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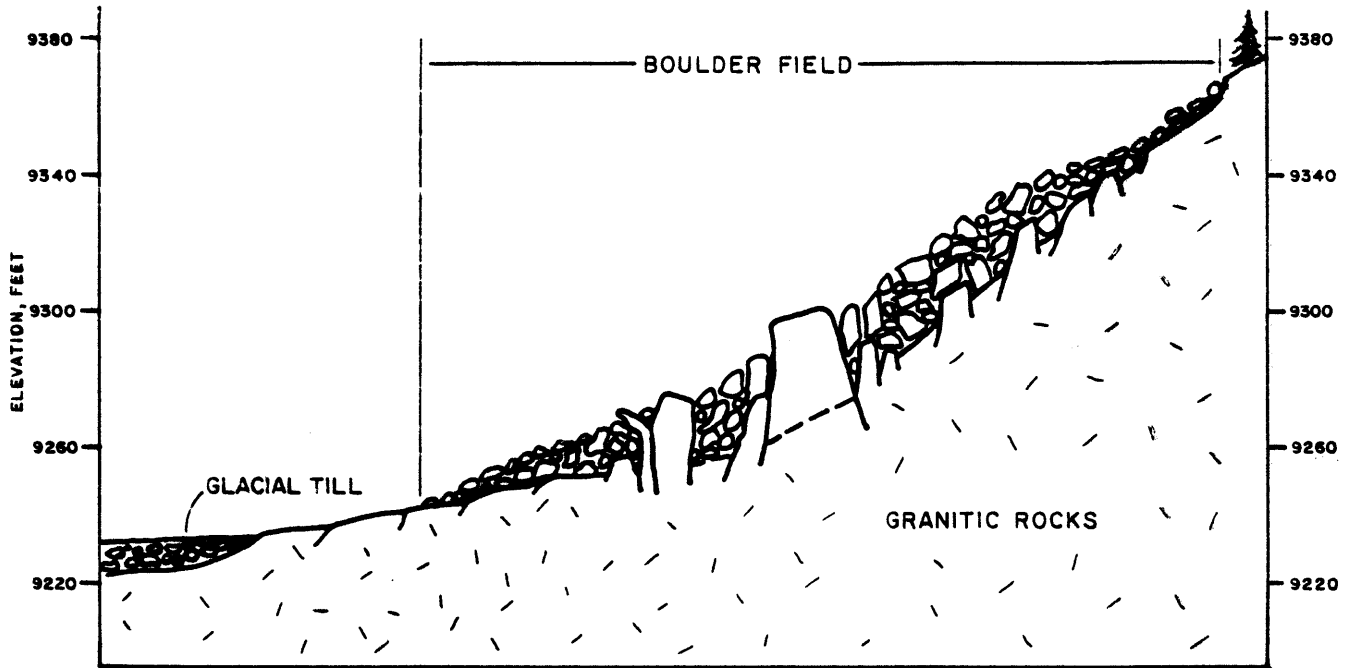
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STATE OF WYOMING WATER DEVELOPMENT COMMISSION



Contract No. 8-05846

SHELL VALLEY WATERSHED Level III

EXECUTIVE SUMMARY

PHASE I: Interim Report on Conceptual Design Lake Adelaide Enlargement

Prepared by:
ESA Geotechnical Consultants
Fort Collins, CO
January, 1986

SHELL VALLEY WATERSHED
Level III
Executive Summary

Phase I: Interim Report on Conceptual Design -
Lake Adelaide Enlargement

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1. INTRODUCTION AND BACKGROUND

Shell Valley is located in semi-arid north-central Wyoming at the western base of the Big Horn Mountains. The Shell Valley Watershed Improvement District (SVWID) was formed in 1981 to bring together various water users in Shell Valley. The SVWID originated out of a need to address additional irrigation water demands in certain parts of the valley. Preliminary discussions and studies indicated that water supplies could be increased by enlarging Lake Adelaide, which is located approximately 40 miles east of Greybull, high in the Big Horn Mountains (Figure 1). Lending impetus to this decision was the fact that the existing Lake Adelaide Dam was in need of rehabilitation to meet current dam safety requirements.

A Level II study was performed in 1984. The study focused on water usage in the Shell Valley Watershed, and on the feasibility and benefits of the proposed reservoir enlargement. In addition, a preliminary geotechnical and hydrological investigation was performed, and a preliminary conceptual design for raising the existing embankment dam was developed. Some of the Level II recommendations, regarding the size and design of the enlarged embankment and reservoir, have been superseded by the Level III study findings and recommendations. The principal findings of the Level II study were as follows:

1. The proposed enlargement of Lake Adelaide was justified and economically feasible on the basis of water demands in Shell Valley.
2. The dam was sized to provide a reservoir storage capacity of 3500 acre-feet, as compared with the present capacity of approximately 1700 acre-feet. This storage increase would require enlarging the embankment dam from the current crest elevation of 9260 feet to a crest elevation of 9280 feet.
3. The preliminary conceptual design for the enlarged dam and appurtenant facilities included the following important features:

- The raised embankment would be constructed directly onto the downstream face of the existing embankment, thus incorporating the existing dam as an integral part of the new structure.
- The alignment of the raised dam would follow the alignment of the existing dam; that is, the right abutment would be "bent" upstream to avoid a boulder field located on the valley slope just downstream of the right abutment of the existing dam.
- A fully penetrating cutoff trench or slurry trench was deemed necessary to prevent excessive seepage losses thought to be occurring through the foundation soil deposits underlying the dam.
- The outlet works were to be rehabilitated by jacking a cement-mortar lined steel pipe into the existing outlet conduit. This pipe would tie into a Morning Glory (drop-inlet) service spillway.
- An auxiliary, or emergency spillway was to be excavated in the borrow area south of the dam.

