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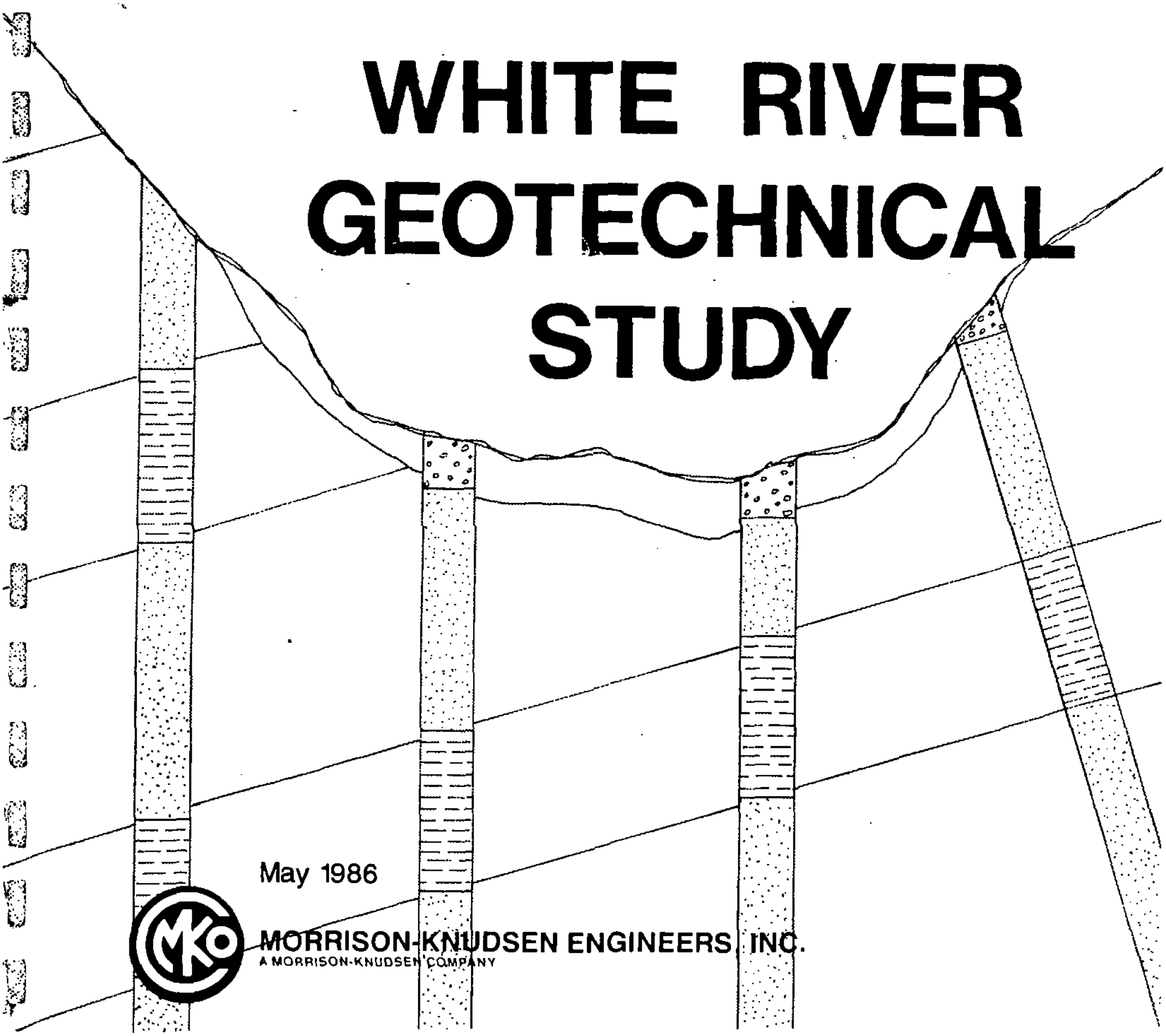
Colorado Water Conservation Board

FINAL REPORT

VOLUME I



WHITE RIVER GEOTECHNICAL STUDY

May 1986

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON-KNUDSEN COMPANY

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WHITE RIVER GEOTECHNICAL STUDY
FINAL REPORT

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WHITE RIVER GEOTECHNICAL STUDY
FINAL REPORT
EXECUTIVE SUMMARY

The 1983 session of the Colorado General Assembly passed House Bill 1102, authorizing the Colorado Water Conservation Board (CWCB) to conduct a geotechnical study of dam sites in the White River Basin. Specifically, the bill called for studying sites located at Powell Park, Lake Avery and two sites between the confluence of the North Fork and South Fork of the White River and the Highland Ditch Headgate. In May of 1985, the CWCB contracted with Morrison-Knudsen Engineers, Inc. (MKE) to perform the study.

The objective of the White River Geotechnical Study was to carry out surficial and sub-surface geotechnical investigations of the designated dam sites, and to locate and evaluate potential sources of construction material. The level of topographic data on the reservoir areas was to be upgraded. In addition, appraisal-level designs and cost estimates of various alternative dam configurations were to be made, based on the findings of the geotechnical investigation, as well as technical and economic comparisons of the sites.

The study was initiated by reviewing available reference material from previously completed geological and water development studies carried out in the White River Basin. A network of targets for air photogrammetry were set and tied in to existing survey control. Aerial photographs were taken of the potential reservoir areas and topographic maps at a scale of one inch to 500 feet, with 10-foot contours were prepared. Aerial photography was also flown for each dam site and topographic maps at a scale of one inch to 100 feet, with 5-foot contours were prepared.

Field work started with general surficial geologic mapping of the study area. The primary objective was to define the principal structural features and lithologic units in the region. In addition, reservoir permeability and slope stability of the potential reservoirs were evaluated. Borrow areas for potential construction materials were also located during the surficial mapping program.

Preliminary site screenings were carried out for the upper canyon section of the river and for the Powell Park site. The upper canyon sites considered in this evaluation were Warner Point, Choke Cherry, Dry Creek, three axes at Veatch Gulch and Canyon. From these, Warner Point and one of the Veatch Gulch sites were selected for sub-surface investigation. Two Powell Park axes and the Smith Gulch axis were screened in the Powell Park area. The furthest upstream axis at Powell Park was chosen for subsurface investigation.

Detailed site-specific geologic mapping was carried out at each dam site. This consisted of locating exposed bedrock outcrops, mapping lithologic contacts, and describing weathering and jointing patterns that might affect the stability and permeability of the dam foundations.

Sub-surface foundation investigation at the selected dam sites included auger holes, cased rotary holes and test pits at Lake Avery, and rotary holes at all other sites. Borehole penetration and permeability tests were performed systematically in all drill holes. Soil samples and rock core were logged and photographed as they were drilled. Samples of foundation material were selected for laboratory testing as necessary. Seismic refraction surveys of foundation substrata were carried out at all sites.

Potential borrow areas for construction material were investigated by auger holes and by test pits. The holes and pits were logged and samples were obtained for laboratory testing at each area.

During the initial drilling at the Warner Point site, it became apparent that the depth-to-bedrock in the river channel and the left abutment was deeper than had been expected. A re-assessment of the scope of the investigation was considered. It was decided jointly between MKE and the CWCB that, considering the objectives of the study, it would be advisable to gather information on as many sites as possible. Therefore, the Choke Cherry and Veatch Gulch sites were added to the subsurface investigative program.

Table E1 summarizes, in quantitative terms, the subsurface investigation at each site:

TABLE E1

SUBSURFACE EXPLORATION SUMMARY

DAM SITES

DESCRIPTION	LAKE AVERY NO.	LAKE AVERY L.F.	WARNER POINT NO.	WARNER POINT L.F.	CHOKE CHERRY NO.	CHOKE CHERRY L.F.	VEATCH GULCH NO.	VEATCH GULCH L.F.	CANYON NO.	CANYON L.F.	POWELL PARK NO.	POWELL PARK L.F.	TOTAL NO.	TOTAL L.F.
Auger Drilling	14	166.0	2	31.5	--	--	1	5.0	--	--	5	159	22	361.5
Cased Drilling	10	110.5	3	68.5	1	1	3	31.0	3	45.0	8	234	28	490.0
Core Drilling	--	--	8	530.0	1	120	4	289.0	5	379.0	10	465	28	1,783.0
Standard Penetration Tests	27	--	22	--	1	--	--	--	16	--	70	--	86	--
Permeability Test (Soil)	24	--	9	--	--	--	--	--	4	--	28	--	65	--
Packer Tests (Rock)	--	--	27	--	4	--	14	--	16	--	32	--	93	--
Backhoe Pits	11	95.5	12	122.0	8	77	10	65.5	10	94.5	5	52	56	507.0
Seismic Refraction	7	2,275.0	15	4,975.0	10	3,250	11	3,575.0	9	2,925.0	19	6,175	71	23,175.0

Samples of soil and rock obtained from the foundation investigation and construction materials investigation were subjected to laboratory testing to determine their physical characteristics, strength and permeability. Table E2 summarizes, quantitatively, the laboratory testing program.

