cost. Costs are indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing have been added to obtain the total investment cost. Estimated construction quantities and estimated costs for the alternative sizes are summarized in Tables 9.1 and 9.2.

Operation Costs

Dam operation and upkeep would be carried out by two damkeepers, an annual technical inspection team and services of specialty contractors. The estimated cost of dam operation and upkeep at the alternative reservoir sizes is summarized on Tables 9.3 and 9.2. Recreation-type services are assumed to be offset by user charges and are not included in this estimate.

The systems to divert water from the Eagle and the Colorado Rivers would require full time operators during the approximately two months of pump operation. One operator with supervision would handle the reservoir release operations during the remainder of the years. Specialty services would be required for pump station and pipeline repairs. Energy consumption for pumping is estimated to be partially offset by energy generation from released flows. The estimated costs of river diversion system operation is summarized on Tables 9.3 and 9.4. Details of annual dam operation and pumping costs are in Appendix B.

TABLE 9.1

HIGH WOLCOTT DAM AND RESERVOIR

SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	2,850	acres	$ 1,710,000
River Diversion			50,000
Dam and Spillway			162,700,000
Shell Volume	39,914,500	cubic yards	
Core Volume	3,141,700	cubic yards	
Drain Volume	474,900	cubic yards	
Filter Volume	530,100	cubic yards	
Transition Volume	474,900	cubic yards	
Excavation	4,924,900	cubic yards	
Structural Concrete	5,800	cubic yards	
Mass Concrete	6,500	cubic yards	
Saddle Dam Embankment	7,199,000	cubic yards	
Outlet Works			380,000
State Highway Relocation	7.8	miles	10,700,000
Eagle River Pump Station			25,700,000
Pumps and Motors	5	each	
Structural Concrete	15,500	cubic yards	
Mass Concrete	6,270	cubic yards	
8.5-foot diameter tunnel	1,300	feet	
8.5-foot diameter pipe	3,700	feet	
12 x 15-foot tunnel	3,200	feet	
Colorado River Pump Station			79,660,000
Pumps and Motors	5	each	
Structural Concrete	15,850	cubic yards	
8.5-foot diameter tunnel	33,700	feet	
Subtotal of Direct Costs			$ 280,900,000
Contingencies (25%)			70,200,000
TOTAL DIRECT COST			$ 351,100,000
Engineering, Legal, Administration (20%)			70,200,000
TOTAL CONSTRUCTION COST			$ 421,300,000
Interest During Construction (7yrs.)			118,000,000
TOTAL CAPITAL COSTS			$ 539,300,000
Financing and Reserve			75,700,000
TOTAL INVESTMENT COST			$ 615,000,000

TABLE 9.2

LOW WOLCOTT DAM AND RESERVOIR

SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	1,950	acres	$ 1,170,000
River Diversion			40,000
Dam and Spillway			86,960,000
Shell Zone	16,528,200	cubic yards	
Core Zone	1,115,200	cubic yards	
Drain Zone	213,300	cubic yards	
Filter Zone	252,200	cubic yards	
Transition Zone	213,300	cubic yards	
Excavation	1,895,300	cubic yards	
Mass Concrete	106,200	cubic yards	
Structural Concrete	1,000	cubic yards	
Outlet Works			380,000
State Highway Relocation	7.8	miles	12,850,000
Eagle River Pump Station			25,700,000
Pumps and Motors	5	each	
Structural Concrete	15,500	cubic yards	
Mass Concrete	6,270	cubic yards	
8.5-foot diameter tunnel	1,300	feet	
8.5-foot diameter pipe	3,700	feet	
12 x 15 foot tunnel	3,200	feet	
Subtotal of Direct Costs			$ 127,100,000
Contingencies (25%)			31,800,000
TOTAL DIRECT COST			$ 158,900,000
Engineering, Legal, Administration (20%)			31,800,000
TOTAL CONSTRUCTION COST			$ 190,700,000
Interest During Construction (7yrs.)			30,500,000
TOTAL CAPITAL COSTS			$ 221,200,000
Financing and Reserve			30,800,000
TOTAL INVESTMENT COST			$ 252,000,000

TABLE 9.3

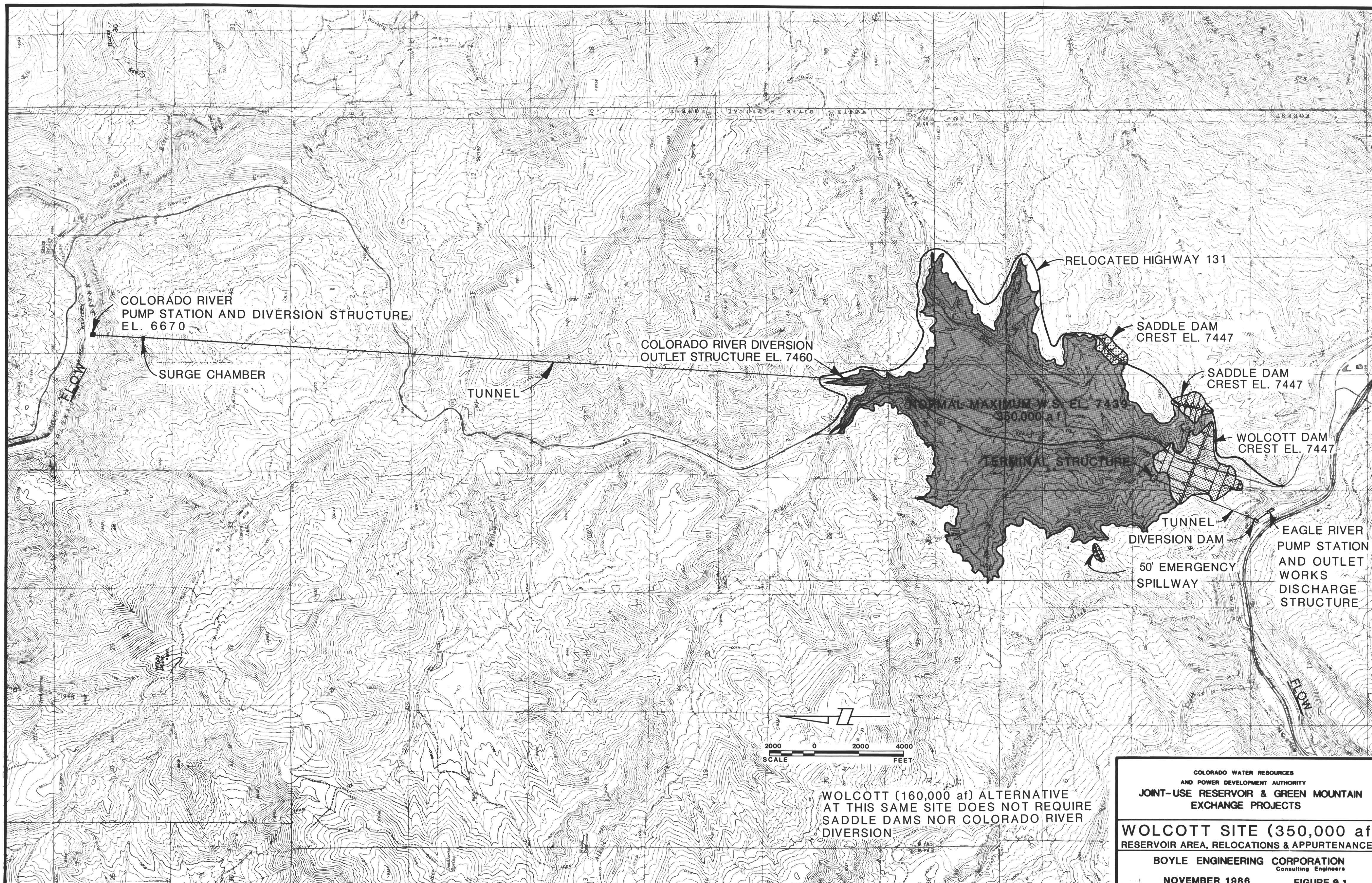
HIGH WOLCOTT DAM AND RESERVOIR
SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Dam Operation and Upkeep		$ 310,000
Pumping Labor and Maintenance		390,000
Operators and Station Labor	$ 157,000	
Equipment and Supplies	18,000	
Maintenance and Repair	215,000	
Pumping Energy		14,300,000
Credit for Generation		(3,570,000)
Total Estimated Annual Operation Cost		$ 11,430,000

TABLE 9.4

LOW WOLCOTT DAM AND RESERVOIR
SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Dam Operation and Upkeep		$ 210,000
Pumping Labor and Maintenance		190,000
Operators and Station Labor	$ 78,500	
Equipment and Supplies	9,500	
Maintenance and Repair	102,000	
Pumping Energy		4,950,000
Credit for Generation		(1,250,000)
Total Estimated Annual Operation Cost		$ 4,100,000



COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

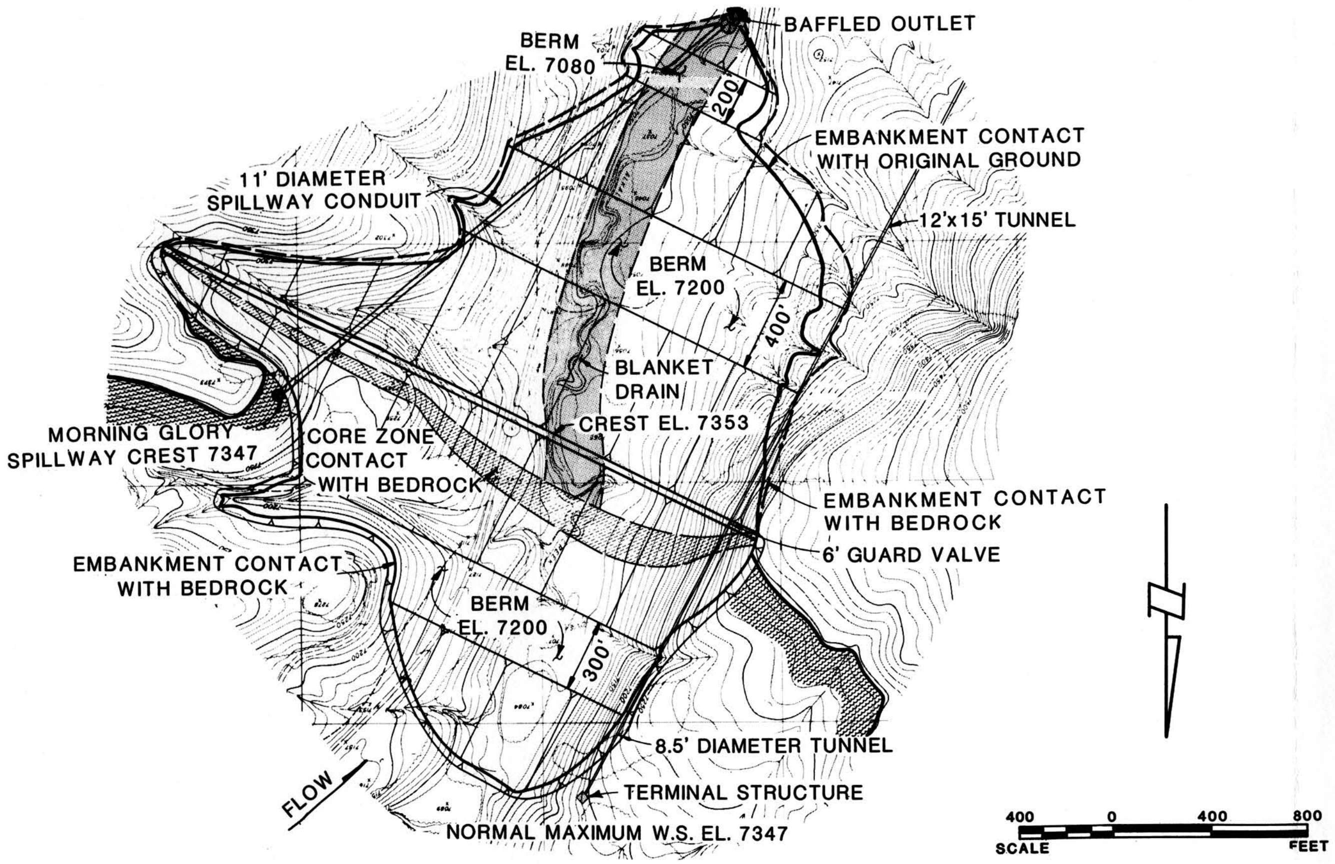
WOLCOTT SITE (350,000 af)