Several important changes have been made to the Level II conceptual design for the enlargement of Adelaide Dam and Reservoir as a result of the current Level III study results. Based on the Level III study, the following conclusions have been reached:

1. Adelaide Dam should be raised to a crest elevation of 9287 feet. Raising the embankment to this height will allow development of the optimum reservoir storage capacity (approximately 4550 acre-feet), and will provide the most economical results in terms of costs per acre-foot of storage increase and in terms of flood routing requirements.

2. The raised embankment should be constructed just downstream of the existing dam, with a straight axis alignment. This configuration will require that large boulders be removed from the right abutment area. However, the excavated boulders will be used to construct a downstream rock toe on the embankment, providing a cost savings in terms of construction materials in addition to providing a more stable dam.
3. Flood routing may be best handled by a combination of a side channel service spillway constructed across the left abutment of the dam, with an emergency spillway located in the borrow area. The side channel service spillway would handle all but very rare flood events.
4. The outlet works should be a newly constructed cut and cover outlet pipe. A totally new outlet works is the only viable option if the reservoir is to remain in operation during construction.
5. It will be necessary to significantly improve the existing access road into Adelaide Lake prior to construction. It would be advantageous to bid out the road improvements as a separate contract and construct the access road during the year prior to building the dam.
6. The estimated total construction cost, escalated to 1987 dollars, for the recommended design Alternative B is approximately \$2.142 million. The average cost of the project is \$755.00 per acre-foot of storage increase.

These design recommendations are discussed in detail in the Level III report. The following section summarizes the Level III, Phase I study findings.

II. SUMMARY OF LEVEL III, PHASE I STUDY

A. PURPOSE AND SCOPE

The Wyoming Water Development Commission (WWDC) contracted with ESA Geotechnical Consultants (ESA) in June, 1985, to perform Level III, Phase I design studies for the enlargement of Adelaide Dam and Reservoir. The primary Phase I tasks were to conduct a thorough investigation of the geotechnical and hydrological conditions at the Lake Adelaide site, develop a final conceptual design, and prepare comprehensive cost estimates for the dam and reservoir enlargement.

A substantial field exploration and testing program was conducted to supplement the data obtained in the Level II effort, and address the critical items identified in that earlier investigation. As a result of the Level III field investigation, major changes were made in the general conceptual design of the embankment, and in the recommendations for the foundation treatment. In addition, a detailed hydrological study of the Adelaide drainage area and the adjacent Buckley Creek drainage area was performed. The purpose of the hydrological study was to quantify the potential reservoir yield, using appropriate methods of analysis. The results of the hydrological study indicated that a substantial increase in reservoir yield was feasible, and could be obtained by increasing the height of the embankment to impound the maximum volume of water that Adelaide Valley could contain topographically.

Thus the scope of the Level III, Phase I effort changed with the accumulation of information from the geotechnical and hydrological investigations. Several conceptual designs were developed during the course of the investigation. A final design configuration was selected which will provide the optimum reservoir yield, allow safe flood routing, and make the best use of available construction materials.

B. GEOTECHNICAL INVESTIGATIONS AND CONCLUSIONS

Table I lists the critical geotechnical considerations investigated in the Phase I effort, and summarizes the results and conclusions of the geotechnical analyses. The most important conclusions derived from the comprehensive geotechnical study include the following.

Table 1
SUMMARY OF GEOTECHNICAL INVESTIGATIONS, RESULTS AND CONCLUSIONS

ITEM	WORK PERFORMED	RESULTS AND CONCLUSIONS
1. Overall Investigation of Dam Site	<ul style="list-style-type: none"> ● Drilled 4 holes - 2 in main channel for pump tests and 2 on right abutment to evaluate boulder pile. ● Excavated 6 backhoe trenches on left abutment and in main channel. ● Conducted seismic surveys to delineate soil layers and bedrock profile. ● Inspected existing dam. ● Reviewed Level II logs and data. 	<ul style="list-style-type: none"> ● Thin till deposits on left abutment should be excavated to firm bedrock to prevent seepage problems and to facilitate construction of a side channel spillway. ● Due to safety considerations, the need to maintain reservoir operations during construction, and final structural stability, THE NEW RAISED EMBANKMENT SHOULD BE CONSTRUCTED JUST DOWNSTREAM FROM THE EXISTING DAM, RATHER THAN INCORPORATING THE OLD DAM AS AN INTEGRAL PART OF THE NEW STRUCTURE.
(a) Seepage through Dam Foundation Soil Deposits	<ul style="list-style-type: none"> ● Multiple well pumping tests. ● Laboratory permeability tests ● Finite element method computer modelling. 	<ul style="list-style-type: none"> ● Effective permeability in foundation soils is relatively low. Therefore, foundation seepage losses should be tolerable ● AN EXPENSIVE AND DIFFICULT TO CONSTRUCT SEEPAGE CUTOFF TRENCH IS NOT JUSTIFIED.
(b) Boulder Pile on Right Abutment	<ul style="list-style-type: none"> ● Constructed a drill pad high up on boulder pile and drilled through boulders into bedrock. Also drilled at base of slope. ● Conducted detailed geologic survey of boulder pile and surrounding features, using hand level, tape and aerial surveys. 	<ul style="list-style-type: none"> ● The boulder field consists of 20 to 25 feet of rubble underlain by competent, very slightly weathered granitic bedrock. The boulders were formed in place by glacial unloading and freeze-thaw action. ● BOULDERS SHOULD BE REMOVED FROM THE RIGHT ABUTMENT CONTACT WITH THE EMBANKMENT AND USED TO BUILD A DOWNSTREAM ROCKFILL TOE.
(c) Construction Materials - Design Analyses	<ul style="list-style-type: none"> ● Performed comprehensive laboratory testing program to characterize material properties. ● Designed filters and drains based on gradation tests. ● Performed slope stability analyses. 	<ul style="list-style-type: none"> ● There is a sufficient amount of high quality construction material in the original borrow area to build the new dam. ● There exists up to 25 percent by volume of rocks and boulders larger than 6 inches in size in the borrow materials. OVER-SIZE ROCKS AND BOULDERS EXCAVATED FROM THE BORROW SOURCE SHOULD BE USED IN THE DOWNSTREAM ROCKFILL SECTION OF THE EMBANKMENT. ● Stability analyses indicate that the recommended embankment cross section will be stable even under large earthquake loads.
2. Borrow-Auxiliary Spillway Area	<ul style="list-style-type: none"> ● Drilled 2 holes for pump testing. ● Conducted seismic surveys. ● Excavated 10 backhoe trenches. 	<ul style="list-style-type: none"> ● Reservoir rim seepage losses should be insignificant. ● AN AUXILIARY SPILLWAY MAY BE CONSTRUCTED TO OPERATE WITH A MAXIMUM RESERVOIR ELEV. OF 9280 FEET, BUT WOULD NOT BE FEASIBLE FOR LOWER MAXIMUM POOL LEVELS DO TO COST PROHIBITIVE BEDROCK EXCAVATION REQUIREMENTS.