TABLE 2

LABORATORY TESTING SUMMARY

DAM SITES

TEST DESCRIPTION	LAKE AVERY NO.	WARNER POINT AND CHOKE CHERRY NO.	VEATCH GULCH NO.	CANYON NO.	POWELL PARK NO.	TOTAL NO.
Samples Obtained	75	37	2	37	47	198
Sieve Analysis	27	24	2	19	26	98
Atterberg Limits	28	22	0	16	22	88
Specific Gravity	0	1	1	1	1	4
Natural Moisture	23	19	0	15	22	79
Proctor Compaction	8	4	0	4	3	19
Consolidation	7	0	0	0	4	11
Tri-Axial Shear	11	4	0	4	4	23
Direct Shear	1	1	0	1	5	8
Permeability	12	4	0	3	6	25
L. A. Abrasion	0	1	1	0	1	3
Sodium Sulfate Soundness	0	1	1	0	1	3

Appraisal-level designs and cost estimates were prepared at each dam site. In order to determine storage volume and properly size the outlet works and spillways associated with each structure, area capacity tables were calculated, hydrographs were developed, and floods were routed through the reservoirs. For the mainstem sites, dams were designed to store 50,000 ac-ft, 150,000 ac-ft, and 300,000 ac-ft. Either embankment dams or roller-compacted concrete dams, or both, were designed at each site, depending on the foundation conditions and material availability. Because Lake Avery Dam is an off-channel structure with reduced storage, potential dams were designed to store 20,000 ac-ft, 40,000 ac-ft, and 60,000 ac-ft.

The following is a site-by-site summary of the conclusions regarding each dam site studied.

Lake Avery Dam

The subsurface investigation on the left abutment confirmed the existence of a cap of relatively impervious transported material overlying the terrace gravels identified by previous investigations. The thickness of the impervious mantle ranged from two feet at the west end of the abutment near the existing dam, to twenty-five feet at the eastern end of the abutment. The material in the western half of the abutment is a brown clay of low plasticity. In the eastern half of the area studied, the clay is underlain and inter-tongued with a light brown mottled silty clay.

It is not recommended that the silty clay be left in place as a foundation material, because of its porous and collapsable nature. The clay, on the other hand, appears to have sufficient strength to tie into an impervious embankment. In the case of a zoned embankment, however, the shell zone should be founded on top of the underlying gravel.

The quality and thickness of the impervious cap should be sufficient to provide an effective impervious barrier, if left in place. It is recommended, however, that in areas where the natural blanket is thinner, it should be augmented by an additional thickness of the same material borrowed from an adjacent location.

A sufficient quantity of impervious fill is available on the left abutment to provide material for the core of the embankment and for the upstream impervious blanket.

Exposed bedrock was mapped above the waterline of the existing Lake Avery for a distance of 2000 feet upstream of the dam. At that point, it is covered with alluvial terrace material from above and cannot be traced further. After treatment, the exposed rock would provide an acceptable surface to tie in the impervious blanket. Future studies, however, should confirm the elevation of the top of bedrock at the upstream contact for the entire length of the blanket.

Seismic refraction on the right abutment showed the terrace deposit to be up to 50 feet in thickness. Even though it occurs at a higher elevation than the terrace on the left abutment, it would be exposed to reservoir water with the highest dam alternatives. Test pits excavated on top of, and upstream of, the terrace, show it to be covered with five to fifteen feet of impervious clay. This clay covers the entire abutment and extends down the upstream slope into the reservoir. Therefore, it appears that no additional blanketing would be necessary on the right abutment.

Geologic mapping along the alignment of the proposed pipeline revealed that the right-of-way would be almost entirely on side slopes composed of the Eagle Valley-Minturn Formation. The formation is composed of weak sandstone and siltstone, and includes soluble minerals such as gypsum and anhydrite. Therefore, there is a potential along the alignment for development of sink holes below the pipline and for slope failure. At several points upstream of Buford, the alignment would cross landslide debris originating from the edge of the basalt flows occurring at higher elevations. These areas also could offer potential for slope instability. Any future, more detailed study of the Lake Avery Project should include geotechnical work at the diversion damsite and along the pipeline alignment.

Since the reservoir capacities considered at Lake Avery do not coincide with those studied at the other sites, the cost estimates were expressed as units of cost per acre-foot of storage in order to draw economic comparisions. The cost per acre-foot at the three storage capacities considered was fairly constant, ranging from a high of $787/acre-feet for the 20,000 acre-foot alternative to $758/acre-feet at the 60,000 acre-foot capacity. These storage costs, however, do not compare favorabaly to the cost per acre-foot of storage at any of the other sites. The most expensive 50,000 acre-foot alternative was the RCC dam at Veatch Gulch at $704/acre-feet or eight percent less than the 60,000 acre-foot alternative at Lake Avery. The most economical 50,000 acre-foot reservoir was the RCC dam at Warner Point at $469/acre-feet or 62 percent less expensive than the Avery alternative.

Warner Point Dam

The most significant finding of the geotechnical investigation at the Warner Point damsite was that the landslides on the left abutment are surficial, extending to 50 feet in depth and do not appear to affect the feasibility of the project. The slide material would have to be completely removed to provide an adequate foundation for any dam design.

The alluvial gravel in the valley bottom is thicker than had been previously expected. The depth to bedrock ranges from 50 to 75 feet below the valley floor. This significantly affects the economics of the project, especially the RCC alternative.

The upper bedrock zone underlying the left abutment consists of a sequence of thick hard sandstone layers separated by thin weathered siltstone beds to a depth of 75 feet. The zone has been characterized as weathered rock by the seismic refraction survey, but in reality the majority of the rock is sound except for the weathered zones that dip about 15^0 toward the river. For the design and cost estimate of this report, the weathered rock was assumed to be an adequate foundation for the embankment dam, but unacceptable for the RCC alternatives. Any further geotechnical investigation at the Warner Point site would have to determine the continuity and strength of the weathered beds by sampling and testing. A detailed stability analysis of the embankment or gravity dam design would be necessary. It can be concluded from the present subsurface investigation, however, that a suitable foundation for either an embankment or gravity dam exists at a reasonable depth at the Warner Point site.

Adequate construction materials are located within close proximity for either an embankment or RCC dam. Alluvial sand and gravel for use as shell material in an embankment dam or for RCC aggregate is available in the river valley and in terraces along the right side of the reservoir. Impervious core material for an embankment design is available less than one mile north of the site.

The slopes of the reservoir created by a dam at Warner Point are considered to be stable, although further studies and surface instrumentation is indicated on the left side of the reservoir upstream of Elk Creek. The biggest question regarding the Warner Point Project is the significance of outcrops of the evaporitic facies of the Eagle Valley-Minturn Formation within the reservoir. This aspect of project feasibility should be the subject of future, more detailed studies. It is felt that the evaporitic deposits are irregular, discontinuous layers; and it is, therefore, unlikely that a single bed occurs that would connect outcrops within the reservoir with a daylight point more than five miles downstream.

The estimates show that except for the 50,000 acre-foot reservoir, the cost of an embankment dam is about equivalent to the RCC gravity dam design. The 50,000 acre-foot RCC dam is 25 percent less expensive than the embankment dam at a similar size because of the additional costs associated with constructing the spillway on the left abutment. This differential might disappear, however, if the centerline were shifted a short distance downstream to a more favorable alignment for the smaller size dam.

The Warner Point Dam compares very favorably from the economic standpoint with the other projects studied on the White River. It ranks as the least expensive site for the 50,000 acre-foot and 150,000 acre-foot storage capacities and second to the Powell Park Dam for the 300,000 acre-foot reservoir.

<u>Choke Cherry Dam</u>

After initial drilling at the Warner Point site indicated the stream-bed alluvium to be from 50 to 75 feet thick, overlying bedrock, it was hoped that the Choke Cherry alignment would provide a more favorable result. The only hole drilled, however, revealed the gravel to be at least as deep as at Warner Point.

The borehole also revealed a thickness of only about 25 feet of sound sandstone of the Maroon Formation overlying poorer quality bedrock of the top of the Eagle Vally-Minturn sequence. Although no gypsum or anhydrite was recovered as core, surficial mapping indicates these rocks to be associated with this formation, making it a very unfavorable foundation for a large water storage facility.

No boreholes were drilled on the right abutment, but the seismic refraction study showed the weathered rock zone to extend over the entire length of the abutment to depths up to 100 feet. The amount of excavation required to provide a suitable foundation for an RCC gravity dam eliminated the feasibility of that type of design.

Adequate construction materials for an embankment dam are located within close proximity to the site. Alluvial sand and gravel for use in the shell zones is abundant in the river valley both upstream and downstream of the dam. Impervious core material is available from the same borrow source as for Warner Point, less than one mile north of the site.

The slopes of the reservoir that would be formed by the construction of Choke Cherry dam are potentially unstable in the area of the surficial landslides along the left side of the reservoir between Choke Cherry and Warner Point.