RESERVOIR AREA, RELOCATIONS & APPURTENANCES

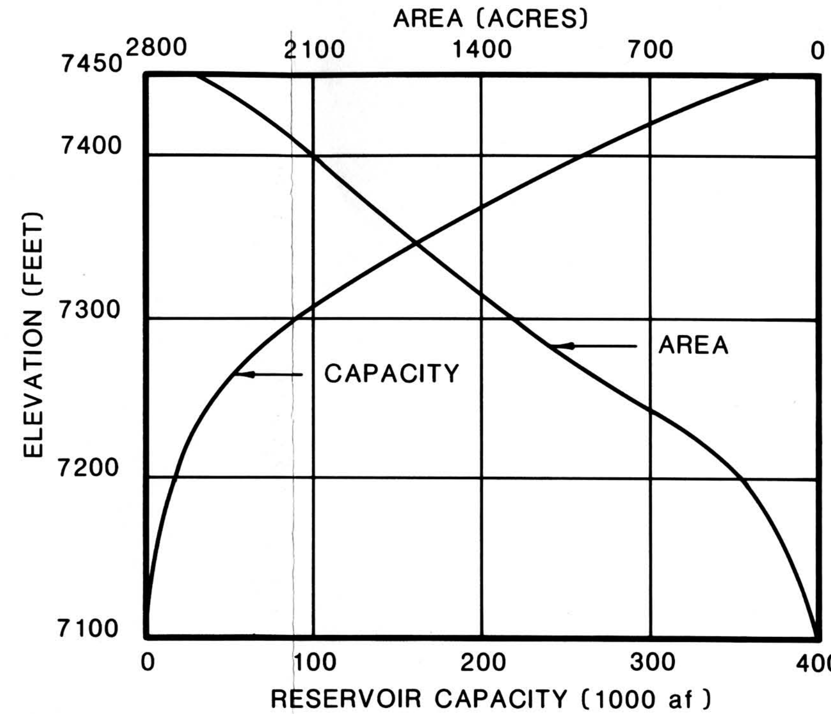
BOYLE ENGINEERING CORPORATION
Consulting Engineers

NOVEMBER 1986

FIGURE 9.1



DAM LAYOUT



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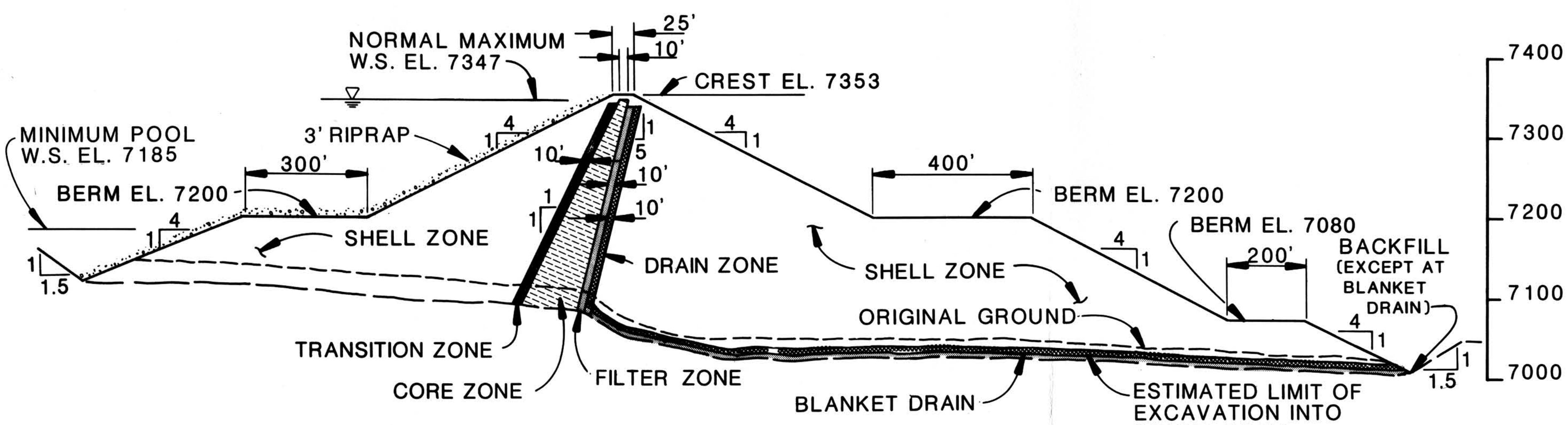

TYPICAL DAM SECTION

200 0 200 400
HORIZONTAL SCALE FEET

NOTE:

THESE SECTIONS ARE FOR THE 160,000 af CAPACITY WOLCOTT DAM.

COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

WOLCOTT DAM
LAYOUT AND SECTION

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FEBRUARY 1987 FIGURE 9.2

10.0 UNA DAM AND RESERVOIR

The Una Dam Site is located on the Colorado River at the Mesa-Garfield county line. It lies between the towns of Parachute and DeBeque, 35 miles northeast of Grand Junction. With respect to Colorado River operation, it is downstream from the Shoshone power plant diversion but lies upstream from the Cameo Gage and Grand Valley diversions. Figure 10.1 presents a plan view of the Una Reservoir Site.

Following is a tabulation of descriptive and dimensional data:

Dam type	RCC
Height of dam	130 feet
Dam crest length	2550 feet
Reservoir volume	150,000 feet
Land required	3800 acres
Railroad relocation	9.0 miles
Highway relocation	7.0 miles

10.1 SITE TOPOGRAPHY

Both dam abutments rise steeply from the 1100-foot wide valley floor at elevation 4925 feet. The right abutment, on the north side of the river, is about twice the height of the proposed dam. To the south of the river, the terrace which forms the left abutment would limit dam height to about 160 feet. The reservoir would extend 9.7 miles up the Colorado River valley to the town of Parachute. A high water elevation limit of 5058 feet would minimize flooding in Parachute. A reservoir of 150,000 af could be retained below this elevation by a 130-foot high dam.

10.2 STUDIES AND FIELD INVESTIGATIONS

A reconnaissance-level study entitled "Una Reservoir Project", was carried out for the Colorado Water Conservation Board by R.W. Beck and Associates (1982). It included hydrological analysis and preliminary project layout with cost estimates for an embankment dam and hydroelectric plant. An alternative RCC dam was also evaluated.

A reconnaissance investigation of the Una Dam Site was completed as a portion of the Bluestone Project, Colorado, by the USBR. An interim report for the project, "Feasibility Geologic Investigations", was published in 1967. A subsequent report, the "Bluestone Project, Colorado, Feasibility Report", was published in 1971.

The USBR (1967) geologic investigation included four borings located about 1000 feet downstream of the presently proposed dam site. No laboratory tests were reported.

The Beck (1982) field investigation included ten auger borings and twenty test pits to investigate potential borrow areas. A seismic refraction survey indicated the depth to various velocity layers, particularly the surface of the bedrock.

During the course of this Study, published geological information was reviewed and correlated with field observations during two site visits. This information is incorporated into this report.

10.3 GEOLOGICAL DESCRIPTION

The Una Reservoir Project is located in the Piceance Basin in western Colorado. The basin lies between the White River Plateau to the east and the Uncompahgre Plateau to the west. The Piceance Basin is comprised of a thick sequence of sedimentary rock beds. The Colorado River has eroded a southwest-trending valley across the basin through the sequence of shale, marlstone and oil shales of the Green River Formation, and into the mudstone and sandstones of the Wasatch Formation (Beck, 1982).

The dam site is located approximately 10 miles west of the axis of the Piceance Basin. In this area the Colorado River occupies a relatively narrow valley. Bedrock which underlies the dam site consists of the middle and upper mudstone and sandstone members of the Wasatch Formation. The river valley tends to be relatively narrow where it passes through the more resistant sandstone units. In areas of the valley where the mudstone shale is predominane, a broader valley section is characteristic.

10.4 DESIGN CONSIDERATIONS

Geotechnical investigations indicated that the Una Site was suitable for either a concrete or earth dam. Preliminary evaluation indicated that the relatively weak nature of the bedrock would be expected to require additional base width for a concrete dam. Also, the alluvial deposits in the project area reportedly contain high percentages of oil shale which may exclude their use in high quality concrete. A major factor in the selection of type of dam is the unusually high PMF estimated for this site. Current criteria results in a greater spillway flow requirement than considered in prior studies. This would favor the selection of a concrete dam that could provide a spillway along most of its crest length.

Surface Material

Surface deposits which cover the bedrock in the vicinity of the dam site consist of river alluvium and terrace deposits near the river, and alluvial fan and debris flow deposits primarily north of the river. Talus deposits occur at the base of the steep slopes. The results of the geophysical investigation indicate that the maximum thickness of overburden may be approximately 50 feet. Debris flow deposits covering the river gravels on the north side of the river contain low density materials that are subject to collapse when loaded and subsequently inundated (Beck, 1982).

Rock Strength

Alternating sandstone beds and shale units exposed on both abutments have been correlated. The lowest sandstone bed exposed on the right abutment at the proposed dam axis has been followed along the valley wall for a distance of more than one mile downstream and found to change very little in physical character from place to place. The sandstone unit averages about 14 feet thick but locally thickens to about 22 feet. Beds strike approximately N.60°W. and dip in the upstream direction from 2° to 8° toward the northeast. Projections of surficial geologic mapping, seismic refraction data and USBR logs indicate that the bedrock pattern of alternating sandstone beds and shale units continues below the surface. Conformation with borings extends to elevation 4855, approximately 80 feet below the river banks.

It is anticipated that the dam would be founded on the first sandstone bed below the overburden of surface material. For analysis of stability, properties of both the sandstone and the underlying mudstone shale would be pertinent. No tests of site materials have been conducted to obtain representative rock strengths. From the rock mass rating system of Bieniawski (1976) values of fair and poor were selected as representative of the sandstone and mudstone shale, respectively. Internal friction and apparent cohesion were then estimated using Mohr failure envelopes (Hoek and Brown, 1980). Details are in Appendix B. Values derived correlated well with those estimated by Beck (1982).

Design Earthquake

To develop dam stability during an earthquake, design criteria must consider the proximity of active faults as well as the probable magnitude of the seismic event. The Piceance Basin, specifically in the vicinity of the project area, has no known potentially active faults.

Toward the west, the boundary area between the Uncompahgre Uplift and Piceance Basin is marked by a series of northwest-trending faults. One of these faults, 38 miles south of the project site, has been reported to have moved during Quaternary time. To the east, the boundary area between the White River Uplift and the Piceance Basin is also marked by a series of northwest trending faults. These features are located approximately 45 miles east of the project area.

The work by Converse Consultants, Inc., reported by Beck (1982) concludes that while potentially active faults in the region could produce earthquakes with magnitudes greater than 6.0, that due to their great distance from the project site they would produce small on-site bedrock accelerations, less than 0.2g. The slopes of the dam faces and safety against sliding employed at this level of preliminary design afford adequate margin to accommodate earthquake-induced forces from this degree of ground motion.

Spillway Requirement

The spillway size for the Una Dam is based on the PMF which has a peak inflow of 340,000 cfs. The inflow hydrology was developed using Hydrometeorology Report No. 49 (NOAA, 1984) as a basis for the Probable Maximum Precipitation and the Corps of Engineers Hydrologic Engineering Center's HEC-1 mathematical modeling of the watershed for the inflow hydrograph (COE, 1981).