1. The embankment should be constructed independently from the existing dam. The old embankment fill is relatively loose compared with either the foundation soils or the probable density of the new embankment. If the new dam is built on top of the old structure, differential settlements may occur which could cause cracking in the new embankment. In addition, it would be difficult and dangerous to maintain reservoir operations if the outlet works in the old dam were to be improved in place rather than constructing new outlet facilities, which is the desirable alternative.
2. A costly and difficult to construct cutoff trench or grout curtain in the dam foundation is not justified. New field data based on multiple well pumping tests indicated much lower permeabilities in the foundation than originally suspected. Seepage analyses based on computer modelling techniques indicated maximum reservoir losses of only 250 acre-feet per year under the worst possible assumptions.
3. There is no need to require that the new dam alignment be "bent" upstream to avoid the boulder pile on the right abutment. Rather, the boulders should be removed from the abutment contact and used to build a stabilizing and cost effective rockfill toe on the downstream section. A straight axis alignment will be more stable and will require less construction material.
4. A field exploration of the borrow area showed that a significant volume of cobbles and boulder size rocks exist in the materials which will be excavated to build the new dam. All these oversize (larger than 6-inch) materials must be separated from the primary embankment fill material. It has been determined that the best use of these large rocks is as additional rockfill in the downstream section. In this manner, all of the materials excavated in the borrow will be used in the

embankment, thus minimizing the surface area disturbed by the borrow operations and, at the same time, providing a more stable embankment.

5. The borrow area lies on the south rim of the reservoir, at approximately the lowest elevation on the ridge which separates the Adelaide Creek drainage from Shell Creek. This is the best location for an emergency spillway, which would be designed to carry extreme flood waters out of the reservoir and into Shell Creek below Shell Reservoir. However, construction of this emergency spillway is feasible only if the reservoir pool elevation can be raised to 9280 feet, which is the approximate bedrock elevation of the south rim. Lower maximum pool elevations would require expensive bedrock excavation to provide a spillway at this location. For maximum pool elevations of 9280 feet and above, only minimal excavation and shaping of the borrow area would be required to form an emergency spillway.

C. HYDROLOGICAL INVESTIGATIONS AND RESULTS

A hydrologic study was performed to estimate the amount of water available for storage in the enlarged reservoir. The study involved an assessment of two separate, but adjoining drainage areas. The Adelaide Creek drainage basin covers approximately 3.5 square miles which drains directly into the reservoir area. Buckley Creek, which lies to the east of Adelaide Lake, drains approximately 3.25 square miles of area above Adelaide Lake whose runoff can be diverted into the reservoir. The potential yield was based on an assessment of the historical precipitation and streamflow records in the area. Since neither Adelaide Creek nor Buckley Creek are gaged, historic streamflows for those creeks were approximated by correlating drainage areas with precipitation and stream gage records on Shell Creek. A direct drainage area ratio correlation was made between the Buckley Creek and Shell Creek drainages which have similar characteristics. Adjustments were made to account for differences in runoff characteristics, evaporation and evapotranspiration losses between the Buckley-Shell drainages and the Adelaide drainage area due to differences in vegetation cover, soil type and topography.

The Buckley Creek water will be diverted into the Adelaide drainage area through a stilling basin designed to return between 1.3 to 2 cfs of flow back into Buckley Creek. The diversion structure will allow a maximum flow of 50 cfs to be diverted into Mud Lake above Adelaide Lake. If the flow in Buckley Creek exceeds 52 cfs, the excess flow will be bypassed down Buckley Creek. Because of potentially unstable stream channel conditions during a large flood event, a relatively simple diversion structure on Buckley Creek was judged to be appropriate. This means that considerable repairs may be necessary after a major flood event. The structure as designed will divert the required flow for optimum reservoir yield, yet maintain minimum flows in Buckley Creek.

Using hydrologic modelling with the input data based on the above assumptions and correlations, reservoir yield estimates were made for various operating constraints. These reservoir operation simulations show that the enlargement of Lake Adelaide will make it possible for Shell Canal to provide 100 percent of its water diversion demands between 68 to 73 percent of the time. Under current conditions, Shell Canal meets 100 percent of diversion requirements only 52 percent of the time. Independent of environmental operating constraints, the enlargement will allow Shell Canal to meet more than 80 percent of its diversion requirements 98 percent of the time.

In order to provide these increases in the probability of meeting irrigation water demands, the reservoir storage capacity must be increased to approximately 4550 acre-feet, or a maximum operating pool of 9280 feet elevation. A storage capacity increase of this size would allow reservoir yield development of up to 4100 acre-feet in 8 out of 10 years. This yield value was calculated assuming instream flows of 1.6 cfs and 1.3 cfs in Adelaide and Buckley Creeks respectively, and a minimum pool storage value of 219 acre-feet. (Actual dead storage will be 234 acre-feet due to downstream adjustments of the dam alignment). As discussed in the previous section, a maximum operating pool elevation of 9280 feet is desirable in terms of emergency spillway use in addition to providing optimum yield conditions.