Veatch Gulch Dam

The alluvial gravel in the valley floor at Veatch Gulch is only about 40 feet thick as compared to 50 to 70 feet upstream at Choke Cherry and Warner Point. The reduced depth of excavation necessary for the gravity dam is favorable for the RCC dam design, but is offset by the width of the canyon.

The massive siltstone bedrock of the Chinle Formation which underlies the alluvium would be an excellent foundation for the RCC dam. No grouting or special foundation treatment would be necessary. The quality of the overlying sandstone of the Entrada Formation that forms the vertical cliffs on either abutment was not investigated at depth. Judging from the outcrops, it appears that the rock would provide an adequate foundation but would probably require grouting.

Adequate material for RCC aggregate is available in proximity to the damsite. Alluvial sand and gravel from required excavation for the dam and spillway would be supplemented by additional gravel borrowed from the riverbed upstream or downstream of the dam.

Reservoir slope stability and reservoir permeability are not expected to be of concern for the Veatch Gulch Project.

The cost estimates show that the Veatch Gulch Dam ranks last among the four sites studied in the upper canyon stretch of the White River, for all reservoir capacities evaluated. It compares most favorably to the Warner Point and Canyon sites at a storage capacity of 150,000 and 300,000 acre-feet, but remains about 40 percent more expensive.

Canyon Dam

The surface investigation at the Canyon Dam site showed that an acceptable foundation for an embankment dam exists at the upper of the two centerlines drilled. The foundation for an RCC gravity dam is questionable due to the apparent low compressive strength of the sandstone of the Entrada Formation. Further investigation and testing would be necessary to determine the feasibility of an RCC design. In addition, the height of the RCC alternative might be limited to the elevation of the contact between the hard sandstone of the lower Morrison Formation and the highly weathered green siltstone and sandstone of the upper part of that unit. The downstream centerline is infeasible due to a deep landslide on the left abutment and an intensely fractured and weathered zone, associated with faulting, on the right abutment.

The seismic refraction lines across the valley floor showed the alluvium at the site to be thinner than at any of the other sites studied in the upper canyon. Although the boreholes at the base of the abutments revealed the bedrock to be about 40 feet deep, in the center of the valley it is apparently only 10 to 20 feet deep. The lesser depth to bedrock should have a positive effect on the economic feasibility of the site.

Geologic mapping at the site identified three northwest trending normal faults within one mile of the dam. The faults are not thought to be potentially active. Any future studies of the Canyon site, however, would have to address these faults in more detail than provided by the scope of this study.

Excavation of the side channel spillway cuts might present problems of stability because of the landslide on the left abutment. Future site investigations would have to characterize this slide in more detail.

Adequate construction materials are located within a short distance of the damsite. Alluvial sand and gravel suitable for RCC aggregate, conventional concrete, filter and drain material, or shell material for an embankment dam, is available in the floodplain immediately upstream and downstream of the dam. In addition, terrace gravels are being explored in commercial pits a few miles downstream of the site. Impervious core material is available from two downstream sites within one mile of the dam axis on either side of the river.

The reservoir formed by Canyon Dam is not expected to present any problems in regard to permeability or slope stability.

The embankment dam alternative at the Canyon site is slightly less expensive than Warner Point for the 50,000 acre-feet reservoir, slightly more at 150,000 acre-feet and equal at 300,000 acre-feet. The estimates are so close, that within the accuracy of this level of design and cost estimate, the Canyon and Warner Point embankment dams can be considered equal at all three heights studied. Unlike Warner Point, however, the dam axis at the Canyon site cannot be adjusted with the height of dam to optimize costs. The RCC dam alternatives at Canyon Dam are all about 30 percent higher than at Warner Point.

<u>Powell Park Dam</u>

The single most favorable natural topographic feature at the Powell Park Dam site is the 200-foot-high vertical cliff on the left abutment that gives way to a nearly flat bench over 800 feet in width. The bench is capped with 10 to 25 feet of terrace deposits but provides an excellent location for an inexpensive side channel spillway in conjunction with an embankment dam. The cliff is at its highest at the location investigated in this study and, at that point, is most favorable for the largest reservoir size. For the smaller reservoir capacities, a centerline further downstream might be more economical, where the left abutment bench is lower and excavation of the spillway less.

Acceptable construction materials for an embankment dam were found in close proximity to the axis. Sand and gravel for the shells and for concrete aggregate is available in the river floodplain and from several terraces occurring at different elevations upstream of the dam, within the reservoir. The required excavation of gravel and rock could be used in its entirety as shell material and riprap. Suitable impervious core material is located just upstream and downstream of the damsite on the right side of the river.

No slope stability or reservoir permeability problems are expected with the Powell Park Reservoir as it would be located wholly within gently rolling terrain formed by claystone and siltstone of the Wasatch Formation.

The cost estimates show Powell Park to be the third least expensive site studied for the 50,000 acre-foot reservoir capacity, ranking behind the Warner Point and Canyon sites by about 40 percent and 15 percent respectively. At the 150,000 acre-foot capacity, Powell Park was 10 to 15 percent more than the least expensive alternatives at Warner Point and Canyon. This is due to the amount of excavation required for the side channel spillways. If the centerline were moved downstream about 2.5 miles where the left abutment bench occurs at a lower elevation relative to the valley floor, spillway costs would be reduced and Powell Park would probably be comparable in cost to Warner Point and Canyon. For the 300,000 acre-foot reservoir, Powell Park is 15 percent less expensive than any of the alternatives at Warner Point or Canyon.

The Final Report concludes that the basic level of geotechnical information and data on potential damsites in the White River Basin has been significantly advanced by this investigation, and that the primary study objectives were accomplished.

The following represent the most significant overall conclusions of the study:

(A) Of the six damsites and associated storage reservoirs evaluated, none can be eliminated from future consideration on the basis of geologic fatal flaws.

(B) While landslides were detected on the left abutments at the Warner Point and Canyon Dam locations, the surface geologic mapping and subsurface exploration revealed that in each case the detrimental effect of these slides can be avoided by either moving the axis of the dam or removing the undesirable material during foundation excavation.

(C) No potentially active faults were identified within the study area. The three faults mapped in the vicinity of the Canyon Dam site were the most prominent instance of faulting near a damsite. The occurrence of these faults does not provide a basis to reject the site or to cause major cost increases to the dam. Future investigation of the Canyon Dam site should, however, include a more detailed evaluation of these faults.

(D) The occurrence of evaporites of the Eagle Valley-Minturn Formation within the reservoir area of Warner Point and Choke Cherry reservoirs could potentially result in above normal reservoir leakage. The unconsolidated sands and pervious limestone layers in the foundation of the Choke Cherry site require additional exploration beyond the scope of this study to assure that a serious problem does not exist.

(E) Once overburden and weathered materials are excavated at the sites investigated, the foundations at each damsite would have adequate strength to support the dam types and sizes presented in the study.

(F) There is an abundance of construction materials within a relatively short distance of each of the damsites. Materials derived from required excavation and selected borrow areas can satisfy project requirements for a wide variety of designs and sizes.

(G) With no identifiable fatal geologic flaws to restrict selection of a project with a dam at any of the sites, the selection of the most desireable site will be based on environmental, operational, institutional, and economic considerations. Only general economic considerations are within the scope of this study. The updated designs and cost estimates were based on the new geotechnical and topographic data acquired during the investigation. This information allows the dams to be ranked and compared according to estimated cost for the range of storage capacities. Tables E3, E4 and E5 summarize and compare the estimated costs for each project at reservoir capacities of 50,000, 150,000 and 300,000 acre-feet.

TABLE E3

COST COMPARISON: 50,000 ACRE-FEET RESERVOIR

Dam Location	Dam Type	Total Cost (Million $)	Percent Increase Above Least Cost Alternative	Cost/Acre-Foot ($)
WARNER POINT	(RCC)	$23.5	0	$469
CANYON	(Emb)	$27.7	18	$553
CANYON	(RCC)	$31.1	33	$622
WARNER POINT	(Emb)	$31.4	34	$628
POWELL PARK	(Emb)	$32.1	37	$641
CHOKE CHERRY	(Emb)	$34.4	46	$687
VEATCH GULCH	(RCC)	$35.2	50	$704

TABLE E4

COST COMPARISON: 150,000 ACRE-FOOT RESERVOIR

Dam Location	Dam Type	Total Cost (Million $)	Percent Increase Above Least Cost Alternative	Cost/Acre-Foot ($)
WARNER POINT	(Emb)	$38.9	0	$259
CANYON	(Emb)	$40.2	3	$268
WARNER POINT	(RCC)	$41.3	6	$276
POWELL PARK	(Emb)	$44.9	15	$299
CHOKE CHERRY	(Emb)	$53.1	37	$354
CANYON	(RCC)	$53.4	37	$356
VEATCH GULCH	(RCC)	$57.9	49	$386

TABLE E5

COST COMPARISON: 300,000 ACRE-FOOT RESERVOIR

Dam Location	Dam Type	Total Cost (Million $)	Percent Increase Above Least Cost Alternative	Cost/Acre-Foot ($)
POWELL PARK	(Emb)	$49.8		$166
WARNER POINT	(Emb)	$58.4	17	$195
CANYON	(Emb)	$58.7	18	$196
WARNER POINT	(RCC)	$60.0	20	$200
CHOKE CHERRY	(Emb)	$70.9	42	$236
CANYON	(RCC)	$80.4	61	$268
VEATCH GULCH	(RCC)	$82.1	65	$274

(H) Lake Avery Dam, the only off-channel project studied, was evaluated for different storage capacities than the mainstem sites. The results are summarized on Table E6.