Construction Materials

Bulk samples were obtained from both the auger borings and test pits completed for the borrow investigation. Selected samples were tested in the laboratory. Tests included grain size analysis, moisture content, Atterberg limits, specific gravity, and compaction data.

Sand and gravel deposits in the valley along the Colorado River between Rifle and Grand Junction contain a high percentage of weathered shale and oil shale fragments that make them undesirable for high quality concrete. The aggregate was reported by the Materials Section of the Colorado Department of Highways to meet all requirements for concrete except freeze-thaw.

A local aggregate materials extraction operator reported that the sand-size fraction was acceptable for concrete but the coarse-grained fraction contains a major portion of the oil shale fragments. At the present time local producers of concrete obtain aggregate either from gravel deposits on the Colorado River in the Rifle area upstream of the oil shale contaminated gravels, or else from the Grand Junction area, where through natural transport downstream the river has reduced oil shale fragments to fines and washed them from gravel deposits (Beck, 1982).

For cost estimates for RCC construction, it has been assumed that the sand-sized fraction could be obtained near the proposed dam site but that the larger size aggregate would be imported from either the Rifle or Grand Junction areas. All aggregate for the facing concrete that must withstand the freeze-thaw cycle has been estimated to be imported to the construction site.

10.5 PROPOSED DAM AND SPILLWAY

Both an earth embankment dam and an RCC gravity dam were considered for the Una Site. The controlling design parameter was the large spillway capacity required to pass the PMF. The RCC gravity dam design allows most of the crest length of the dam to be used for spillway. Total crest length would be 2550 feet. The spillway of an embankment dam would be constructed of concrete over an excavation into the left abutment. Total crest length including the spillway was estimated as 3400 feet. The construction quantities involved with the embankment design exceeded those estimated for the RCC design. The RCC dam is proposed as being better suited to the physical requirements of the Una Site.

Dam and Overflow Description

The interior of the dam would be constructed of RCC with conventional concrete on both faces and the overflow section. A section is shown in Figure 10.2. The downstream face, with a slope of 0.75:1, provides a section that would meet internal stress requirements at all levels of the dam. The downstream facing of the Ogee overflow would be stepped for ease of construction and to provide some measure of energy dissipation during spillway operation.

Sliding stability to resist four times the static loading at the contact with the sandstone foundation rock was provided by adding a 4-foot thick concrete spillway apron extending 350 feet downstream from the toe. It would be doweled into bedrock to resist uplift. As well as increasing sliding stability the apron would provide erosion protection.

Internal drain holes would be drilled through the RCC material from near the top of the dam into a drainage gallery. Weep holes would be provided to relieve uplift pressure under the aprons.

Overflow Capacity

The overflow was designed as an uncontrolled Ogee spillway section with a lower sill elevation for the principal spillway section. The placement and length of the overflow sections were based on the present river location and the projected storm frequency. The principal spillway is sized to pass the 100-year flood of 41,700 cfs while larger floods would pass over both spillway sections. The PMF stage would flow over the primary spillway at a depth of approximately 17 feet.

Foundation Excavation

All overburden would be removed from the area to be occupied by the gravity dam and spillway apron. The sandstone bedrock beneath the dam and the apron would be cleaned of weathered material to receive the foundation. Foundation treatment would include excavation of weathered materials and backfilling with concrete. In some locations eroded zones would be shaped by filling with concrete. Consolidation grouting would be carried out to improve continuity, strength and stiffness of the foundation rock.

Seepage Control

A single row grout curtain is anticipated near the contact of the upstream face of the dam and the foundation with the deepest holes extending approximately 100 feet below the rock surface. Lateral hole spacing should be at 10-foot centers with split spacing where large grout takes are encountered.

10.6 APPURTENANCES TO THE DAM

Appurtenances include the outlet works for the Colorado River and the power plant.

Outlet Works

The outlet works would be located in the left abutment. Design discharge is 17,400 cfs to meet the requirement of the Colorado State Engineer that the reservoir could be drawn down 5 feet in 5 days. Trashracks, the inlet to the outlet works pipes, the guard valves and the guard valve chamber would all be contained in a concrete structure at the face of the RCC dam. Access to the valve chamber would be by stairwell from the top of the dam face structure.

Hydropower Plant

The power plant would be located downstream of the left abutment. Bifurcation from two of the five 12.5-foot diameter outlet works conduits would supply the turbines. Maximum power generation would be 17,600 kilowatts. Two Francis turbines operating on flows bypassed or released to meet downstream requirements and flows which would be spilled would produce an average of 88.5 million kWh per year. Evaluation of hydropower at this dam is described in Section 12.2.

10.7 RELOCATIONS

Both railroad and highway relocations would be required. Approximately three dwellings and ranch structure groups would be purchased when acquiring land for the reservoir.

Railroad

Relocation of the D&RGW Railroad is required to raise the grade above the proposed water surface level along the reservoir. From approximately 1 mile northeast of DeBeque the railroad on the relocated alignment would maintain a maximum grade of 0.8 percent to the dam site at elevation 5080. From that point the grade would be level to a junction with the existing track approximately 1 mile southwest of Parachute. Total length of relocated railway would be 9.0 miles.

The relocation includes approximately 1.5 miles of passing track, a 450-foot single-track bridge to overpass Interstate 70 and U.S. Highway 6, and a 3000-foot single-track tunnel to pass through the ridge that forms the right dam abutment.

Highway

Relocation of approximately 7 miles of U.S. Highway 6 is required to raise the grade above the maximum normal water surface elevation of 5058 feet. The alignment selected for the relocation parallels the relocated section of the D&RGW Railroad. The road would also provide access to the reservoir.

10.8 SUMMARY OF ESTIMATED CONSTRUCTION AND OPERATION COSTS

A RCC dam at the Una Site, with a dam height of 130 feet and crest length of 2550 feet, would create a reservoir with 150,000 af of capacity. Reconnaissance-level cost estimates have been developed for both construction and operation costs. The detailed cost estimates and their derivation are described in Appendix B. Estimated construction quantities, prices, and allowances have been combined to obtain a total construction cost. Costs are indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing have been added to obtain the total investment cost. Estimated construction quantities and estimated costs are summarized in Table 10.1 for Una Dam and Reservoir, and in Table 10.2 for hydropower addition to Una Dam and Reservoir.

TABLE 10.1
UNA DAM AND RESERVOIR
SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	4,040	acres	$ 2,540,000
River Diversion			3,000,000
Dam and Spillway			78,670,000
Excavation	1,835,300	cubic yards	
Mass Concrete	117,200	cubic yards	
Structural Concrete	100,420	cubic yards	
Outlet Works			14,970,000
Access Road	0.8	miles	320,000
Highway Relocation	6.7	miles	15,600,000
Railroad Relocation			31,800,000
Tunneling	0.6	miles	
Total	9.9	miles	
Subtotal of Direct Costs			$ 146,900,000
Contingencies (25%)			36,750,000
TOTAL DIRECT COST			$ 183,650,000
Engineering, Legal, Administration (20%)			36,750,000
TOTAL CONSTRUCTION COST			$ 220,400,000
Interest During Construction (4 yrs.)			35,300,000
TOTAL CAPITAL COSTS			$ 255,700,000
Financing and Reserve			35,300,000
TOTAL INVESTMENT COST			$ 291,000,000

TABLE 10.2

HYDROPOWER ADDITION AT UNA DAM AND RESERVOIR

SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Power Plant			$ 10,130,000
Turbine Generator	2	each	
Penstock	175	feet	
Rock Excavation	350	cubic yards	
Concrete	3,000	cubic yards	
Transmission and Connection	1.0	miles	1,420,000
Subtotal of Direct Costs			$ 11,550,000
Contingencies (25%)			2,900,000
TOTAL DIRECT COST			$ 14,450,000
Engineering, Legal, Administration (20%)			2,900,000
TOTAL CONSTRUCTION COST			$ 17,350,000
Interest During Construction (4 yrs.)			2,750,000
TOTAL CAPITAL COSTS			$ 20,100,000
Financing and Reserve			2,900,000
TOTAL INVESTMENT COST			$ 23,000,000

Dam operation and upkeep would be carried out by two damkeepers, an annual technical inspection team and services of specialty contractors. The estimated average annual cost of dam operation and upkeep at the proposed Una Dam and Reservoir is $200,000. Recreation-type services are assumed to be offset by user charges and are not included in this estimate.

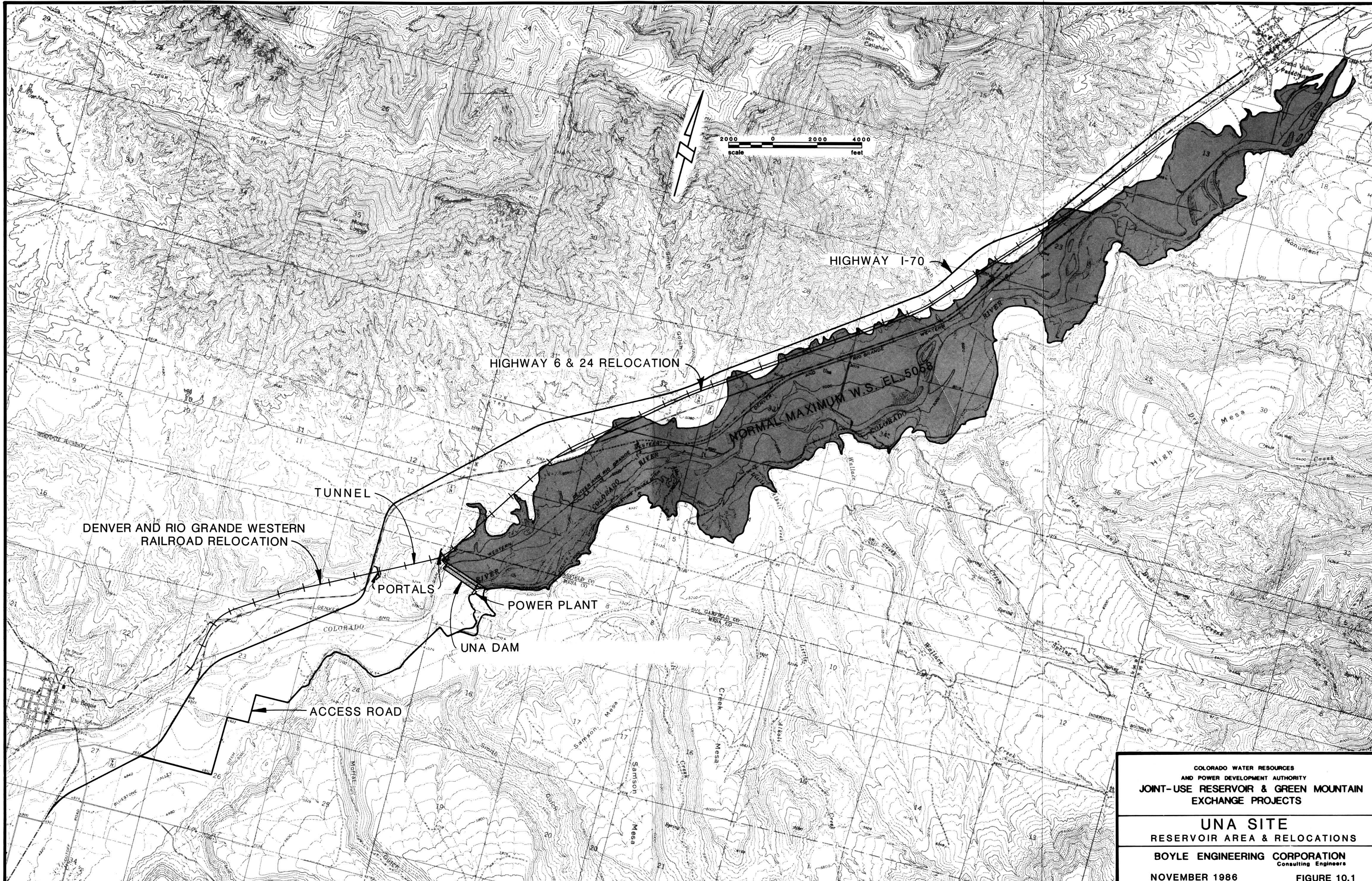
Operation of the hydropower plant has been estimated to require full-time operators. Specialty services would be employed for equipment repair and maintenance not performed by operators. Estimated annual costs are summarized in Table 10.3. Details of pumping and dam operation costs are in Appendix B.