The hydrological studies also included flood estimates. Two design flood events were evaluated; the 100-year frequency storm and the probable maximum flood (PMF). The 100-year storm would produce 4.2 inches of rain in a 24-hour period, resulting in a peak inflow to the reservoir of 665 cfs. The PMF was determined to be a 72-hour event which would result in 30.6 inches of precipitation. A PMF storm would result in a peak inflow of 6765 cfs. The inflow hydrographs developed from the flood analyses were used to design the spillway configurations and dimensions.

D. HYDRAULIC STRUCTURES

The WWDC instructed the ESA team to design a spillway configuration which will handle a PMF event. The recommended flood routing scheme utilizes a side channel service spillway, located on the left abutment, which will be capable of passing up to 2800 cfs, or about one-half of the PMF outflow. The side channel spillway will operate in tandem with an emergency spillway to pass the full PMF outflow. The emergency spillway will not start spilling until outflows reach about 1500 cfs in the side channel spillway. The frequency of any spills through the emergency spillway will be extremely rare, about once in 1,000 years. The emergency spillway will be located in the vicinity of the borrow area, on the south rim of the reservoir. The emergency structure will be a shallow, broad crested weir. Construction of the emergency spillway should be relatively inexpensive and should involve only minor shaping and earthwork during reclamation of the borrow area.

Normal water releases will be handled by a 36-inch cut and cover outlet pipe extending through the embankment. A completely new outlet works should be constructed because the existing structure is in very poor shape. In addition, a new structure will be necessary if the reservoir is to remain in operation during construction.

E. CONCEPTUAL DESIGNS

1. Design Alternatives

During the course of the Level III, Phase I effort, several conceptual designs were developed as information was accumulated.

Table 2 summarizes three embankment design alternatives that were evaluated and presented in the report. These design alternatives were labeled Alternative A, Alternative B and Alternative C.

Alternative A was similar to the originally conceived design except that the alignment was moved downstream to avoid incorporating the existing embankment in the new dam. This alternative was reevaluated early in the design process after the hydrologic studies showed that higher reservoir yield could be developed with a larger embankment. In addition, the emergency spillway could not be economically constructed with the lower reservoir elevation this design would produce.

The Alternatives B and C design configurations both have a crest elevation of 9287 feet to produce a maximum pool elevation of 9280 feet. The basic difference between these designs has to do with the size of the downstream rockfill section. Alternative C was designed with only a minimal rock toe, to be constructed solely from the rock source stripped from the right abutment. The rock fill section for the Alternative B design configuration was designed to incorporate the oversized materials removed from the borrow source in addition to the right abutment rock source.

2. Recommended Design Concept

Alternative B is recommended as the best embankment design alternative. This configuration makes the best use of available construction materials, provides for the maximum reservoir yield, produces the most stable slope conditions and is the least expensive design in terms of cost per acre-foot of storage increase. Figure 2 shows the maximum section for the Alternative B design configuration.

The hydraulic structures recommended for use with the Alternative B embankment are shown on Figure 3. The side channel service spillway, capable of passing a one-half PMF flood event, will be used in tandem with a 36-inch cut and cover outlet pipe. The outlet pipe will extend through the embankment and will be controlled by a stem operated sluice gate mounted upstream.

TABLE 2
SUMMARY OF EMBANKMENT DESIGN ALTERNATIVES DEVELOPED
ADVANTAGES AND DISADVANTAGES

ALTERNATIVE	DESCRIPTION	ADVANTAGES	DISADVANTAGES
A	<p>Similar to originally conceived design presented in Level II study.</p> <ul style="list-style-type: none"> ● Crest elevation = 9280 feet ● Max. reservoir elev. = 9273 feet ● Max. reservoir capacity = 3636 acre-feet 	<ol style="list-style-type: none"> 1. Smaller reservoir and dam impact less area regarding environmental considerations. 	<ol style="list-style-type: none"> 1. Auxiliary spillway construction not feasible due to excessive bedrock construction requirements and costs. Therefore, the maximum flood that can be economically handled is a one-half PMF event. 2. Cannot develop optimum reservoir yield.
B RECOMMENDED DESIGN	<p>Larger dam than originally conceived. Utilizes all oversize rock materials to construct a large rockfill section on the downstream toe.</p> <ul style="list-style-type: none"> ● Crest elevation = 9287 feet ● Max. reservoir elev. = 9280 feet ● Max. reservoir capacity = 4548 acre-feet 	<ol style="list-style-type: none"> 1. Can develop optimum reservoir yield. 2. Can construct an emergency spillway to accommodate PMF events at low additional cost to project. 3. Utilization of all oversize materials is desirable in terms of dam stability and optimal use of borrow source. 	<ol style="list-style-type: none"> 1. Larger reservoir and dam impact greater area in terms of environmental considerations. 2. More hauling, treating and handling of oversize materials than option C.
C	<p>Same dam crest elevation and reservoir capacity as Alternative B. However, rockfill section is constructed solely from right abutment rock source. Oversize material from the borrow source is wasted.</p>	<ol style="list-style-type: none"> 1. Can develop optimum reservoir yield. 2. Can construct an emergency spillway. 3. Less treating, handling and hauling of oversize materials. 	<ol style="list-style-type: none"> 1. Larger reservoir and dam impact greater area in terms of environmental considerations. 2. Since borrow area rock is wasted, more excavation is required to provide required volume of fill. 3. Design is less stable than Alternative B due to smaller rockfill section and higher phreatic surface.

The side channel spillway will operate in tandem with the emergency spillway to handle the full PMF event. The emergency spillway can be constructed simply and inexpensively, since it will probably not operate during the life of the project. The environmental impacts associated with routing part of a PMF event down into Shell Creek will be minor when compared with the overall impact that a flood of this magnitude would have on the area.