TABLE E6

COST COMPARISON: LAKE AVERY

Dam Location	Storage Capacity (Acre-Feet)	Total Cost (Million $)	Cost/Acre-Foot
LAKE AVERY	60,000	45.5	$758
LAKE AVERY	40,000	30.7	$768
LAKE AVERY	20,000	15.7	$787

(I) The topographic characteristics of the valley are unique at each of the mainstem damsites. The change in the valley cross section for the various dam heights, strongly influences the relative cost of each reservoir size considered. The valley shape is also a prominent factor in the location and resulting cost of the spillways for each of the dam heights considered.

These topographic changes, over the range of dam heights needed to form the 50,000, 150,000 and 300,000 acre-foot reservoirs, have resulted in the Warner Point Dam being the least expensive at the lower sizes while the Powell Park Dam becomes the most economical at the larger sizes. The relative order of the dam location and types by cost are not the same for any of the three reservoir sizes.

The Warner Point, Canyon and Powell Park dams are within 20 percent of each other for any given size. It is concluded, therefore, that dams at the Warner Point, Canyon, and Powell Park sites should remain in consideration for future water development projects in the White River Basin.

(J) Lake Avery enlargement does provide an alternative for obtaining storage capacity without constructing a dam on the mainstem of the White River. Compared to equal-sized mainstem reservoirs, the construction cost for the larger Lake Avery Dam is substantially greater.

K) For major water resource development to be provided on the White River Basin within the State of Colorado, a moderate to large mainstem reservoir will eventually be required and there appears to be no general geotechnical restriction that would prohibit such a project at one of the three most cost-effective sites identified in this study.

001121

INTRODUCTION

001122

1. INTRODUCTION

The 1983 session of the Colorado General Assembly, passed House Bill-1102, authorizing the Colorado Water Conservation Board (CWCB) to conduct geotechnical studies on damsites in the White River Basin. Shortages in state funding resulted in the investigation being postponed for about one year. In November 1984 the CWCB requested and received proposals from several engineering firms and contracted with International Engineering Co. (IECO) to conduct the study. In early 1985 IECO became Morrison-Knudsen Engineers, Inc. (MKE).

The objective of the White River Geotechnical study was to conduct subsurface geotechnical exploration of previously identified potential dam sites, evaluate the source and characteristics of construction materials and upgrade the level of detail of the topographic and physical data on the reservoir areas. HB-1102 specified that the geotechnical investigations be conducted "on dam sites located at Powell Park, Lake Avery, and from the Highland Ditch Headgate upstream to the confluence of the North Fork and South Fork of the White River" (Plate 1).

1.1 Background

The White River Basin above the dam sites designated for the study produces an average annual runoff volume of approximately 450,000 acre-feet. At the present level of water use, only about 10 percent of that average annual volume is depleted from the White River within the State of Colorado. The White River, therefore, presents a major potential for development of new water supply for future water users in the region.

Water development in the region was first investigated in the 1920's as a part of the upper Colorado River basin-wide studies by the U.S. Bureau of Reclamation (USBR).

Additional studies in the 1940's and 1950's culminated in a reconnaissance report on the Yampa-White project in 1957. In 1959 local water interests formed the Yellow-Jacket Water Conservancy District (YJWCD), who now hold conditional water rights for levels of development conceived in the USBR plans of 1957 and subsequent studies completed in 1968. However, attempts through 1980 to obtain congressional projects authorization were unsuccessful.

In 1981, renewed interest in water supplies for agricultural, municipal and oil shale developments, was the basis for the State of Colorado to conduct the Yellow Jacket Project Study. This study completed in 1982 presented yield evaluations of projects utilizing the YJWCD water rights and presented cost estimates of alternative projects to deliver new supplies to potential users. In the following year, the YJWCD and seven major private energy companies with interest in regional oil shale development combined to direct the White River Study. The White River Study examined potential multi-purpose water projects that might employ the water rights of other entities in combination with YJWCD water rights in an effort to reduce development costs and to minimize the potentially damaging impacts of numerous, smaller, and scattered projects.

In the early 1980's several federal agencies conducted studies of water availability and potential water demands related to oil shale development in the region. This included an investigation of yields of potential reservoirs in the White River Basin.

The cornerstone of all the water resources projects formulated in the above mentioned investigations is reservoir storage. It is not infrequent for fully developed river basins in the Rocky Mountain region to require a total storage capacity of two to three times the total annual average runoff. This translates into a potential ultimate reservoir capacity development of 450,000 to 1,200,000 acre-feet within the White River Basin. The previous water planning studies were not directed toward conducting subsurface geotechnical investigations at potential damsites. Given the importance of reservoir storage and with the prominent dam and reservoir alternatives identified in the Yellow Jacket Project and White River Study, the next logical step in the basin development process, was to advance the level of geotechnical knowledge of the potential damsites and identify as early as possible any fatal flaws or prominent cost factors that might eliminate a site from further consideration. In addition, this higher level of geotechnical information and data would allow more refined cost estimates to be made over the range of dam and reservoir sizes to be considered in the future.

1.2 Report Contents

The following report summarizes the activities and presents the results of the work contracted by MKE with the State of Colorado. It discusses the geology of the region as well as the project area. A brief discussion of flood hydrology is included as well as a description of spillway and dam sizing. The process that was followed in selecting the specific sites to be studied is presented . An overview of the geotechnical methods employed in the study is described and the parameters for preparing the appraisal-level designs and cost estimates are outlined.

Each site at which subsurface exploration took place is treated as a separate section in the report. The narrative includes: the history of study at each site, a description of the reservoir characteristics, site geology, a summary of the findings of the foundation investigation and construction materials investigations, as well as observations regarding the engineering geology of the site. A description of the appraisal-level designs and cost estimates that were prepared, along with a discussion of the findings and conclusions as to the technical and economic feasibility, is included in each Section. Finally, overall conclusions are drawn with regard to the geotechnical feasibility of the various projects and economic comparisons are made of the sites for the three reservoir sizes evaluated.

Project and site topographic maps, a property ownership map, borehole and test pit logs, and photographs are included as appendices to the report. Other items contained in the appendices are laboratory test results, stability analyses of the dams, hydrologic related data, and construction cost estimates. The aerial photographs and topographic maps that were prepared in conjunction with this study were submitted to the CWCB under separate cover.

SCOPE

2. SCOPE

2.1 Study Area and Size Range

The sites studied consist of the two preselected damsites at Lake Avery and Powell Park plus at least two damsites within the boundaries of the upper White River Basin designated in HB 1102. Plate 1 presents the study area and indicates the locations of the sites initially examined. The specific locations of the dam axes were selected after examination of the storage characteristics of the resulting reservoirs and the surficial geology of the project area.

Due to the wide range of potential reservoir capacities that could ultimately be required, the study examined the site geotechnical characteristics for dams impounding 50,000 to 300,000 acre-feet at all mainstem sites and 20,000 to 60,000 acre-feet at the off-channel site of Lake Avery.

2.2 Study Activities

The following is a short description of the work tasks contained in the contract between Morrison-Knudsen Engineers and the State of Colorado that served to define the Scope of Work for the White River Geotechnical Study.

<u>Task 1 - Data Review</u>: Review previous work done in the White River Basin, including the Yellow Jacket Study, the White River Study, studies by the USBR and other reports by the U.S. Geological Survey, private individuals and companies.

<u>Task 2 - Aerial Surveys and Topographic Mapping</u>: Take aerial photographs and prepare topographic maps at a scale of one inch to 500 feet, with 10 foot contours for the project area and at a scale of one inch to 100 feet, with five foot contours for each damsite studied.