TABLE 10.3

UNA DAM AND RESERVOIR

SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Total Dam Operation and Upkeep	$ 200,000
Hydropower Plant Operation	
Operators, Supervision	$ 176,000
Equipment and Supplies	16,000
Repair and Replacement	128,000
Total Hydropower Plant Operation	$ 320,000



COLORADO WATER RESOURCES
AND POWER DEVELOPMENT AUTHORITY
JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

UNA SITE
RESERVOIR AREA & RELOCATIONS

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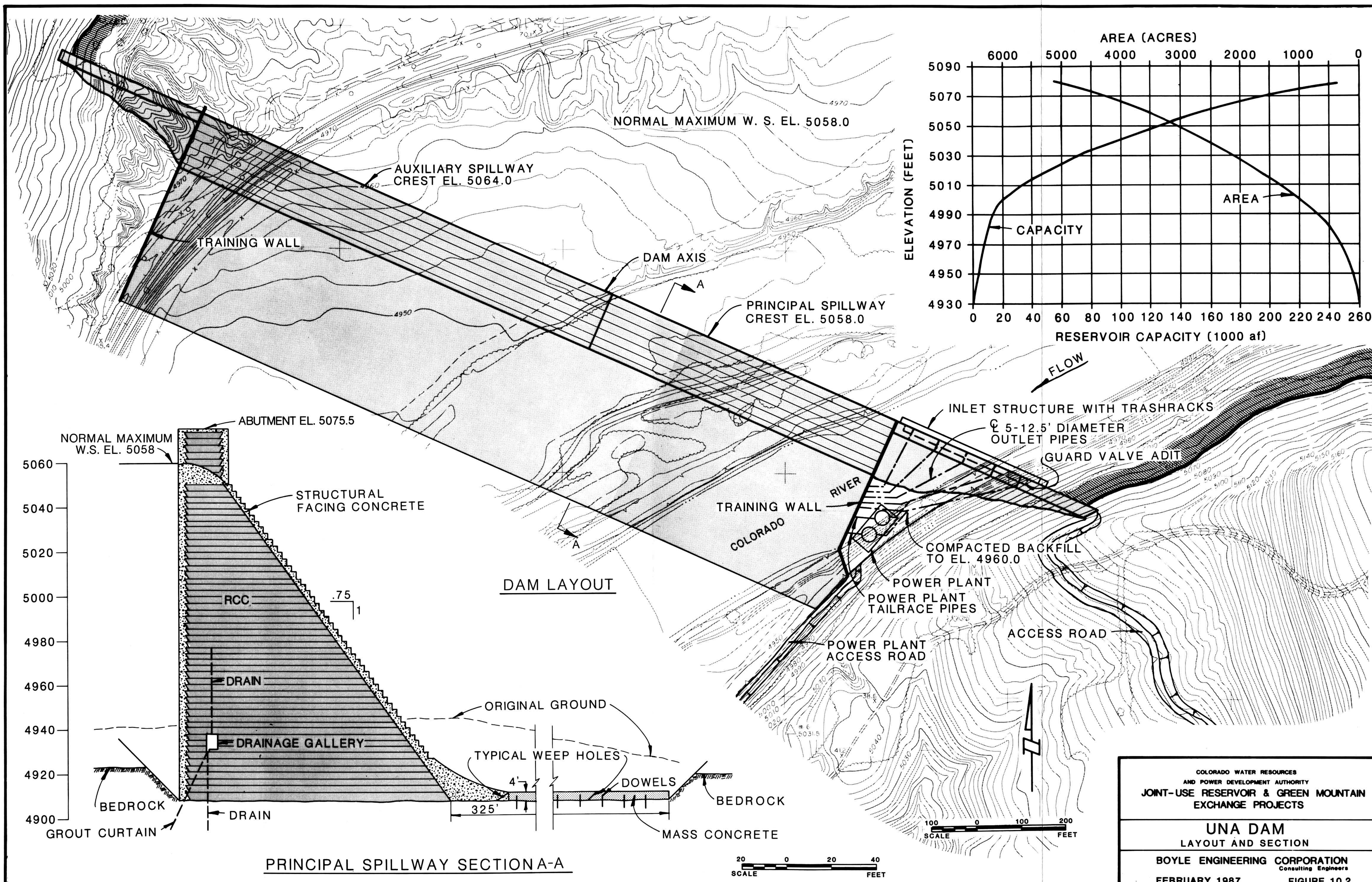
NOVEMBER 1986 FIGURE 10.1



COLORADO WATER RESOURCES AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

UNA DAM

LAYOUT AND SECTION

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FEBRUARY 1987

FIGURE 10.2

11.0 GREEN MOUNTAIN RESERVOIR TO DILLON RESERVOIR CONVEYANCE SYSTEM

The Green Mountain Reservoir to Dillon Reservoir conveyance system is the pumpback element of the proposed Green Mountain Exchange Project. New Replacement Reservoir(s) would make water stored in Green Mountain Reservoir available for use in the Green Mountain Exchange Project. This water would be pumped to Dillon Reservoir to augment the flow in Roberts Tunnel.

11.1 LOCATION

Both Green Mountain and Dillon Reservoirs are located on the Blue River, a tributary to the Colorado River. Green Mountain Reservoir is located 26 miles downstream of Dillon Reservoir. The difference in water surface elevation between the two reservoirs is 1070 feet. A map of the system is presented in Figure 11.1.

11.2 FLOW CAPACITY

Two different pumping capacities have been studied for the conveyance system: 12,000 af per month and 8,000 af per month. These capacities were selected for the purpose of providing a reference range of cost estimates to cover the various alternative pumpback capacities which may eventually be incorporated into the water exchange project.

11.3 ROUTE AND GEOTECHNICAL CONSIDERATIONS

The selected pipeline route follows the west shore of Green Mountain Reservoir and the west side of Colorado Highway 9 to the town of Silverthorne. It would cross under Interstate 70 along side the Blue River and proceed to a below-ground terminal near the east abutment of the dam at Dillon Reservoir. Using this route would allow the installation of the pipeline in the highway right-of-way where practical. It would require only one crossing of the Blue River. Selection of the route is described in Section 2.2.

A geotechnical investigation was conducted by Chen & Associates as a part of this Study (Chen, 1986). Investigation included review of previous reports, ten field borings along the route and laboratory testing of materials. Exploratory boring locations and logs are included in Appendix C.

Surface material encountered was generally clean to silty sand and gravel with cobble and boulders. Excavation of the material and road fill along most of the pipeline corridor should be possible with heavy duty excavation equipment. Difficult excavation conditions can be expected where the trench encounters large boulders or cemented beds of mudstone and sandstone. Based on this investigation, bedrock should be anticipated within 10 feet of the ground surface in five zones with a total length of 17,000 feet.

The pipeline alignment would cross areas of large ancient landslides and areas of more recent shallow landslides. The large ancient landslides showed no signs of present movement based on cursory observation. The potential for future movement of these large landslide masses was estimated as relatively low (Chen, 1986). Small, relatively shallow recent slope failures, occurring along the pipeline corridor on the steep slopes, should not represent a major impact to the proposed construction.

Dewatering of the pipeline excavation might be necessary in many areas. Free water was encountered in five of the ten borings at depths of less than 10 feet below the ground surface. Dewatering could be accomplished by a combination of well points, drain sumps and pumps.

11.4 PUMP STATIONS

This section describes the pump stations, their number, siting, equipment and power source. The pipeline length between the intake at Green Mountain dam and the discharge at Dillon Reservoir would be about 140,900 feet. The average water surface elevation at the intake at Green Mountain Reservoir, based on projected high future level of water use would be 7883 feet. The discharge elevation into Dillon Reservoir would be 9030 feet. The resultant static lift would be 1147 feet. Friction losses are approximately 208 feet for an estimated total pump lift of 1355 feet.

Number of Pump Stations

For the purposes of this report, cost estimates are based on the use of conventional equipment. To attempt to handle this flow and lift with one or two stations would involve extraordinary pressures of 300 to 600 psi in the pumps and in the pipeline. This would be beyond the range of normal municipal water system equipment and beyond the experience of maintenance personnel. Dividing this lift among three pump stations, however, would permit the use of conventional pumping and surge control equipment. The average discharge working pressure would be 196 psi. An economic comparison between three-pump-station and four-pump-station alternatives found the three-pump-station alternative more cost effective.

Pump Station Siting

The space requirement for a pump station would be about 3 acres. This space would be developed to contain:

- An open forebay reservoir with a water surface of about 1.5 acres. It would receive the discharge of the conduit from a lower pump station and supply the pumps of the site pump station. Differences in the pumping rates of succeeding pump stations would be accommodated by changes in storage of the forebay reservoirs.

- A pump station building of approximately 130 feet by 50 feet located partially underground. It would house three operating and one standby pumping units, control valves, electrical switchgear, and an operating and maintenance office.

- A surge control building, approximately 40 feet by 60 feet. It would house air and water cushion pressure tanks to control the transient hydraulic pressure waves which occur following an electrical power failure in an operating pumping system.

- An access driveway with truck turn-around space.

- An open area containing an electrical switchyard at the termination of the electrical supply transmission line.

Location of pump stations were selected to satisfy the following criteria:

- Ground elevations which permit distributing the total pumping lift among the three stations so that similar sized pumping equipment could be employed and pressures could be maintained below 250 psi.

- Space for a forebay reservoir at a higher elevation than the pump building to satisfy the intake pressure requirement of the pumps.

- Areas away from ancient or active landslides.

- Terrain not presently dedicated to a use incompatible with a pump station.

Two sites were found along the pipeline route that meet these criteria. One is located south of Slate Creek on the west bank of the Blue River and the other is located west of Colorado Highway 9 about 4500 feet north of the Silverthorne sewage treatment plant. Each of these sites would accommodate a pump building, a surge control building and a forebay reservoir.

The intake pump station site would be at the base of the dam for Green Mountain Reservoir close to the existing hydropower plant. It would draw water from a penstock which supplies the power plant. A bifurcation structure would be constructed immediately upstream of the power plant to branch into the pump station header piping. Pump station structure dimensions would be approximately 130 feet by 50 feet and the interior crane would have 25 feet vertical clearance over the pump base floor. The locations are shown on Figure 11.1.

Alternative Intake Location

The orientation of Green Mountain Reservoir suggests an alternative water intake location. An alternative intake within the lake and a pump station at the lake shore near Heeney would save about 2.5 miles of pipeline as compared with the location at the base of the dam. No subsurface drilling nor geotechnical evaluation has been carried out at the site. At this stage of investigation, it can not be projected that the savings in pipeline length would be sufficient to offset the more expensive pump station and intake structure and the contingencies involved in underground works. Further consideration can be given to the lake intake alternative at a later stage of investigation, prior to final design.

Pump Station Equipment

Each of the three pump stations would house four horizontal, single-stage, double-suction pumps. Three pumps together would provide the design flow of the station. The fourth pump at each station would be a "stand-by" to cover shut downs for repair and maintenance. Normal operation would be rotated among the four units to equally distribute operating time.

Design flow of the pump stations for the 12,000 af per month system would be 199 cfs or approximately 90,000 gallons per minute (gpm). Cost estimates have been based on 42-inch diameter suction by 30-inch diameter discharge Allis-Chambers model WSID pumps operating at 600 revolutions per minute (rpm). Each unit would be capable of pumping 30,000 gpm at 500 feet of head with a peak efficiency of 82 percent.