III. PLAN OF DEVELOPMENT

Due to the remote location of the site, and the relatively short construction season, the construction operations should be carefully planned. In order to construct the dam, it will be necessary to significantly improve the existing access road. It would be advantageous to bid out the road improvements as a separate contract and construct the access road in the season prior to building the dam.

The dam construction itself will require two seasons. The first season will be used to prepare the borrow materials, remove the boulders from the right abutment, and construct the outlet works. The design will permit continued operation of the reservoir during construction. However, it will be desirable to completely draw down the reservoir during the last 4 to 8 weeks of the second construction season. This will enable the use of the existing embankment as construction material and completion of the outlet works in the most cost effective manner.

IV. COST ESTIMATES

The construction cost estimates for this project were approached in the same manner that a contractor would prepare a bid estimate. This takes into account the scope of work, materials, labor and equipment. Particular attention was given to the remote location of the site and the need to improve the existing access road, and special treatment requirements of the borrow source and abutments. Unit costs were developed and applied to quantities derived from the conceptual design options to arrive at direct construction costs. Indirect costs were added to account for engineering design, construction management, construction supervision, inspection and contract administration.

Costs were evaluated for individual elements of the project; namely, dam construction, side channel spillway and outlet works combined, emergency spillway, Buckley Creek diversion, and the access road improvements. The itemized and total costs for the recommended design Alternative B are summarized in Table 3.

Table 3
ESTIMATED CONSTRUCTION COSTS

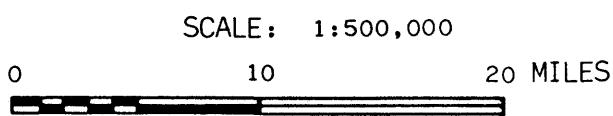
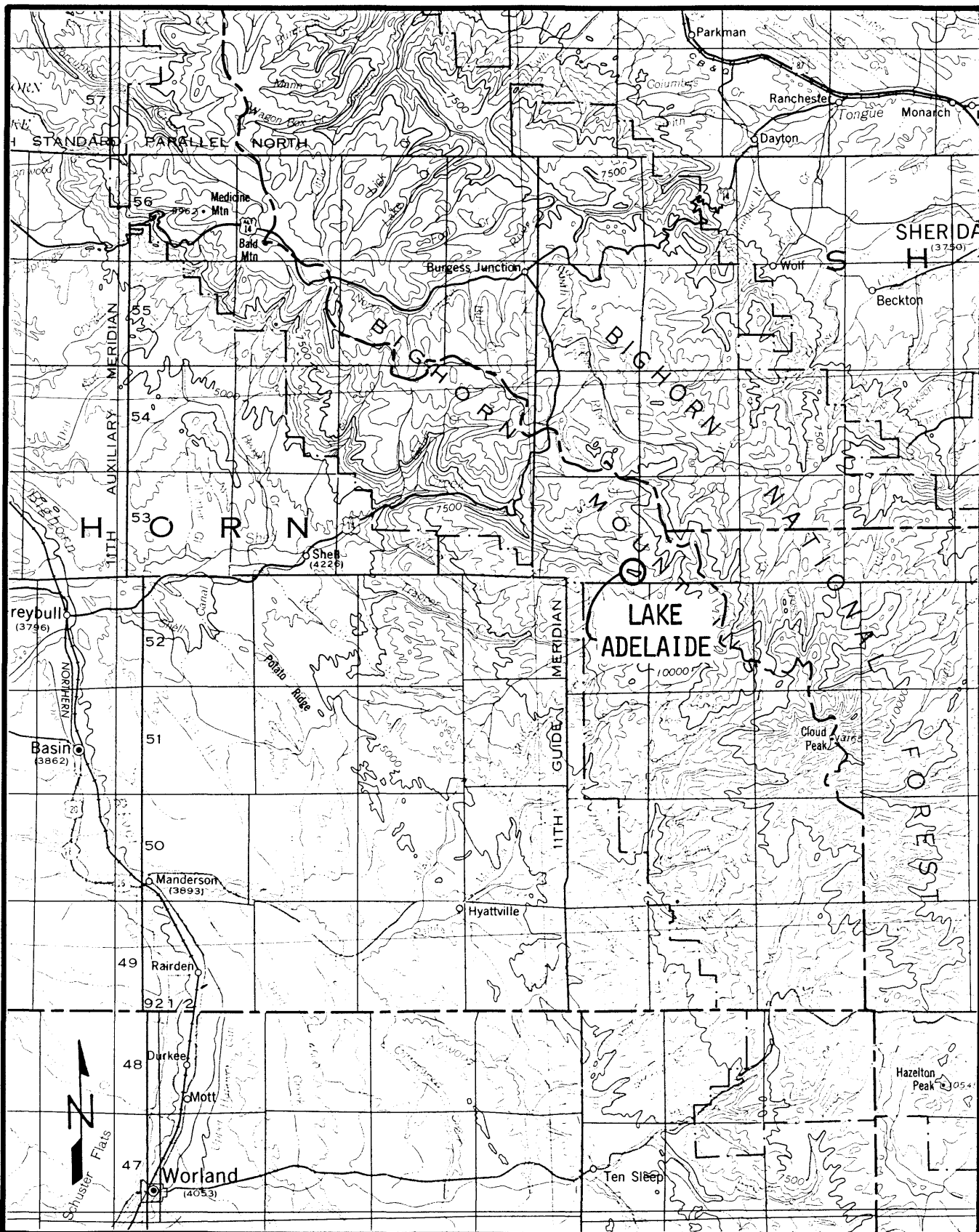
Based on the recommended conceptual design, the following costs have been estimated for the enlargement of Lake Adelaide Dam to a crest elevation of 9287 feet and a total storage capacity of 4548 acre-feet.