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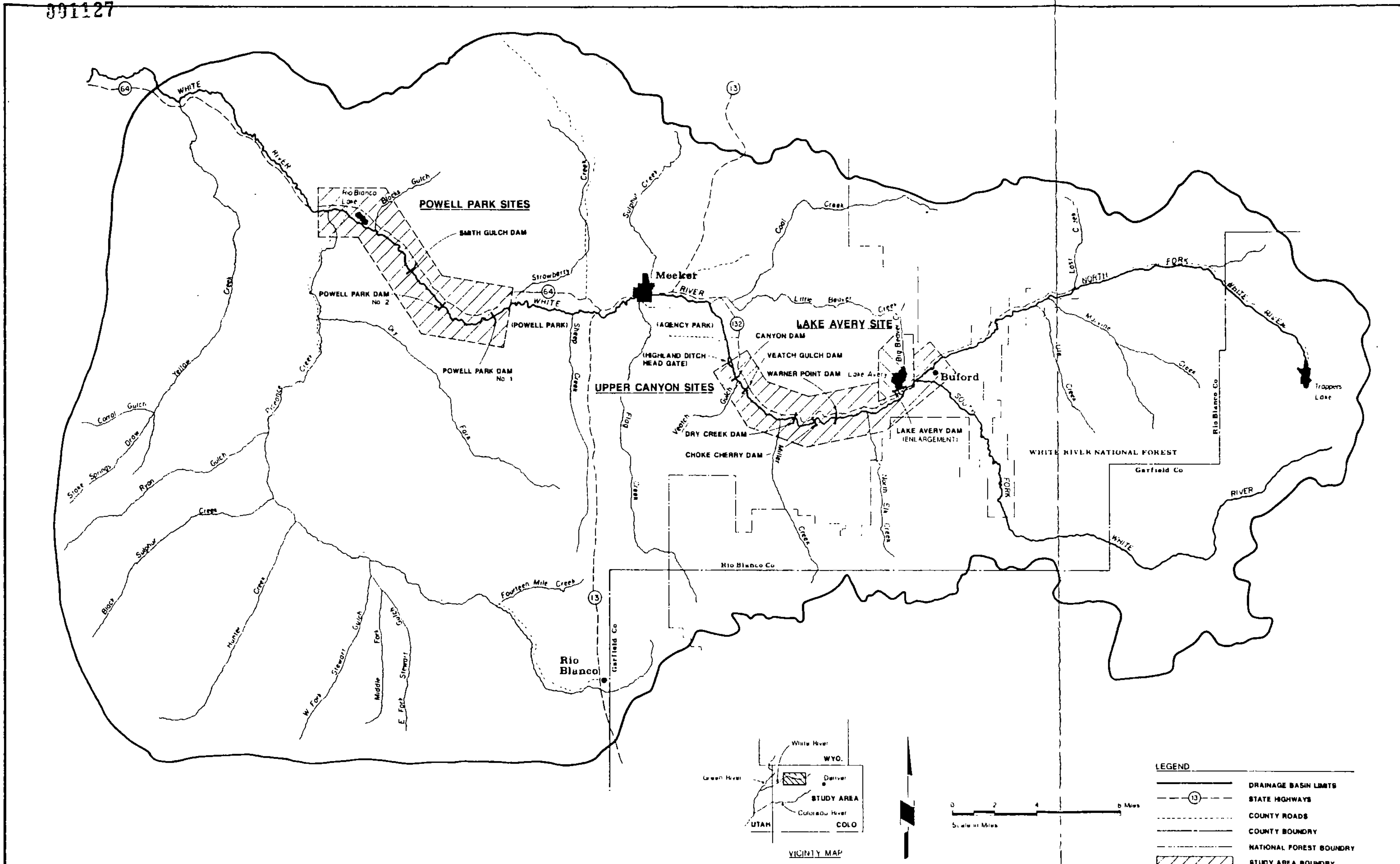

MORRISON-KNUDSEN ENGINEERS, INC.

COLORADO WATER CONSERVATION BOARD

WHITE RIVER GEOTECHNICAL STUDY

STUDY AREA POTENTIAL DAM SITES

Plate 1

Task 3 - Obtain Site Access: Make all necessary arrangements to obtain access to private and public property for the purpose of geologic mapping and where indicated, subsurface investigation.

Task 4 - Geologic Mapping: Prepare surficial geologic maps at two scales. The first covering the entire project area including all damsites and potential reservoir areas. The primary objective being to define the principal structural features and lithologic units in the project area. The second scale map was to be a detailed surficial geologic map of each damsite describing in more detail than the areal mapping, the condition of the soil and exposed bedrock outcrops, weathering, and jointing patterns that might affect the stability and permeability of the dam foundations.

Task 5 - Reservoir Area Evaluation: Evaluate the permeability and slope stability of the potential reservoirs based on the surficial geologic mapping with special attention to the area of landslides along the south side of the river near Warner Point.

Task 6 - Foundation Investigation: Carry out subsurface investigations at each damsite selected, utilizing a combination of backhoe pits, auger holes, cased and cored rotary borings, and seismic refraction surveys. Obtain undisturbed samples of the foundation materials for laboratory testing and perform borehole penetration and permeability tests in the foundation soil and water pressure tests in the foundation rock. Log and photograph all test pits, samples, and core.

Task 7 - Construction Materials Investigation: Conduct a thorough reconnaissance of available construction materials at each damsite selected. Investigate the quantity and quality of the materials by means of auger holes and back hoe pits, and obtain samples for laboratory testing.

Task 8 - Laboratory Testing: Perform laboratory tests on soil and rock samples obtained from the dam foundations and potential borrow areas to determine their physical characteristics, strength parameters, and suitability for use in the proposed dams.

<u>Task 9 - Dam Type Evaluation and Cost Estimates</u>: Prepare appraisal level designs and cost estimates for a variety of dam heights at the selected sites based on the results of the subsurface geotechnical investigation.

<u>Task 10 - Reporting and Public Meetings</u>: Keep CWCB personnel informed of the progress of study activities and of any key decisions regarding the conduct of the study. Assist the CWCB in conducting public meetings.

<u>Task 11 - Report Preparation</u>: Prepare a final report on the conduct and results of the study.

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REGIONAL GEOLOGY

3. REGIONAL GEOLOGY

3.1 Structure

Plate 2 is a reproduction of the USGS geologic map of Colorado in the region of the White River Basin. The study area is located in the Southern Rocky Mountain and Colorado Plateau physiographic provinces. The provinces are separated by the Grand Hogback, a northerly-trending monoclinal ridge that is cut by the White River just west of the town of Meeker. The White River structural basin to the east of the Grand Hogback is influenced by the White River Uplift, a broad northwesterly-elongated dome about 50 miles in length and 40 miles wide. The region to the west of the Grand Hogback is dominated by the Piceance Creek Basin, a structural depression about 100 miles long and about 40 miles wide. These two significant regional features, the White River Uplift and the Piceance Creek Basin, were formed over a period of about 35 million years beginning near the end of the Mesozoic Era during the Laramide Revolution about 60 million years ago. Block faulting, associated with tectonics, is most prevalent in the southern part of the White River Uplift; faults generally trend east-west or northwesterly.

3.2 Stratigraphy

Plate 3 is a geologic column illustrating regional stratigraphy, including ages and relative thickness of the geologic units mapped in the study area. The Plate also serves as a key to the Project Geologic Map. Over most of the region, the surface rocks are composed of a thick sequence of sedimentary formations ranging in age from Pennsylvanian to Tertiary. Small areas of Precambrian crystalline rocks and early Paleozoic sedimentary rocks are exposed in the White River Uplift and Tertiary lava flows form the "Flat Tops" of the uplift. Unconsolidated Quaternary deposits are found throughout the region.

The oldest rocks in the White River Basin are located within the White River Uplift, cropping out in small areas along the South Fork of the White River.

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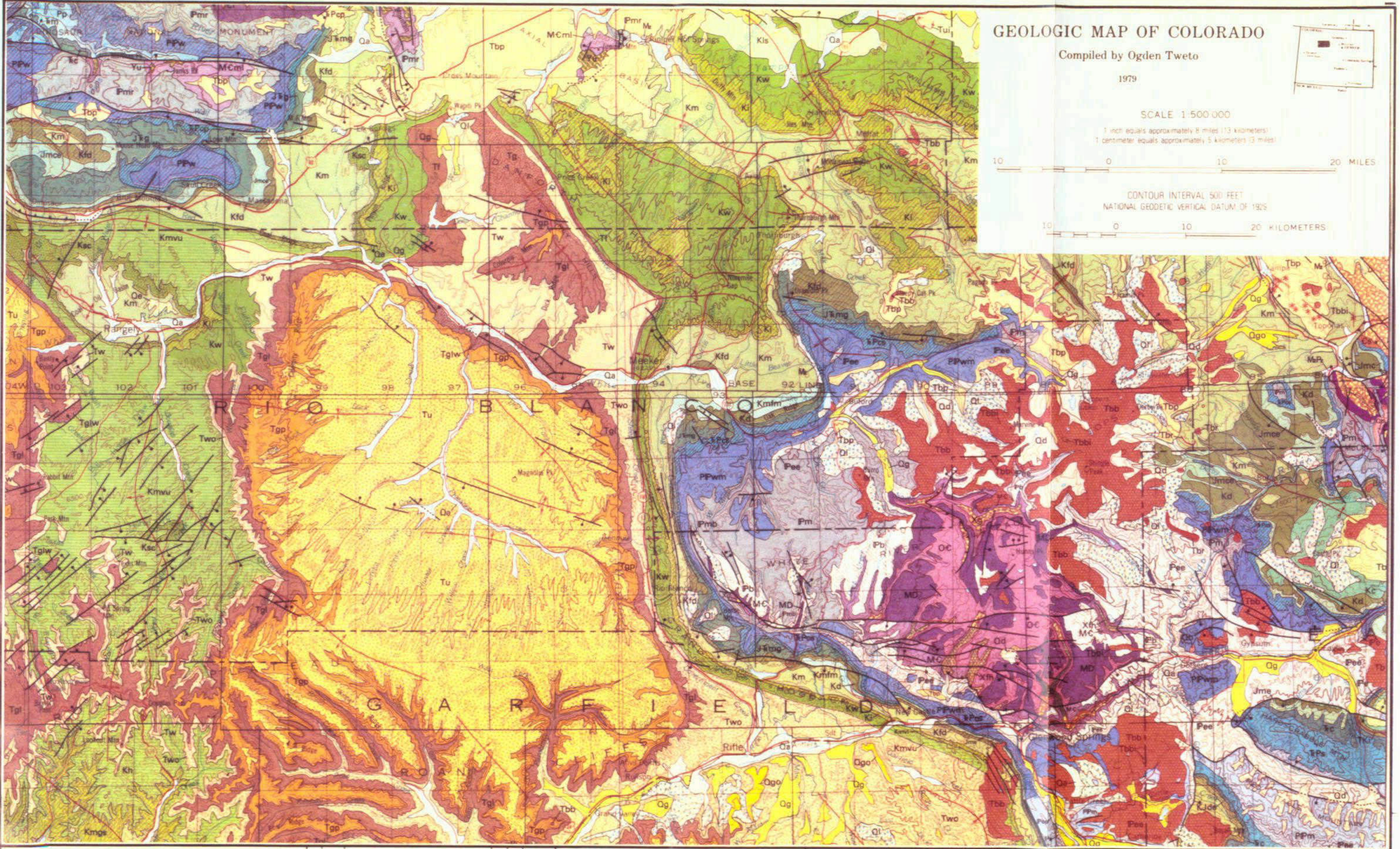