For the 8000 af per month system, design flow would be 134 cfs (approximately 60,000 gpm). Cost estimates have been based on 30-inch diameter suction by 20-inch diameter discharge Allis-Chambers model WSID pumps operating at 885 rpm. Each unit would be capable of pumping 20,000 gpm at 500 feet of head with a peak efficiency of 82 percent.

Pump station layouts are shown on Figure 11.2. The pumps were positioned to provide access for servicing. An overhead crane would permit removal of the motor and the pump for replacement or repair.

Motors

Motor size requirements range from 3000 horsepower at Slate Creek Station for 8000 af per month capacity to 5000 horsepower at the Green Mountain Dam Station for 12,000 af per month capacity. Motor speeds would be 600, 720 or 885 rpm. In this range either induction or synchronous motors would be appropriate. The choice at the time of final design may be based on cost of motor and controller or operator preference. Cost estimates have been based on synchronous motors with reduced voltage starting and an operating voltage of 4160 volts. Source of electrical supply is described in Section 11.6.

Valves and Header Piping

A power-operated plug valve, known as a rotovalve, located on the discharge side of each pump, would control pump discharge during pump start up and shut down. Upon power failure it would close by use of a stored-pressure hydraulic system. On the suction side of each of the four pumps, butterfly valves would serve to isolate the pumps from the reservoir.

Suction and discharge header piping would contain the valves, gages and operational control sensors and connect the pumps with the forebay reservoir and the conveyance pipeline. The header piping includes several changes in diameter which effect an energy-efficient transition between the 7 to 13 feet per second flow velocity at the pumps to the 6.5 feet per second velocity of the conveyance piping.

11.5 PIPELINE

This section describes the selection of pipe size, forebay reservoirs and surge control structures to optimize pumping energy and system capacity. Major pipeline appurtenances are also described.

Optimization of Pumping Energy and Pipe Diameter

The optimum pipe diameter is an economic selection between pumping energy and pipe cost. A large diameter, more expensive, pipeline offers less resistance to flow and requires less pumping energy.

The Green Mountain Reservoir to Dillon Reservoir pumpback pipeline size was optimized on a present worth basis for the flow rates of 8000 af per month and 12,000 af per month. The optimal size economic analysis considered construction costs of the conveyance facility elements: pipelines, pump stations, surge chambers, pump station forebay reservoirs, and the operation and maintenance costs. Electrical energy and connected load rates were escalated at 0.5 percent per year. This is the amount that power costs would exceed the general inflation rate as projected through year 2003 by Public Service Company of Colorado (P.S.Co., 1986). Labor costs, repair, replacement and construction costs were all assumed to maintain a constant relationship with each other, so were not escalated.

The present worth of future costs of the alternatives were discounted to a common date of start of construction at an interest rate of 8 percent. Contingencies, engineering and administration costs, interest during construction, reserve funds and financing expenses were included.

The lowest present worth cost of the 8000 af per month facility occurred with a 63-inch pipeline and the lowest cost of the 12,000 af per month facility employed a 75-inch diameter pipeline. These sizes have been used as the basis for estimating construction quantities and costs.

Forebay Reservoir Size and Siting

The use of a reservoir between pump stations permits some flexibility in pump operation. Pump flows would not be exactly matched. With forebay reservoirs to regulate the differences in flows between stations, the necessity to cycle pumps on or off line would be reduced. Also, hydraulic surges would be isolated by the open reservoirs. The capacity of the reservoirs was selected to provide at least 6 af of usable storage to assure that no pump need be started more than once per hour.

Surge Control

Sudden acceleration or retardation of fluid motion would cause pressure surge waves in a pipeline. Surge, also called water hammer, is usually initiated by valve closure or the starting or stopping of pumps. The critical case for the Green Mountain Reservoir to Dillon Reservoir pipeline would occur with an electrical power failure that would stop all the pumps at a station. Surges originating at any one of the three pump stations would be isolated from the next pump station and its associated discharge pipeline by the forebay reservoirs.

To predict the surge, an analysis was conducted using Boyle's Surge Analysis computer program. The computer program facilitated the analysis of several models employing different sized devices selected to reduce the effect of surges. Evaluating the models, air-water cushion pressure vessels and open-ended surge lines proved to be optimal for two pipeline reaches, and the pressure vessel alone for the third reach. Table 11.1 shows the sizes of the surge control devices developed by the model studies to serve as a basis for cost estimates for pipeline surge protection.

TABLE 11.1

ESTIMATED SIZES OF SURGE CONTROL DEVICES

LOCATION	PRESSURE VESSEL	OPEN-ENDED SURGE LINE
Green Mountain Pump Station	one 4500 cu. ft., 350 psi maximum	66" diameter 1100 ft. long
Slate Creek Pump Station	two 4000 cu. ft., 250 psi maximum	72" diameter 1000 ft. long
Blue River Pump Station	three 4500 cu. ft., 300 psi maximum	None

Conveyance Discharge and Metering Structure

The conveyance system outlet would be located near the eastern end (right abutment) of Dillon Dam. The proposed structure consists of an enclosed channel which receives the pipeline discharge and crosses under the roadway at the crest of the dam. The channel section would widen to accommodate a 20-foot weir. Below the weir, a stair-stepped open-channel section would provide energy dissipation for the flow from the outlet structure to Dillon Reservoir. An instrument shelter adjacent to the weir, would contain the water level sensor transmitter to communicate rate-of-flow data to the centralized control center.

Alternative Discharge Location

A conveyance system discharge alternative has been proposed that would avoid mixing Green Mountain Reservoir water with Dillon Reservoir water. Approximately 21,800 feet of additional pipeline and a junction structure would be required to conduct the flow directly into Roberts Tunnel. The tunnel intake, at elevation 8846, is 171 feet below the full water surface of Dillon Reservoir. For the pipeline discharge to enter the tunnel without circulating into the reservoir, a vertical shaft and energy dissipation structure, a metering facility and possible modification of the existing tunnel inlet control works may be required.

Air Valves

Vaults containing air valves would be located at every high point along the conveyance pipeline. Water flowing in a pipeline releases dissolved air which accumulates at the high points. If not properly vented, this entrapped air would restrict pipeline flow. Air valves allow this air to escape and also vent air during initial pipeline filling. Air vacuum valves are also included in the design to admit air and prevent negative pressure when the line is drained. Access manways in air valve vaults provide entry to the dewatered pipeline for inspection and maintenance.

Pipeline Drains

The preliminary design includes outlets with drain valves, sometimes called blow-offs, at every low point in the pipeline. During pipeline draining, water would flow into the drain vault and either through the top of the vault and onto the ground surface or where terrain permits, through a drain pipe into a stream bed or drainage channel. The drain valve would be controlled from the ground surface. Two gate valves would provide redundancy in this major diameter pipeline to allow valve replacement or repair.

Stream Crossings

At eighteen points along the main line, there are creeks or streams that would be crossed. The conveyance pipe would be buried in the stream bottom and encased in concrete at the crossing for protection from erosion and to prevent flotation of the pipeline when it is dewatered.

11.6 CONVEYANCE SYSTEM OPERATION

This section describes the schedule of operation, electrical power requirements, costs and contingencies, and manpower requirements. All contribute toward evaluation of the cost of operation.

Schedule of Operation

The conveyance system is intended to supply Dillon Reservoir with water from Green Mountain Reservoir, to supplement the storage at Dillon, and to meet the diversion demand of Roberts Tunnel. The diversion demand has been established by DWB based upon a 28-year hydrological record and projected water needs. Under these conditions the annual amount of water that would be supplied by the alternative pipeline capacities was analyzed. The minimum supply would be provided by the 8000 af per month capacity pipeline operating with 95,000 af of available effective storage in Green Mountain Reservoir. The maximum supply would be provided by 12,000 af per month capacity pipeline operating with 147,000 af of available effective storage. The range of annual quantities of water that would be conveyed are presented in Table 11.2.

TABLE 11.2

RANGE OF CONVEYANCE SYSTEM FLOWS IN AF PER YEAR
TO MEET AVERAGE SUPPLEMENTAL WATER REQUIREMENT OF DILLON RESERVOIR

Precipitation Classification	95,000 af Storage 8000 af/mo System	147,000 af Storage 12,000 af/mo System
Dry (4 years)	95,600	144,000
Wet (4 years)	45,000	68,400
Average (1951-1983)	81,000	119,000

The annual flows would change due to the variable demand. The demand on Dillon Reservoir is less during years of higher precipitation and greater during dry years. The range of estimated operation costs and energy costs for preliminary design of conveyance system components has been based on the average annual flows of Table 11.2. For overall project cost comparisons, the conveyance system flow and corresponding energy costs have been selected to meet the pumpback flow requirement of the particular replacement reservoir(s) alternative.

Electrical Power Requirements

Based on the above schedule of operation, pumping power demand for the conveyance system flow combinations would range from a high annual average of 30,300 kilowatts (kW) to a low average of 20,200 kW. The estimated billable energy demand and the energy consumption during an average year for each station for high and low flow combinations is presented in Table 11.3. Megawatt (equal to 1000 kW) hours per year are shown as MWh/yr.

TABLE 11.3

ESTIMATED BILLABLE ENERGY DEMAND AND AVERAGE ENERGY CONSUMPTION

High Flow Combination: 12,000 af/mo System, 147,000 af Storage, 119,000 af/yr

Pump Station		Demand kW	Energy MWh/yr
Green Mountain Dam		10,600	76,600
Slate Creek		9,750	70,500
Blue River		9,950	71,900
	Total	30,300	219,000

Low Flow Combination: 8,000 af/mo System, 95,000 af Storage, 81,000 af/yr

Pump Station		Demand kW	Energy MWh/yr
Green Mountain Dam		7,070	51,100
Slate Creek		6,500	47,000
Blue River		6,630	47,900
	Total	20,200	146,000

Source of Electrical Supply

The proposed Green Mountain Dam and Slate Creek Pump Station Sites lie within the present service boundaries of Mountain Parks Electric, Inc., a Rural Electric Administration utility. The southern station, Blue River Site, is within the present Public Service Company of Colorado service boundary. It may be desirable that one utility serve all conveyance system power needs in which case a "wheeling" agreement between the two utilities could be affected whereby the power of one utility would be carried on the transmission lines of another.

Adequate transmission line capacity exists at the adjacent hydroelectric plant for the proposed electrical load of the pump station at Green Mountain Dam. At the Slate Creek Pump Station, a 2-mile distribution line extension would apparently be necessary. Planning for the Hayden-Blue River transmission line project includes a new substation at a site about 2 miles to the east (DOE/WAPA, 1985). At the Blue River Pump Station Site, near the north boundary of the town of Silverthorne, a new substation would be required with the pump station to maintain line voltage for existing customers during pump motor starting.

The projected pumping power requirement of 20 to 30 MW can be compared with the 35 MW electrical power demand of the Windy Gap Project which recently (1985) began operation near Granby. It would constitute about 20 percent of the projected 1993 electrical power demand in the Middle Park Area (Gore Pass, Granby, Green Mountain) (DOE, 1985). Sufficient power source and transmission capacity would be available from the scheduled Hayden-Blue River transmission line project.

Electrical Power Cost

Mountain Parks Electric and Public Service Company each have separate rate schedules with different demand and energy charges. Public Service Company has both peak and off-peak demand charges as well as separate rate schedules for transmission and distribution voltage. In addition, monthly energy costs that are not recovered by the effective total energy rates are assessed as an electric cost adjustment. Each utility may also assess a line extension or facility charge. Evaluation of the charges for the average schedule of operation projected for the Conveyance System based on presently (1985-1986) filed rates indicated an average of $0.0425 per kWh.

The possibility was investigated of increasing off-peak pumping and decreasing on-peak pumping in order to balance the monthly demand charge for the two rates. It was determined that the estimated cost of construction of additional capacity to permit a higher pumping rate during off-peak hours exceeded by a considerable margin the estimated saving in peak hour electrical demand charges.