<u>ITEM</u>	<u>TOTAL</u>
Access Road	\$ 60,000.00
Dam Construction:	
Mobilization	\$ 100,000.00
Foundation Clearing	22,000.00
Left Abutment Preparation	29,000.00
Right Abutment Preparation	67,500.00
Embankment Construction	929,200.00
Side Channel Spillway	\$ 90,000.00
Outlet Facilities	\$ 66,200.00
Buckley Creek Diversion	\$ 24,900.00
Final Grading and Revegetation (Emergency Spillway Area)	\$ 20,000.00
	<hr/>
1985 Total Direct Cost	\$1,408,500.00
Contingency of 15%	\$ 211,500.00
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Subtotal	\$1,620,000.00
Escalation to 1987 (say 15%)	\$ 243,000.00
	<hr/>
Total 1987 Construction Cost	\$1,863,000.00
Future Engineering Costs	\$ 186,000.00
Construction Contract Administration	93,000.00
	<hr/>
TOTAL CONSTRUCTION COST	\$2,142,000.00

The estimated construction cost to raise the dam to a crest elevation of 9280 feet as originally proposed in the Level II study is \$1,831,000.00. This raise would provide a reservoir with a total capacity of 3636 acre-feet.

The average cost of the project as recommended therefore is about \$755.00/acre-foot of storage increase vs \$952.00/acre-foot for the smaller reservoir.

O & M costs have been estimated to be about \$4,500.00/year, based on the sponsor's experience.

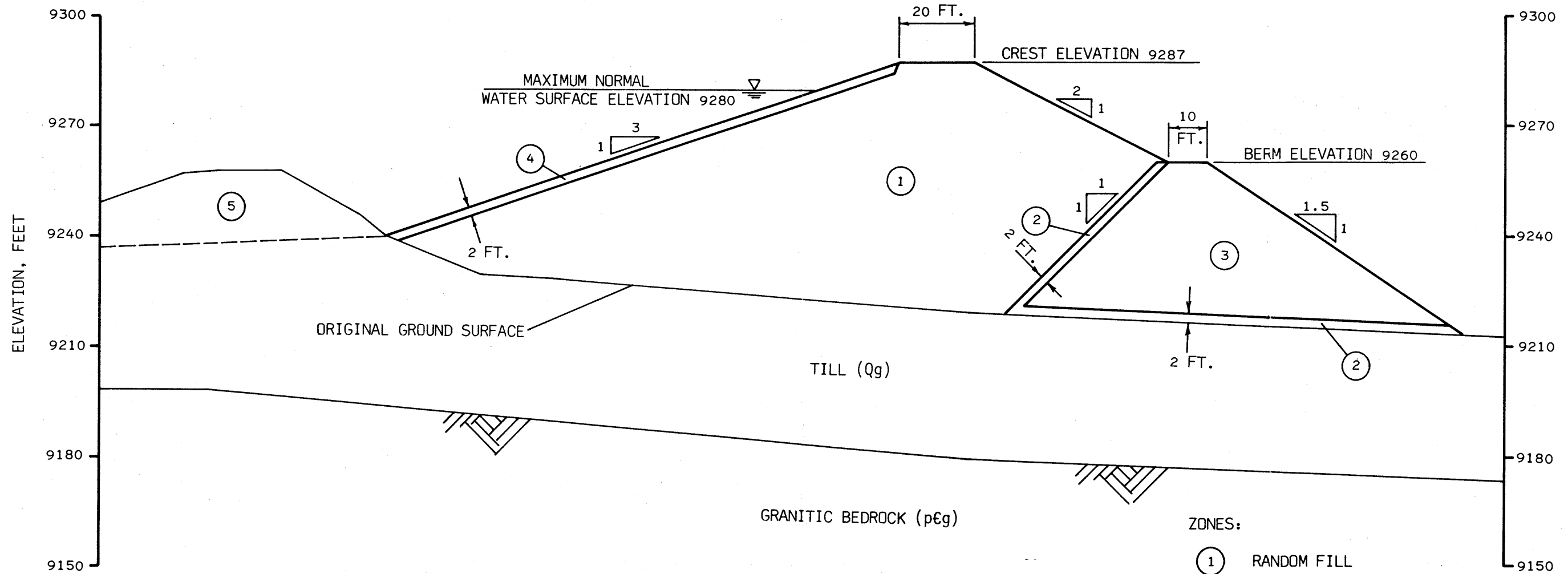


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Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
PROJECT LOCATION MAP

Checked by <i>A. J. Hambley</i>	Date <i>10-22-85</i>	Project No. <i>3235</i>	Figure No. <i>1</i>
Approved by <i>R. E. Volpe</i>	Date <i>10/22/85</i>		

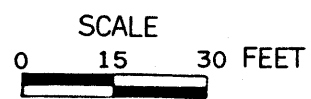
CONCEPTUAL DESIGN
ALTERNATIVE B



ZONES:

- ① RANDOM FILL
- ② FILTER
- ③ ROCK FILL
- ④ RIP RAP SLOPE PROTECTION
- ⑤ EXISTING EMBANKMENT

- NOTES:
1. This alternative design reflects an option whereby the contractor would elect to use rock for Zone 3 from both the mandatory excavation within the right abutment area as well as the +6 inch oversize material from the borrow area.
 2. The existing dam (Zone 5) will be removed for use in construction of the new dam, assuming that the reservoir operation is sufficiently lowered to allow the safe removal of the existing dam down to approximately Elevation 9240. The cut slope will be inclined upstream at a slope of 20:1.