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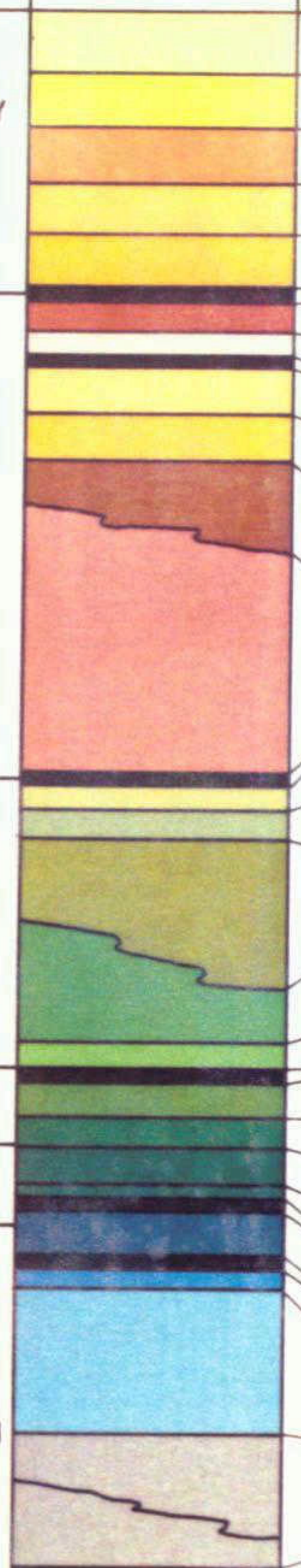
WHITE RIVER GEOTECHNICAL STUDY
REGIONAL GEOLOGIC MAP

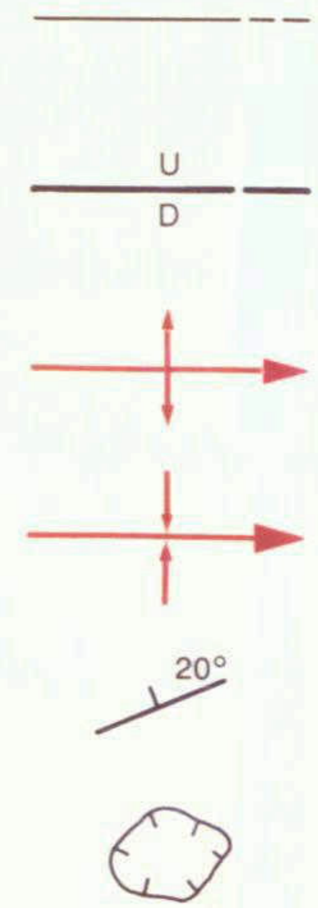
IECO NO.
SHEET OF REV
Plate 2

801133

GEOLOGIC AGE (MILLIONS OF YEARS AGO)		MAP COLOR	MAP SYMBOL AND DESCRIPTION	
CENOZOIC ERA	QUATERNARY PERIOD 11 ± 1		Qa	ALLUVIUM
			Qf	ALLUVIAL FAN
			Qc	COLLUVIUM
			Qls	LANDSLIDES
			Qt	TERRACE DEPOSITS
	TERTIARY PERIOD 70 ± 2		Tbb	BASALT FLOWS
			Tbp	BROWNS PARK FORMATION
			Tu	UINTA FORMATION
			Tqp	PARACHUTE CREEK MEMBER GREEN RIVER FORMATION
			Tql	LOWER MEMBER GREEN RIVER FORMATION
			Tw	WASATCH FORMATION
MESOZOIC ERA	CRETACEOUS PERIOD 135 ± 5		Kw	WILLIAMS FORK FORMATION MESA VERDE GROUP
			Kl	ILES FORMATION MESA VERDE GROUP
			Km	MANCOS SHALE
			Kfm	FRONTIER SANDSTONE and MOWRY SHALE
			Kd	DAKOTA SANDSTONE
	JURASSIC 180 ± 5		Jm	MORRISON FORMATION
			Je	ENTRADA SANDSTONE
	TRIASSIC 225 ± 5		℞c	CHINLE FORMATION
			℞q	GARTRA SANDSTONE
PALEOZOIC ERA	PERMIAN 270 ± 5		℞Ps	STATE BRIDGE FORMATION
			P₽w	WEBER SANDSTONE
	PENNSYLVANIAN PERIOD 310 ± 5		P₽m	MAROON FORMATION
			₽em	EAGLE VALLEY and MINTURN FORMATIONS
PRECAMBRIAN	older than 950	unit not mapped in project area	P€	PRECAMBRIAN UNDIFFERENTIATED



LEGEND



CONTACT

FAULT (RELATIVE MOVEMENT)

ANTICLINE (PLUNGE)

SYNCLINE (PLUNGE)

STRIKE and DIP (BEDDING)

SINK HOLE

GEOLOGIC MAPPING ON USGS 7 1/2' QUADRANGLES

0 2 4 6 FEET

SCALE

1 Inch= 2,000 Feet

TOPOGRAPHIC CONTOURS @ 40 FOOT INTERVALS

WHITE RIVER GEOTECHNICAL STUDY

REGIONAL GEOLOGIC COLUMN and KEY

Plate 3

They are Precambrian igneous and metamorphic crystalline rocks composed of granite, schist, quartzite, and gneiss. The crystalline rocks are overlain by several hundred feet of lower Paleozoic sedimentary rocks of Cambrian and Devonian age, including sandstone, dolomite, and limestone. The largest part of the exposed uplift area, however, is formed by upper Paleozoic rocks of the Pennsylvanian and Permian periods. These sedimentary deposits consist of dark shale, yellow sandstone, gray limestone reefs, and evaporites overlain by more than 3,000 feet of clastic red beds ranging from shale and mudstone to sandstone and conglomerate. The Paleozoic rocks are in turn overlain by rocks of the Mesozoic Era consisting of several thousand feet of red beds. The younger rocks are alternating shale, siltstone, and sandstone of the Triassic Period and light-colored sandstones and dark marine shales of the Cretaceous Period. These rocks crop out in relatively steep ridges in a narrow belt around the margins of the White River Uplift. In the study area the youngest rocks of this age form the Grand Hogback.

Younger Tertiary rocks of the Cenozoic Era are exposed in the Piceance Basin west of the Grand Hogback. They comprise a sequence at least 5,000 feet thick consisting in part of weak, variegated claystone, mudstone, and shale that generally weathers to form broad valleys. The upper part of the sequence consists of more resistant dark marlstone and light-colored sandstone that form plateaus, frequently terminating in sheer cliffs.

The youngest bedrock units in the region are the dark basalt flows and light colored sediments of Tertiary age that uncomformably cap the higher elevations in the northeast part of the White River Plateau. These rocks have a total thickness of over 500 feet and are composed of several flows interbedded with contemporaneous deposits of volcanic ash, sandstone and siltstone. Volcanic plugs form the cores of several of the higher peaks in the region.

Quaternary geology in the region is characterized by Pleistocence glacial deposits. Moraines representing several intervals of glaciation are present along most of the major valleys in the higher portions of the basin. Several levels of Pleistocene pediments and terrace deposits occur along the White River and its larger tributaries. The upper part of the study area had extensive landslide activity during the Pleistocene Epoch. Mass slumping has occurred near the limits of the basalt flows.