Curtailment of Green Mountain Hydropower Production

The present hydropower production at Green Mountain Dam would be curtailed due to reduced flow with the Exchange Project in operation. This is described in Chapter 12 of this report. Revenue lost from the curtailment of hydropower production would amount to $340,000 per year, based on current rates and historic power generation. For the purpose of project cost estimates this figure has been assessed against the pumpback conveyance system as a cost of operation.

Retrofitting Green Mountain Hydropower Plant

Even though the present turbines may not operate with the pumpback conveyance in operation, nevertheless, the projected flows and heads offer a significant hydropower potential. The existing power plant could be retrofitted with smaller turbines scaled to produce power from required releases throughout the year.

Analysis of this potential indicates that 6.2 million kWh could be produced per year. This power could be supplied to substitute for the purchase of commercial power. Economic analysis of this hydropower potential is contained in the hydropower evaluation section of Appendix B.

Operating Personnel Requirements

It is anticipated that one of the three pump stations would contain a control center. Conditions vital to the protection and maintenance of the equipment and equipment operational status would be electronically displayed in front of a full time operator at the control center. Sensors at each station and reservoir would monitor these conditions and transmit the data along a buried signal cable. A total of 12 employees was estimated including allowance for sick leave and vacations.

Equipment Repair and Replacement

The cost of repair and replacement of mechanical and electrical equipment is evaluated as the cost of materials and services beyond those which are provided by the full time operating and maintenance employees. These special materials and supplies would be purchased from the product distributor or manufacturer for the specific repair or replacement requirement. The labor would be provided by a repair contractor, a manufacturer's technician or a specialist on the staff of the owner.

11.7 SUMMARY OF ESTIMATED CONSTRUCTION AND OPERATION COSTS

A conveyance system for the 8000 af per month alternative would use 63-inch steel pipe and deliver water at a peak rate of 132 cfs. A conveyance system for the 12,000 af per month alternative would use 75-inch steel pipe and deliver at a peak rate of 199 cfs. Reconnaissance-level cost estimates have been developed for construction and operation of both conveyance system size alternatives. The detailed cost estimates and the basis of their derivation are described in Appendix B.

Construction Cost Estimates

Estimated construction quantities, prices and allowances have been combined to obtain a total construction cost. Costs are indexed to January 1986. Future inflation is not included. To determine the funding necessary to complete the project, additional elements of project administration and financing have been added to obtain the total investment cost. Estimated construction quantities and estimated costs are summarized in Tables 11.4 and 11.5.

Operation Costs Estimates

Operating personnel requirements, equipment repair and maintenance and average annual energy required is described in Section 11.5. Estimates of repair and replacement maintenance costs were derived using statistics of mechanical and electrical equipment operation experience as published in the "Hydroelectric Plant Construction Costs and Annual Production Expenses - 1981" (DOE/EIA, 1983). It is detailed in Appendix B. Electrical energy was evaluated using the average of Mountain Park Electric, Rate Code LP, effective March 22, 1985 and Public Service Company of Colorado, Rate Schedule PT, effective December 13, 1985.

TABLE 11.4

8000 af CONVEYANCE SYSTEM
SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	350	acres	$ 760,000
Pipeline			62,680,000
Excavation	1,048,000	cubic yards	
Backfill	526,000	cubic yards	
Pipe and installation	140,900	feet	
Reservoirs			920,000
Excavation	12,800	cubic yards	
Embankment	12,400	cubic yards	
Concrete	1,700	cubic yards	
Pump Stations			15,310,000
Excavation	17,500	cubic yards	
Concrete	8,500	cubic yards	
Number of Stations	3	each	
Pumps and Motors	12	each	
Surge Control			2,880,000
Surge Tanks	6	each	
Tank Buildings	3	each	
Concrete	1,270	cubic yards	
Surge Pipeline	2,180	feet	
Excavation	10,200	cubic yards	
Backfill	8,100	cubic yards	
Subtotal of Direct Costs			$ 82,550,000
Contingencies (25%)			20,640,000
TOTAL DIRECT COST			$ 103,190,000
Engineering, Legal, Administration (20%)			20,640,000
TOTAL CONSTRUCTION COST			$ 123,830,000
Interest During Construction (4 yrs.)			19,770,000
TOTAL CAPITAL COSTS			$ 143,600,000
Financing and Reserve			21,400,000
TOTAL INVESTMENT COST			$ 165,000,000

TABLE 11.5

12,000 af CONVEYANCE SYSTEM
SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITIES		ESTIMATED COSTS
Land Acquisition	350	acres	$ 760,000
Pipeline			77,020,000
Excavation	1,268,000	cubic yards	
Backfill	588,000	cubic yards	
Pipe and installation	140,900	feet	
Reservoirs			1,120,000
Excavation	12,800	cubic yards	
Embankment	12,400	cubic yards	
Concrete	1,710	cubic yards	
Pump Stations			16,620,000
Excavation	17,500	cubic yards	
Concrete	8,500	cubic yards	
Number of Stations	3	each	
Pumps and Motors	12	each	
Surge Control			3,110,000
Surge Tanks	6	each	
Tank Buildings	3	each	
Concrete	1,270	cubic yards	
Surge Pipeline	2,180	feet	
Excavation	10,200	cubic yards	
Backfill	8,100	cubic yards	
Subtotal of Direct Costs			$ 98,630,000
Contingencies (25%)			24,670,000
TOTAL DIRECT COST			$ 123,300,000
Engineering, Legal, Administration (20%)			24,700,000
TOTAL CONSTRUCTION COST			$ 148,000,000
Interest During Construction (4 yrs.)			24,700,000
TOTAL CAPITAL COSTS			$ 172,000,000
Financing and Reserve			26,000,000
TOTAL INVESTMENT COST			$ 198,000,000

TABLE 11.6

HYDROPOWER RETROFIT AT GREEN MOUNTAIN DAM
SUMMARY OF ESTIMATED CONSTRUCTION QUANTITIES AND COSTS

ITEM	ESTIMATED QUANTITY		ESTIMATED COSTS
Power Plant Modification			
Turbine Generator	2	each	$ 1,830,000
Structural, Mechanical			665,000
Electrical			375,000
Subtotal of Direct Costs			$ 2,870,000
Contingencies (25%)			720,000
TOTAL DIRECT COST			$ 3,590,000
Engineering, Legal, Administration (20%)			720,000
TOTAL CONSTRUCTION COST			$ 4,310,000
Interest During Construction (2 yrs.)			340,000
TOTAL CAPITAL COSTS			$ 4,650,000
Financing and Reserve			650,000
TOTAL INVESTMENT COST			$ 5,300,000

TABLE 11.7

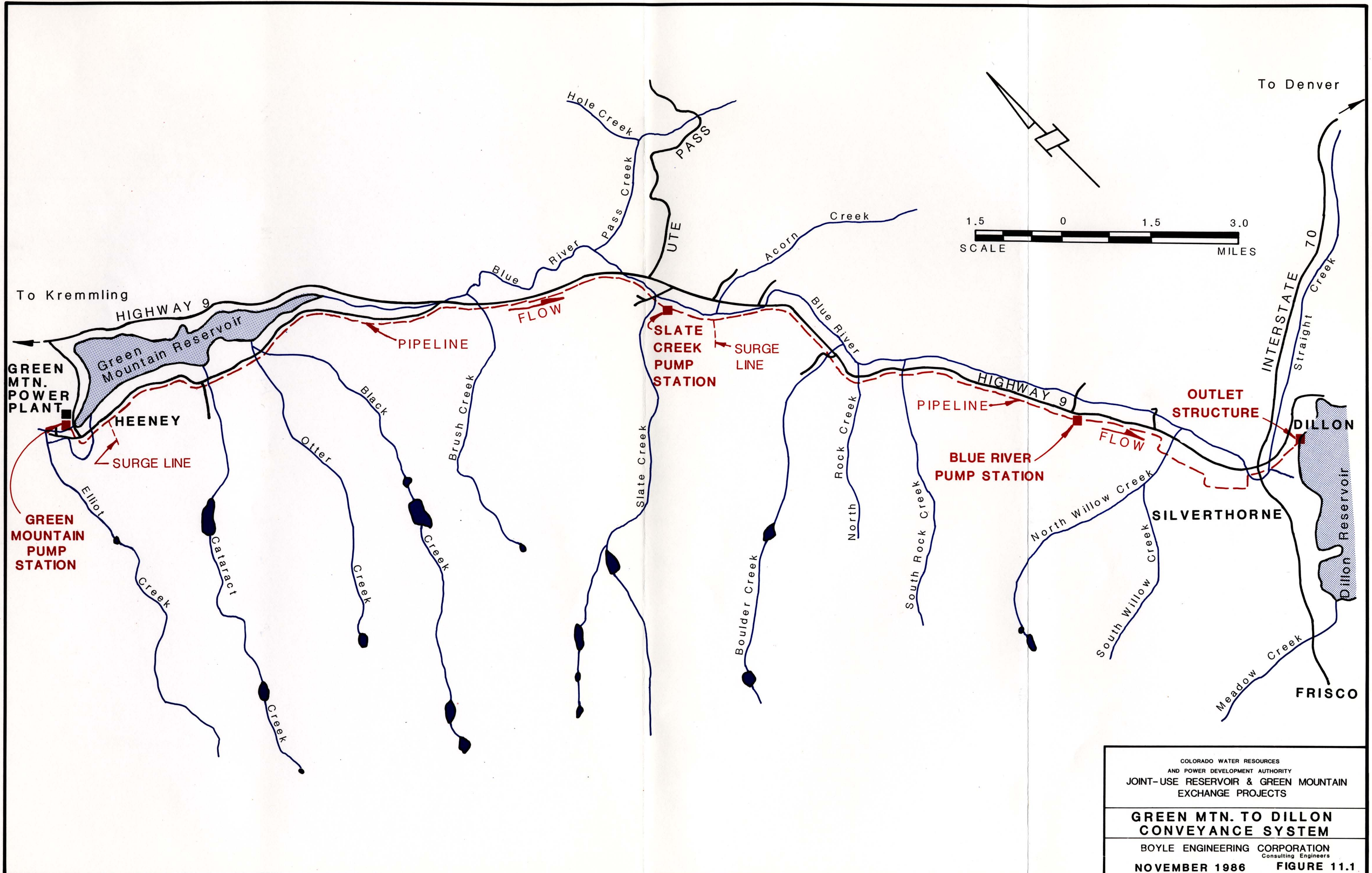
8000 af CONVEYANCE SYSTEM
SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Operating Labor and Maintenance		$ 700,000
Operators, Station Labor	$ 442,000	
Equipment Repair	136,000	
Pipeline Maintenance	125,000	
Pumping Energy		6,260,000
Green Mtn. Hydropower Replacement		340,000
Total Estimated Annual Operation Cost		$ 7,300,000

TABLE 11.8

12,000 af CONVEYANCE SYSTEM
SUMMARY OF ESTIMATED ANNUAL OPERATION COSTS

Operating Labor and Maintenance		$ 800,000
Operators, Station Labor	$ 442,000	
Equipment Repair	184,000	
Pipeline Maintenance	174,000	
Pumping Energy		9,460,000
Green Mtn. Hydropower Replacement		340,000
Total Estimated Annual Operation Cost		$ 10,600,000



COLORADO WATER RESOURCES AND POWER DEVELOPMENT AUTHORITY

JOINT-USE RESERVOIR & GREEN MOUNTAIN EXCHANGE PROJECTS

GREEN MTN. TO DILLON CONVEYANCE SYSTEM

BOYLE ENGINEERING CORPORATION
Consulting Engineers

NOVEMBER 1986 — FIGURE 11.1

12.0 HYDROPOWER EVALUATION

The combined impact of the pumpback conveyance system operation and the increased storage retained in Dillon Reservoir due to the Green Mountain Exchange Project, reduces the hydropower generation potential at Green Mountain Power Plant. A revenue loss has been evaluated and debited to the conveyance system.