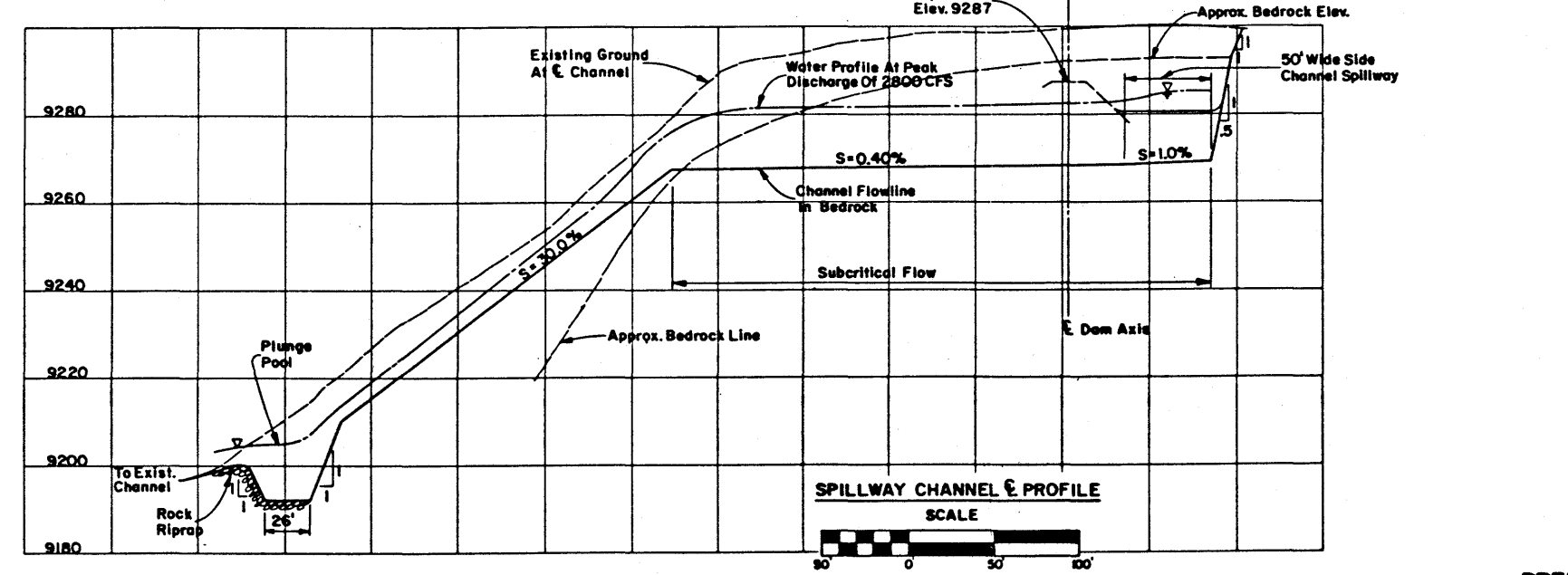
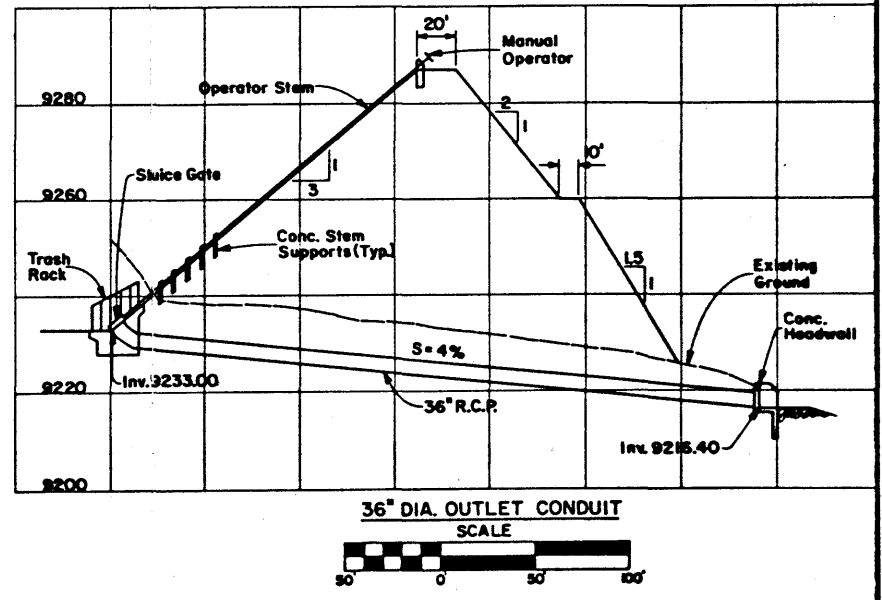
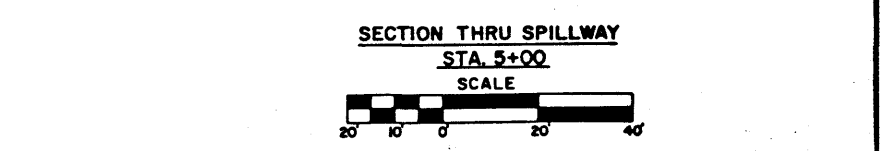
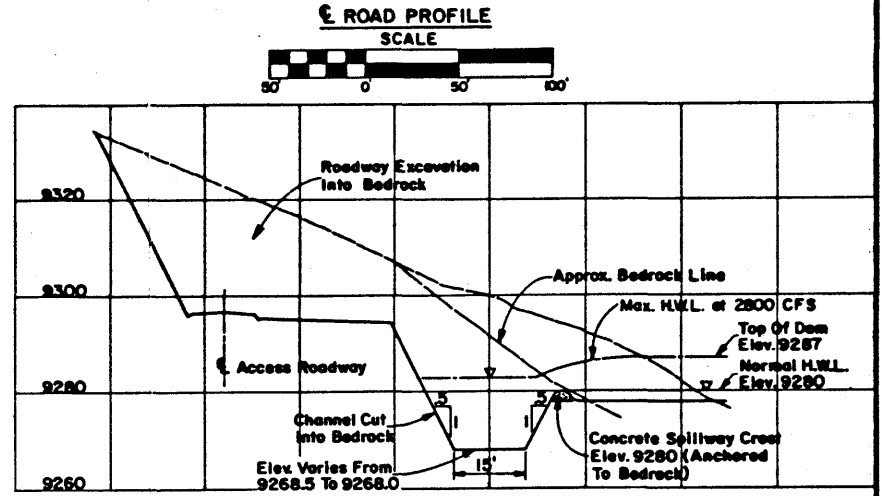
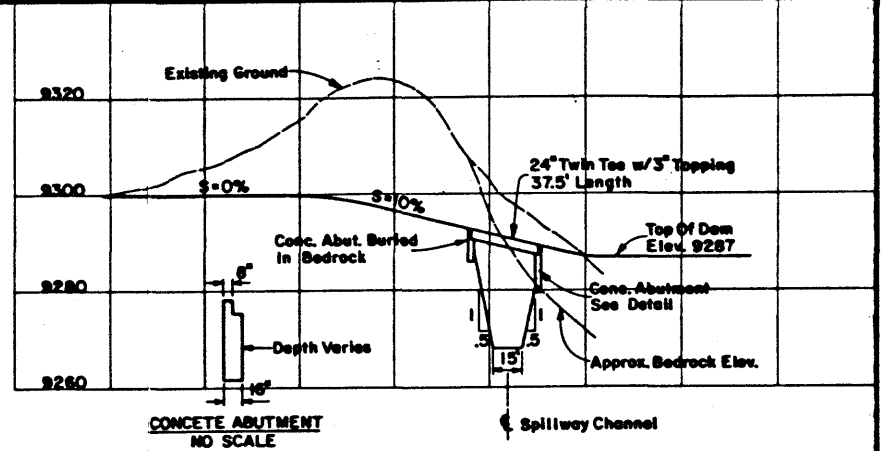
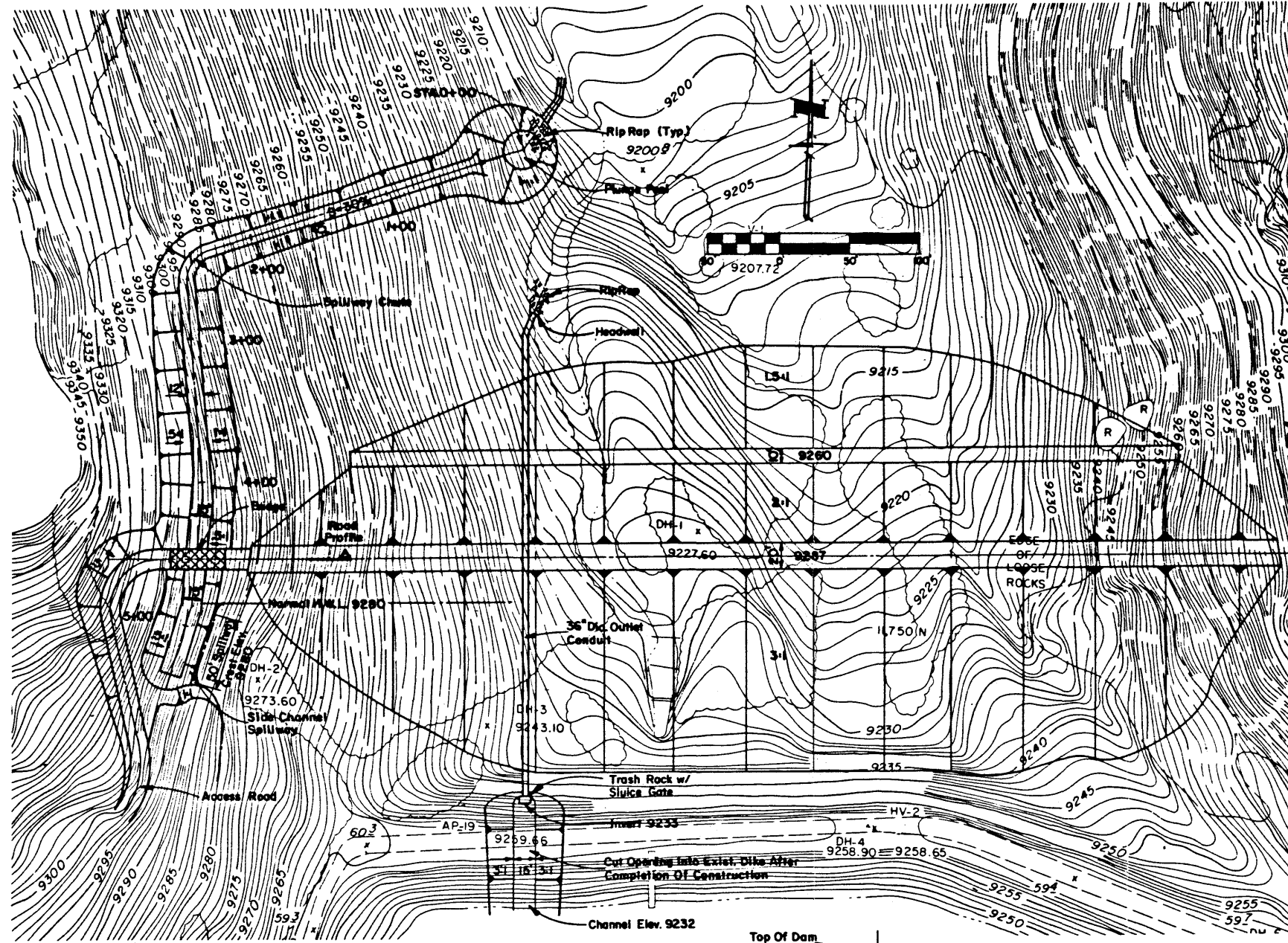
HORIZONTAL = VERTICAL

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Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
CONCEPTUAL DESIGN
ALTERNATIVE B

Checked by <i>D.J. Hamby</i>	Date <i>10-22-85</i>	Project No.	Figure No.
Approved by <i>R.K. Volpe</i>	Date <i>10/22/85</i>	3235	2



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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
SIDE CHANNEL SPILLWAY AND RESERVOIR OUTLET

Checked by WRK Date 10/23/85 Project No. 3235 Figure No. 3
Approved by DDA Date 10/23/85

PREPARED BY ARIX