The youngest deposits in the region are of the Holocene Epoch of the Quaternary Period and are characterized by recent alluvial sand and gravel, landslides, colluvium, and talus.

3.3 Seismicity

The White River study area is within Area 1 on the seismic risk map of the western United States. Structures in Area 1 can be expected to suffer minor damage in the event of an earthquake. The closest recorded significant earthquakes in the region were in the vicinity of Rangely, Colorado, 50 miles to the west of the study area. They were associated with secondary recovery in the neighboring oil field during the 1960's. Six seismic events were recorded between 3.7 and 4.6 on the Richter scale, and one of IV on the Mercalli scale.

The strongest recent earthquake within a 200-mile radius of the area occurred October 11, 1960 and had a modified Mercalli intensity of VI. The epicenter was near the town of Ridgeway in Ouray County, Colorado. Several other earthquakes have been recorded in the area, two with an intensity of V. The earthquake activity in the Ridgeway area is probably associated with tectonic activity and associated faults in the Uncompahgre Uplift.

An earthquake of Mercalli intensity VII in 1901 was reported to have caused damage northeast of Gunnison, Colorado. On November 7, 1882, an earthquake with an estimated intensity between V and VI occurred in Colorado. There is lack of agreement as to the location of the epicenter. Some analysts have placed the epicenter in northwest Colorado; others indicate Grand Junction or even Denver.

Two other centers of seismic activity are within a 200-mile radius of the study area. The nearest of the two is in the vicinity of the Rocky Mountain Arsenal complex just north of Denver, where 35 earthquakes ranging from 2.1 to 5.3 on the Richter scale occurred between 1962 and 1971. That activity was associated with the subsurface disposal of fluid waste. The other center of earthquake activity is near Price, Utah, 150 miles southwest of the study area. Fifteen earthquakes have been recorded in this area, but few have been felt and no damage has been reported.

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PROJECT GEOLOGY

4. PROJECT GEOLOGY

The definition of the project area for the purpose of geologic mapping is those areas which would be impacted, or are adjacent to areas that would be impacted, by construction of one of the dams studied. Plates 4 through 8 are geologic maps prepared at a scale of one inch to 2,000 feet. The maps on Plates 4 through 6 cover the eastern, or upstream part of the project area. The easternmost limit is about three miles upstream of Buford, where the diversion facility for the Lake Avery enlargement would be located. The mapped area extends west to about one half mile below the Highland Ditch Headgate. Plates 7 and 8 cover the area from the state highway bridge just downstream of Meeker, west to Smith Gulch which is adjacent to the furthest downstream site studied.

4.1 Bedrock Stratigraphy

Eagle Valley-Minturn Formations (PPem)

The oldest rocks exposed in the mapped area are of Pennsylvanian-Permian age and crop out along the North and South Forks of the White River and along Miller Creek where the streams cut into the northwest side of the White River Uplift. The rocks are yellow to gray sandstones, shales, and carbonate reefs of the Minturn Formation intertongued with white anhydrite and gypsum of the Eagle Valley Evaporite facies. The Minturn Formation is locally transitional with gray and reddish-gray siltstones, shales, and sandstones of the Eagle Valley Formation. The terrain is characterized by steep uneven slopes with small deeply cut stream drainages. Where the Eagle Valley Evaporite facies occurs, the ground surface is frequently pockmarked with sink holes caused by the collapse of subsurface solution cavities. These units were mapped as "undifferentiated" in the field, as contacts were irregular and poorly defined. The thickness of this sequence was not determined; the literature indicates it to be in excess of 3,000 ft.

Maroon Formation (PPm)

Conformably overlying the Minturn and Eagle Valley Formations are red beds about 4,000 feet in thickness, the oldest of which is the Maroon Formation.

The contact ranges from transitional, with the multifacied Eagle Valley Formation as observed in Miller Creek and on the ridge north of Buford, to abrupt with the Eagle Valley Evaporite facies near Buford. The contact is often clearly defined by the presence of a hard gray limestone bed about 10 feet thick. This unit is harder than both the younger and older rocks and is easily mapped in the field, primarily up Elk Creek and south of Warner Point. The Maroon Formation is Pennsylvanian-Permian in age and is characterized by hard micaceous maroon-to-pink sandstone interbedded with thinner layers of dark red siltstones and mudstones. Limestone beds ranging in thickness from one inch to two feet occur throughout the formation. The Maroon is the predominant formation in the upstream part of the study area, cropping out on both sides of the river from above Lake Avery to the confluence of Miller Creek. Owing to the gentle ($\pm 15^{0}$) dip to the northwest through this area, the exposures along the right side of the river are usually spectacular barren red cliffs, with a steep stair-stepped appearance caused by the differential weathering of the thin, weak mudstone layers and the more resistant sandstone. The exposures on the left of the river are generally gentle dip slopes covered by thick vegetation with good outcrops visible only in tributary ravines.

Weber Sandstone (PPw)

The Maroon Formation is overlain by the Weber Sandstone, also of Pennsylvanian-Permian age, which is a resistant unit composed of light-gray to tan, fine to medium grained, well sorted, well consolidated sandstone that forms the crest of high ridges and northwest-facing dip slopes. The unit ranges from 100 to 250 feet in thickness in the project area. Utilizing the Weber Sandstone as a marker was one of the most useful tools in establishing the structural geology in the eastern part of the study area.

State Bridge Formation (TPs) and Chinle Formation (TPc)

Unconformably overlying the Weber are the Triassic-Permian State Bridge and Chinle Formations. These formations are characterized by orange-to-red, massive siltstone and pebble conglomerate deposits with occasional discontinuous light buff sandstone and gray limestone lenses. One of these beds was tentatively identified as the Gartra Sandstone and mapped as the contact between the two formations. The relatively weak rock of the State Bridge and Chinle Formations forms gently rolling topography covered by scrub oaks through the middle two miles of the upper canyon section of the study area, causing the valley to widen between Miller Creek and about one mile upstream of Veatch Gulch. The combined thickness of the two formations is about 2,000 feet. They mark the upper limit of the red bed deposition that started in Pennsylvanian time.

Entrada Sandstone (Je)

The Chinle Formation is overlain by the Jurassic Entrada Sandstone which is distinguished by the spectacular, near vertical, pale orange sandstone cliffs formed on either side of the White River valley. The sandstone is of eolian origin and is uniformly medium grained, poorly cemented, and crossbedded. The total thickness of the formation ranges from 120 to 160 feet. The northwesterly dipping cliffs intersect the valley floor just upstream of the confluence of Veatch Gulch.

Morrison Formation (Jm)

The Jurassic Morrison Formation overlies the Entrada sandstone. The basal 150 to 200 feet of the formation is composed of white-to-gray, well sorted, well cemented, fine-to medium-grained sandstone. This unit forms fairly steep slopes and cliffs on the right side of the river and caps the dip slopes on the left side. The upper 500 to 600 feet of the Morrison is characterized by variegated green and purple mudstone and shale. The best exposure of these weak units is in the road cut just upstream of Agency Park. The thickness of the Morrison Formation in the study area is approximately 800 feet.

Dakota Sandstone (Kd)

The highest elevations in the eastern part of the mapped area are formed by the Cretaceous Dakota Sandstone that unconformably overlies the Morrison Formation and forms northeasterly trending ridges and gentle northwest dip slopes. The sandstone is light grey-to-tan, fine-to medium-grained, and well cemented. The Dakota has a thickness of 50 to 100 feet.

Frontier-Mowry Formations (Kfm) and Mancos Shale (Km)

To the northwest, at the limit of the eastern map area, the Dakota Sandstone dips below a sequence of Cretaceous gray shales and interbedded sandstones. The oldest unit is the Mowry Shale, which is characterized topographically by a trough between the Dakota Sandstone and the overlying Frontier Sandstone. The sandstones form resistant hogback ridges striking northeast. The Frontier Sandstone, in turn, dips below the gray Mancos Shale to the northwest to form the broad rolling topography bordering Agency Park just upstream of Meeker.

Iles (Ki) and Williams Fork (Kw) Formations

The western map area is bordered on the east by the Grand Hogback, a monoclinal ridge of resistant Cretaceous rocks dipping steeply to the west. The monocline is formed by the Iles and Williams Fork Formations which overlie the Mancos Shale and are predominantly light colored massive sandstones with thin shale interbeds and coal seams.

Wasatch Formation (Tw)

The broad flat valley known as Powell Park west of the Grand Hogback is flanked and underlain by the Tertiary Wasatch Formation. It is composed of variegated fine sandstones, siltstones, and claystones more than 5000 feet in thickness. Weathering of these weak rocks has produced a gently rolling topography with low buttes formed by the more resistant sandstone layers.

Lower Member Green River Formation (Tgl)

At the west end of Powell Park the White River enters into a broad valley incised into the youngest rocks in the lower western end of the study area. The oldest of these is the Tertiary Lower Member of the Green River Formation which conformably overlies the Wasatch Formation. It is about 2,000 feet thick and is composed of interbedded sandstones and shales. The sandstone ranges from fine-grained, well-rounded, well-sorted, thinly-bedded quartz sandstone to coarse-grained, moderately-sorted, crudely cross-bedded feldspathic sandstone. The sandstone units range in thickness from a few feet to over one hundred feet. While some thick sandstone units are continuous throughout the project area, many units are lense-shaped and discontinuous with lateral thickening and thinning. The thickness of the sandstone beds controls the development of the prominent bluffs and uneven slopes along the river.