Potential hydropower installations have been evaluated at Red Mountain Dam, Azure Dam and Una Dam. A hydropower retrofit installation has been evaluated at Green Mountain Dam.

12.1 PROJECT IMPACT ON GREEN MOUNTAIN HYDROPOWER

Projected monthly releases from Green Mountain Reservoir with the pumpback conveyance system in operation are generally less than the minimum operating rate of flow of the installed turbines. Curtailment of power production from these turbines would mean a loss of revenue. To some extent this apparent revenue loss would be offset by a reduction in operating costs. For example, the dam and reservoir could be maintained by fewer personnel if an operator were not required at the power plant.

Revenue From Current Power Sales

For the cost estimate of conveyance system operation the apparent loss of revenue has been evaluated. Average annual production over the last 10 years (1976-1985) has been 58.3 million kWh. This power has been sold to the Western Area Power Administration (WAPA). The 1985 revision to the power sale agreement with WAPA provides for a capacity payment of $1.65 per kW per month and an energy payment $0.0051 per kWh (USBR, 1986).

Revenue from the sale of this power would amount to $340,000 per year, based on current rates and historic power generation. For the purpose of project cost estimates this figure has been assessed against the pumpback conveyance system as a cost of operation.

Remaining Hydropower Generation at Green Mountain Dam

Projected flows and heads indicate the existing power plant could be retrofitted with smaller turbines scaled to produce power from required releases throughout the year. Maximum power production was estimated as 5700 kW with an average of 6.2 million kWh per year. An economic analysis is contained in the hydropower evaluation section of Appendix B.

12.2 HYDROPOWER POTENTIAL OF PROJECT RESERVOIRS

Hydropower could be generated from flows which would be released or which would spill at Red Mountain Dam, Azure Dam, Una Dam and Green Mountain Dam. The possible installation of hydropower facilities at each of these dams has been evaluated on a marginal cost basis. This cost has not been included in the cost of each project as a joint-use or replacement reservoir. Instead each hydropower facility has been treated as an optional feature for which an independent financial rate of return has been estimated.

Flows and hydraulic heads available to the turbines was estimated with a reservoir operation simulation model. In this study the reservoirs were operated solely to meet the replacement requirements of the Green Mountain Exchange Project. No modification or special operation was simulated to enhance hydropower generation. Maximum heads for hydropower generation at the proposed sites range from 58 to 252 feet. Typical yearly peak monthly flows range from 400 to 800 cfs with the flow at the Una Site exceeding 4400 cfs.

Performance curves of several turbines with different head and flow characteristics were checked for total power production over a simulated operation period. With the TURB6 computer program of Boyle, 30 years of reservoir operation data was input from prior hydrologic analyses. For each site trial turbine selections and performance curves were input, the power production was calculated and cash flow analysis developed. The turbine characteristics and cash flow details are in Appendix B. A summary of hydropower plant capital and operating costs from Chapters 7, 8, 10 and 11 of this report is presented in Table 12.1.

Capital Cost of Hydropower Plants

Costs of turbines, generators, switchgear and of the other principal mechanical components were determined based on the suggested unit price of the manufacturer with installation added. Switchyard, station electrical and transmission line costs were estimated by adjusting prior construction contract prices to the scale of these installations. Penstock, transmission line and structure unit costs are similar to the river diversion pump station estimates of the Wolford A' and Wolcott sites.

Operating Cost of Hydropower Plants

The budget allocation for operating personnel would be influenced by the magnitude of power plant investment. Full time operators and maintenance personnel would be employed at larger plants and the smaller plants would be managed with daily inspection visits and perhaps remote electronic monitoring.

Estimates of repair and replacement maintenance costs were derived using statistics of mechanical and electrical equipment operation experience as published in the "Hydroelectric Plant Construction Cost and Annual Production Expenses - 1981" (DOE/EIA, 1983).

Cost of Potential Power Production

Construction and operating costs have been projected for 30-years along with the power production each year. With this annual cost, the cost per kWh was calculated to produce an 8 percent return. No market study has been conducted, however, potential revenue was based on filed rates of the Public Service Company of Colorado. Table 12.2 is a summary of the projected average annual energy production and the cost per kWh.

TABLE 12.1

SUMMARY OF ESTIMATED HYDROPOWER PLANT COSTS

PLANT	ANNUAL PRODUCTION (GWh/yr)	DIRECT CONSTRUCTION COST	TOTAL INVESTMENT COST	ANNUAL OPERATION & MAINT. COST
Red Mountain	3.4	$ 3,240,000	$ 4,785,000	$ 82,000
Azure	36.2	11,980,000	18,970,000	200,000
Una	88.5	14,430,000	22,885,000	320,000
Green Mountain	6.2	3,590,000	5,300,000	85,000

TABLE 12.2

PROJECTED HYDROPOWER GENERATION
AND ESTIMATED PRODUCTION COST

SITES	CAPACITY (kW)	ANNUAL GENERATION (GWh)	PRODUCTION COST ($/kWh)
Red Mountain	1300	3.4	0.111
Azure	7000	36.2	0.036
Una	17,600	88.5	0.019
Green Mountain	3000	6.2	0.057

Note: Units shown are kilowatt (kW) and gigawatt (GWh), (1 GWh = 1,000,000 kWh).

13.0 ALTERNATIVE COMPARISONS

Single reservoirs of this Study were analyzed for their ability to meet Joint-Use Reservoir objectives. These single reservoirs and combinations of reservoirs were also analyzed for their ability to supply the greater yield objectives of the Green Mountain Exchange Project based upon the yield of Green Mountain Reservoir. A reservoir could initially serve as a Joint-Use Reservoir and later, in combination with additional reservoir(s) also serve as a Replacement Reservoir for the Exchange Project.

13.1 JOINT-USE RESERVOIRS

Four reservoirs are within range of the 30,000 af annual yield objective of a Joint-Use Reservoir. They are shown in Table 13.1. The annual yield is based on the High Future-Use scenario as defined in paragraph 3.3 and on the assumption that the pumpback system would not be in operation.

TABLE 13.1

JOINT-USE RESERVOIR ALTERNATIVES

PROJECT	CAPACITY (af)	YIELD [1] (af/yr)
Wolford Mountain C	60,000	25,000
Wolford Mountain A'	120,000	40,000
w/Colo. diversion [2]	120,000	49,000
Azure	85,000	48,000
Red Mountain	140,000	56,000

1) Based on High Future-Use Scenario, without pumpback to Dillon Reservoir from Green Mountain Reservoir.

2) Wolford Mountain A' would receive inflow only from Muddy Creek, or alternatively, supplement Muddy Creek storage with water diverted and pumped from the Colorado River.

13.2 REPLACEMENT CAPABILITY OF THE PROJECT RESERVOIRS

Reservoir operation analyses were performed to evaluate the capabilities of each single reservoir and of representative combinations of reservoirs to serve as Replacement Reservoirs for the Green Mountain Exchange Project. For preliminary evaluation it was assumed that the Replacement Reservoir firm yield should approximate the estimated firm yield from the Green Mountain Reservoir storage pool that is considered to be available for exchange. In addition, it was assumed that a Replacement Reservoir(s) would provide the replacement water for CBT out-of-priority diversions and to supplement natural flows thereby meeting in-basin irrigation and municipal demands. It (they) would also meet USBR water sales requirements that could otherwise have been supplied by Green Mountain Reservoir storage.

For Replacement Reservoir alternatives, analyses were made of Wolcott Reservoir and combinations of reservoirs including Wolcott and Red Mountain, Wolcott and Azure, Red Mountain and Una Reservoirs, and Una and Wolford Mountain A'. They are shown in Table 13.2. Wolcott Reservoir when combined with another reservoir, includes diversions from the Eagle River only.

TABLE 13.2
REPLACEMENT RESERVOIR ALTERNATIVES FOR
GREEN MOUNTAIN EXCHANGE PROJECT

RESERVOIR(S) [1]	CAPACITY (af)	YIELD [2] (af/yr)
Wolcott w/diversion from:		
Eagle & Colo.	350,000	135,000
Eagle only	160,000	65,000
Red Mountain & Wolcott w/Eagle diversion	300,000	89,000
Azure & Wolcott w/Eagle diversion	245,000	114,000
Red Mountain & Una	290,000	159,000
Una & Wolford Mountain A' w/Colo. diversion	270,000	148,000

1) Diversion involves pumping from Colorado R. or Eagle R. or both rivers, as cited.

2) Based on High Future-Use Scenario with pumpback to Dillon Reservoir from available storage in Green Mountain Reservoir.

TABLE 13.3

GREEN MOUNTAIN EXCHANGE PROJECT
YIELD WITH ALTERNATIVE REPLACEMENT RESERVOIR(S)

REPLACEMENT 1) RESERVOIRS	ESTIMATED RESERVOIR FIRM YIELD (1000 af/yr)	GREEN MOUNTAIN 2) RESERVOIR EFFECTIVE STORAGE AVAILABLE FOR EXCHANGE (1000 af)	ANNUAL EXCHANGE PROJECT YIELD 3)	
			AVERAGE CONVEYANCE 4) SYSTEM FLOW (1000 af/yr)	INCREASED YIELD TO ROBERTS TUNNEL (1000 af/yr)
Wolcott w/Eagle & Colo. diversion	135	138	113	119
Red Mountain & Una	159	147	119	124
Una & Wolford A' w/Colo. diversion	148	147	119	124
Azure & Wolcott w/Eagle diversion	114	117	87	101
Red Mountain & Wolcott w/Eagle diversion	89	92	81	93

1) Diversion involves pumping from Colorado R. or Eagle R. or both rivers, as cited.

2) The portion of the total of 147,000 af of Green Mountain Reservoir effective storage made available for exchange was selected to provide the same amount of firm yield as that expected from the Replacement Reservoir(s).

3) Project yield includes conveyance flow and additional flows retained in Dillon Reservoir. See Table 4.6, Chapter 4.

4) The 8000 af/mo conveyance system was combined with the Azure-Wolcott and Red Mountain-Wolcott Replacement Reservoirs. The 12,000 af/mo system was combined with the Wolcott/Eagle-Colo., Red Mountain-Una and Una-Wolford A' Reservoirs.

The combination of Red Mountain and Una Reservoirs was analyzed with the assumption that Red Mountain Reservoir would release water to make up natural water shortages for in-basin uses (with water rights perfected by use prior to October 16, 1977), CBT replacement requirements and water sales above Dotsero. This supplemental water would relieve these rights and water sales from the Shoshone call. Una Reservoir would replace the supplemental water diverted for rights and water sales downstream of Dotsero. It would also satisfy the Cameo demands. It has been estimated that either Red Mountain Reservoir or Wolford Mountain A' (supplemented with diversions pumped from the Colorado River) could meet the variable demands above Dotsero as determined by the hydrologic simulation model.

Reservoir operation analyses indicated that two of the reservoir combinations (Red Mountain and Una, and Una and Wolford Mountain A') could provide sufficient firm yield to replace the entire Green Mountain storage (the total effective storage is approximately 147,000 af with a firm yield capability of 144,000 af/yr). This is shown in Table 13.3. Other reservoir combinations can only partially replace the firm yield capability of Green Mountain Reservoir. In these cases, it was assumed that only a portion of of the capacity of Green Mountain Reservoir could be made available to the Exchange Project. The remaining capacity, combined with the Replacement Reservoir(s) would then serve as replacement for the firm yield capability of Green Mountain Reservoir. For example, Wolcott Reservoir (with diversion by pumping from both the Eagle and Colorado Rivers) can only provide the firm annual yield of 135,000 af, as opposed to the 144,000 af yield of Green Mountain Reservoir. Thus, effective storage available to the Exchange Project from Green Mountain Reservoir was assumed to be 138,000 af. This storage would yield approximately 135,000 af/yr, which would be equivalent to Wolcott Reservoir.

Also noted in Table 13.3 is the average annual pumpback or conveyance system flow quantity and increased yield to Roberts Tunnel. These were derived by interpolating figures presented in Table 4.6, Chapter 4. "Increased Yield to Roberts Tunnel" refers to the increase in the average annual transmountain diversions of the Metropolitan Denver Water Supply System provided by the Exchange Project concept with alternative Replacement Reservoir(s).

TABLE 13.4

SUMMARY OF ESTIMATED INVESTMENT COSTS

FEATURE [1]	CAPACITY (af)	TOTAL [2] CONSTRUCTION COST	INTEREST DURING CONSTRUCTION	RESERVE AND FINANCING	TOTAL INVESTMENT COST
Wolford Mountain A'	120,000	$ 63,600,000	$ 5,100,000	$ 9,300,000	$ 78,000,000
w/Colo. diversion	120,000	89,300,000	7,100,000	13,600,000	110,000,000
Wolford Mountain C	60,000	35,200,000	2,800,000	5,000,000	43,000,000
Red Mountain	140,000	124,000,000	15,000,000	19,000,000	158,000,000
Azure	85,000	98,000,000	23,600,000	16,400,000	138,000,000
Wolcott w/diversion from:					
Eagle & Colo.	350,000	421,000,000	118,000,000	76,000,000	615,000,000
Eagle only	160,000	190,600,000	30,400,000	31,000,000	252,000,000
Una	150,000	220,000,000	36,000,000	35,000,000	291,000,000
Conveyance System					
12,000 af/mo	-	148,000,000	24,000,000	26,000,000	198,000,000
8000 af/mo	-	124,000,000	20,000,000	21,500,000	165,000,000

1) Diversion involves pumping from the Colorado R. or Eagle R. or both rivers, as cited.

2) Construction costs are indexed to January, 1986.

13.3 CAPITAL AND TOTAL INVESTMENT COSTS

From the preliminary design drawings, measurements have been made to estimate construction quantities. These quantities were grouped into approximately 40 different categories for which unit and lump sum construction prices were estimated. A contingency of 25 percent was added to the total of estimated construction quantities and prices. This is considered appropriate for a reconnaissance-level study. An additional allowance of 20 percent provides for engineering, legal and administrative expenses. With the addition of interest during construction, the sum is Total Capital Cost.

To determine the Total Investment Cost, estimated financing expenses and a reserve fund were added to the total capital cost. The reserve fund was evaluated as the annual principal and interest payment on 30-year, 8 percent financing of the total investment cost. The financing expenses were estimated as 3 percent of the total investment cost. Table 13.4 summarizes the cost elements comprising the Total Investment Cost.

13.4 YEARLY DEBT SERVICE AND OPERATING COSTS

Comparison of alternative reservoirs and combinations of reservoirs for this Study is based on estimates of debt service and average annual operating costs. The costs are comprised of the yearly principal and interest payment (debt service) on assumed 30-year, 8 percent financing of the total investment cost and the estimated average annual cost of pumping power, operating personnel, maintenance and repair costs. No price escalation was incorporated. A summary of yearly debt service and operating costs is presented in Table 13.5.

Appendix B contains an annualized cost analysis of the alternative reservoirs presented in a manner similar to the cost analyses of the Metropolitan Denver Water Supply EIS (COE, 1986). The annualized cost analysis provided results similar to those obtained with this analyses of yearly debt service and operating costs.

13.5 COST AND YIELD COMPARISONS

The average cost of each acre-foot of water was derived from the yearly debt service and operating costs analysis and the yield expressed in acre-feet per year. For alternative Joint-Use Reservoirs and Replacement Reservoirs for the Green Mountain Exchange Project, the cost per acre-foot of estimated firm annual yield of water is shown in Tables 13.6 and 13.7, respectively. Operating costs were estimated on an average annual basis.

TABLE 13.5

SUMMARY OF YEARLY DEBT SERVICE AND AVERAGE ANNUAL OPERATING COSTS

FEATURE		DEBT SERVICE 1)	POWER COST	OTHER COSTS 3)	TOTAL COST PER YEAR
RESERVOIR	CAPACITY (af)				
Wolford Mountain A'	120,000	$ 6,960,000	-	$ 110,000	$ 7,070,000
w/Colo. diversion	120,000	9,770,000	$ 410,000	240,000	10,420,000
Wolford Mountain C	60,000	3,820,000	-	90,000	3,910,000
Red Mountain	140,000	14,000,000	-	150,000	14,150,000
Azure	85,000	12,260,000	-	80,000	12,340,000
Wolcott w/diversion from:					
Eagle & Colo.	350,000	54,650,000	10,750,000	700,000	66,100,000
Eagle only	160,000	22,400,000	3,700,000	400,000	26,500,000
Una	150,000	25,850,000	-	200,000	26,050,000
CONVEYANCE SYSTEM	FLOW RATE af/yr				
12,000 af/mo	119,000	17,600,000	9,800,000 2)	800,000	28,200,000
	113,000	17,600,000	9,300,000 2)	800,000	27,700,000
8000 af/mo	87,000	14,700,000	7,100,000 2)	700,000	22,500,000
	81,000	14,700,000	6,600,000 2)	700,000	22,000,000

1) Annual principal and interest payment on 30-year, 8 percent financing in the amount of the total investment cost. Construction costs are indexed to January, 1986.

2) Compensation of $340,000 per year for Green Mountain hydropower replacement included in conveyance power costs.

3) Operating and maintenance labor, supplies and repair costs.

TABLE 13.6

ESTIMATED COST OF JOINT-USE RESERVOIR YIELD

RESERVOIR	CAPACITY (af)	FIRM YIELD 1) (af/yr)	TOTAL COST 2) PER YEAR ($)	UNIT COST 3) OF YIELD ($/af)
Wolford Mountain A'	120,000	40,000	7,070,000	180
w/Colo. diversion	120,000	49,000	10,420,000	210
Wolford Mountain C	60,000	25,000	3,910,000	160
Red Mountain	140,000	56,000	14,150,000	250
Azure	85,000	48,000	12,340,000	260

1) Based on High Future-Use scenario, without pumpback to Dillon Reservoir from Green Mountain Reservoir.

2) Total cost per year is the sum of yearly debt service, average annual power costs for the alternative pumped Colorado River flows to supplement Muddy Creek flows at Wolford Mountain A', and other operating costs from Table 13.5. Construction costs are indexed to January, 1986.

3) Estimated cost per acre-foot of firm reservoir yield during a year of average operating costs.

TABLE 13.7

ESTIMATED COST OF REPLACEMENT RESERVOIR YIELD

RESERVOIR(S) [1]	CAPACITY (af)	FIRM YIELD [2] (af/yr)	TOTAL COST [3] PER YEAR ($)	UNIT COST [4] OF YIELD ($/af)
Wolcott w/diversion from:				
Eagle & Colo.	350,000	135,000	66,100,000	490
Eagle only	160,000	65,000	26,500,000	410
Red Mountain & Wolcott w/Eagle diversion	300,000	89,000	40,650,000	460
Azure & Wolcott w/Eagle diversion	245,000	114,000	38,840,000	340
Red Mountain & Una	290,000	144,000 [5]	40,200,000	280
Una & Wolford Mountain A' w/Colo. diversion	270,000	144,000 [5]	36,470,000	250

1) Diversion involves pumping from Colorado R. or Eagle R. or both rivers, as cited.

2) Based on High Future-Use scenario with pumpback to Dillon Reservoir from available storage in Green Mountain Reservoir.

3) Total cost per year is the sum of yearly debt service, average annual power costs for pumping supplementary Colorado River flows at Wolford Mountain A' and Eagle and Colorado or alternatively Eagle River only flows at Wolcott, and other operating costs from Table 13.5.

4) Estimated cost per acre-foot of firm reservoir yield during a year of average operating costs. Construction costs indexed to January 1986.

5) Yield useful for exchange has been limited to the 144,000 af estimated annual yield of Green Mountain Reservoir.

The unit cost of water delivered by the conveyance system is shown in Table 13.8. Both the conveyance system cost and the costs of selected Replacement Reservoirs must be combined to indicate the total cost of additional water made available at Dillon Reservoir. This is presented in Table 13.9. The annual costs from Tables 13.7 and 13.8 are combined on Table 13.9. The Unit Cost of Water in Table 13.9 is the average Total Cost Per Year divided by the Exchange Project Yield. It provides a relative cost comparison of an acre-foot of water among the Exchange Project alternatives of this Study.

13.6 PROJECT DEVELOPMENT

This report summarizes the results of a 21-month study which has provided reconnaissance-level engineering and hydrology information on two conceptual projects: Joint-Use Reservoir and the Green Mountain Exchange. For the projects designated, hydrological, geotechnical and preliminary engineering design studies have been carried out which permit comparisons among alternatives based on an approximately uniform level of investigation. The development schedule for each of the alternatives addressed in this Study would require a series of additional steps including selection of preferred alternatives, feasibility and site-specific environmental studies, regulatory compliance, financing, design and construction and definition of institutional arrangements for project implementation. Neither the Colorado River Water Conservation District nor the Denver Water Board has made any decision with respect to the future of these projects.

A minimum of 6 additional years from the decision to proceed would be a reasonable projection of the time needed before any of the Joint-Use Reservoir Projects would be completed. A minimum of 14 years is a reasonable projection for any of the Green Mountain Exchange Project alternatives. Recognizing that this Study has covered only a limited number of the facets involved in selection of projects for construction, no ranking or preference has been made. In accordance with the scope of work, water yields and estimated costs for alternatives have been derived and presented for consideration in the next level of implementation of these projects.

TABLE 13.8

ESTIMATED COST OF CONVEYANCE SYSTEM WATER TRANSFER

FEATURE	AVERAGE ANNUAL FLOW (af/yr)	TOTAL [1)] COST PER YEAR ($)	AVERAGE [2)] UNIT COST ($/af)
Conveyance System			
12,000 af/mo	119,000	28,200,000	240
	113,000	27,700,000	250
8000 af/mo	87,000	22,500,000	260
	81,000	22,000,000	270

1) Total cost per year is the sum of yearly debt service, average annual power costs for pumping from Green Mountain Reservoir to Dillon Reservoir, Green Mountain hydropower revenue replacement, and other operating costs from Table 13.5. No Replacement Reservoir costs are included. Construction costs indexed to January, 1986.

2) Estimated cost per acre-foot of water conveyed during a year of average flow and average costs. To serve as an exchange project, Replacement Reservoirs must also be included. See Table 13.9.

TABLE 13.9

COST OF EXCHANGE PROJECT WATER
AVAILABLE TO ROBERTS TUNNEL

PROJECT [1] REPLACEMENT RESERVOIR(S)	AVERAGE [2] CONVEYANCE SYSTEM FLOW (1000 af/yr)	EXCHANGE PROJECT YIELD (1000 af/yr)	TOTAL [3] COST PER YEAR ($)	UNIT [4] COST OF WATER ($/af)
Wolcott w/diversion from Eagle & Colo.	113	119	93,800,000	790
Red Mountain & Una	119	124	68,400,000	550
Una & Wolford Mountain A' w/Colo. diversion	119	124	64,670,000	520
Azure & Wolcott w/Eagle diversion	87	101	61,340,000	610
Red Mountain & Wolcott w/Eagle diversion	81	93	62,650,000	670

1) Diversion involves pumping from Colorado R. or Eagle R. or both rivers, as cited.

2) The 8000 af/mo conveyance system was combined with the Azure-Wolcott and Red Mountain-Wolcott replacement reservoirs. The 12,000 af/mo system was combined with the Wolcott/Eagle-Colo., Red Mountain-Una and Una-Wolford A' replacements reservoirs.

3) Total cost per year is the sum of yearly debt service and operating costs from Table 13.7 for the listed reservoir(s) and from Table 13.8 for the corresponding conveyance system. Construction costs are indexed to January, 1986.

4) Estimated cost per acre-foot of project firm yield in a year of average operating costs.

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