Parachute Creek Member - Green River Formation (Tgp) and Uinta Formation (Tu)
The highest elevations in the lower study area to the south and southwest of the river are formed by marlstone cliffs of the Parachute Creek Member of the Green River Formation. The oil rich marlstone unit is about 1,000 feet thick. The cliffs are capped by weak sandstones and marlstones of the Uinta Formation. Weathering of this unit has produced the gently rolling topography of the Piceance Creek Basin which supports sparse vegetation, primarily of scrub oak.

Basalt Flows (Tbb) and Browns Park Formation (Tbp)
The youngest bedrock units in the study area are basalt flows and contemporaneous sandstone, siltstone, and ash deposits of the Browns Park Formation. These units are of Tertiary age and are found in the upper White River Basin. They uncomformably overlie the older sedimentary units and their near-horizontal dip results in the "Flat Tops" of the highest mountains in the White River National Forest.

4.2 Quaternary Geology

Quaternary deposits are generally confined to lower elevations along the course of the White River. With the exception of the older stream terraces and alluvial fans, these deposits are still accumulating. Units are generally unconsolidated and crudely stratified. Thickness and morphology range widely between different units and are dependent upon the rate of outcrop weathering, mechanism of sediment transport, and transport distance.

Terrace Deposits (Qt)
The oldest Quaternary units are the terrace gravels, which are distinguished by nearly horizontal, even surfaces. Although younger terraces occurring at lower elevations are more extensive, remnants of older terraces are found at elevations as high as 280 feet above the present level of

the White River. Generally, these deposits are unconsolidated, poorly sorted, and contain rounded, sand-to cobble-size clasts of varying composition. Terraces unconformably overlie bedrock and have thicknesses less than 50 feet.

Terraces are more extensive at lower elevations, where the youngest terraces exhibit less erosion or are covered by still younger colluvium and alluvial fans. At higher elevations, slope wash and development of small drainages have left only small remnants of previously extensive terraces.

Unlike other Quaternary units whose source material is in close proximity to the deposit, terrace gravels are derived from erosional detritus of many exposed bedrock formations in the White River drainage basin. The deposits are formed as post-glacial alluvial outwash from glacial deposits upstream that provide the source for stream-transported sediment.

<u>Alluvial Fan Deposits (Qf)</u>

Alluvial fans occur at the mouths of small streams draining into the White River. These are evenly-sloped, fan-shaped deposits. The areal extent and thickness of fans is proportional to the area of the drainage basin serving as the sediment source. In some areas, older alluvial fans have completely covered portions of stream terraces and smaller secondary fans have developed on the next lower terrace. The fans are most pronounced in the downstream study area below Powell Park.

Alluvial fan material is unconsolidated, crudely stratified or massive, and poorly sorted at the heads of fans changing to moderately sorted at the base. Grain sizes range from silt to pebble-size clasts, probably because granular disintegration and transport of sedimentary rocks tends to produce finer grained deposits than would denser, more resistant rock that form the terraces.

<u>Colluvial Deposts (Qc)</u>

As rock outcrops weather and disintegrate, pluvial and creep processes transport material to the bases of slopes. The transported material, or colluvium, is a poorly-sorted mixture containing angular fragments of all grain sizes. The upper portions of slopes generally have only a thin veneer of colluvium covering bedrock. Colluvium is mapped only where its accumulation is thick enough to form a distinct landform, usually unstratified, unconsolidated, wedge-shaped deposits on the lower portions of slopes.

Thicknesses of mapped colluvium range greatly, depending on slope angle, proximity of a stream to the base of a slope, and the competence of the bedrock source. In areas where slopes are gentle and the rock formations are easily eroded, maximum thicknesses appear to be as great as 75 to 100 feet. In areas where exposures of resistant rock form a high angle slope facing toward a stream flowing along the base of the slope, maximum thicknesses appear to be only five to ten feet.

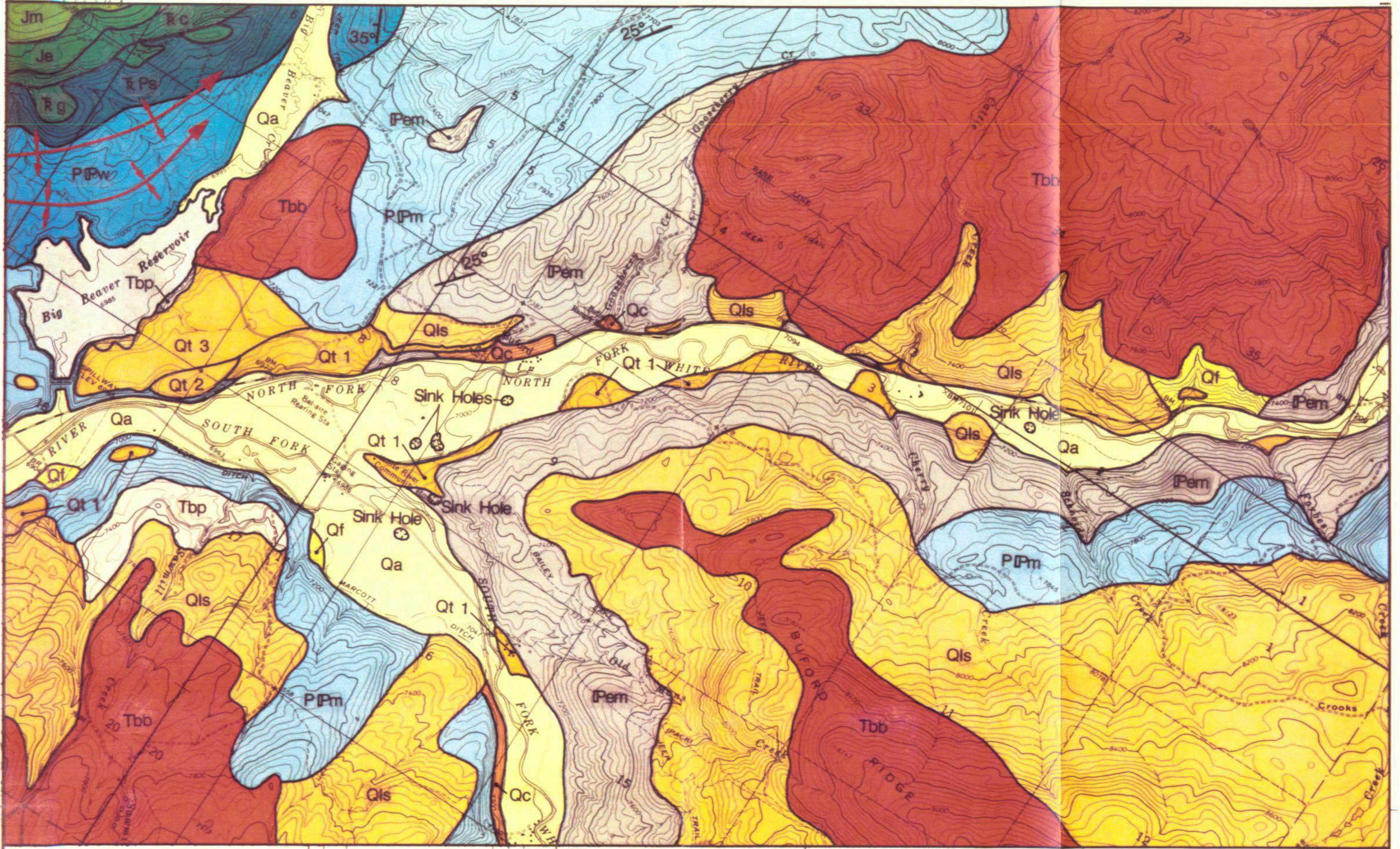
Landslide Deposits (Qls)

Extensive landslide deposits occur along the left side of the river in the upper canyon section in the vicinity of Warner Point. No in-place bedrock was mapped in this area, and the terrain is characterized by a hummocky appearance. There is no indication of recent movement of these slides. Massive slumping is also evident upstream where sound Tertiary basalt flows overlie weak sedimentary rocks.

Recent landslide activity can also be seen in the lower part of the study area downstream of Powell Park. There, occasional massive toppling has occurred as the weathering of weak shale beds has undermined overlying sandstone layers, releasing large blocks to slide down the slopes.

Alluvial Deposts (Qa)

In modern stream drainages, including the White River, interbedded gravel, sand, and silt form unconsolidated deposits in both active channels and small floodplains.



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WHITE RIVER GEOTECHNICAL STUDY

PROJECT GEOLOGIC MAP

IECO NO.	
SHEET OF	REV.
Plate 4	

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WHITE RIVER GEOTECHNICAL STUDY

PROJECT GEOLOGIC MAP

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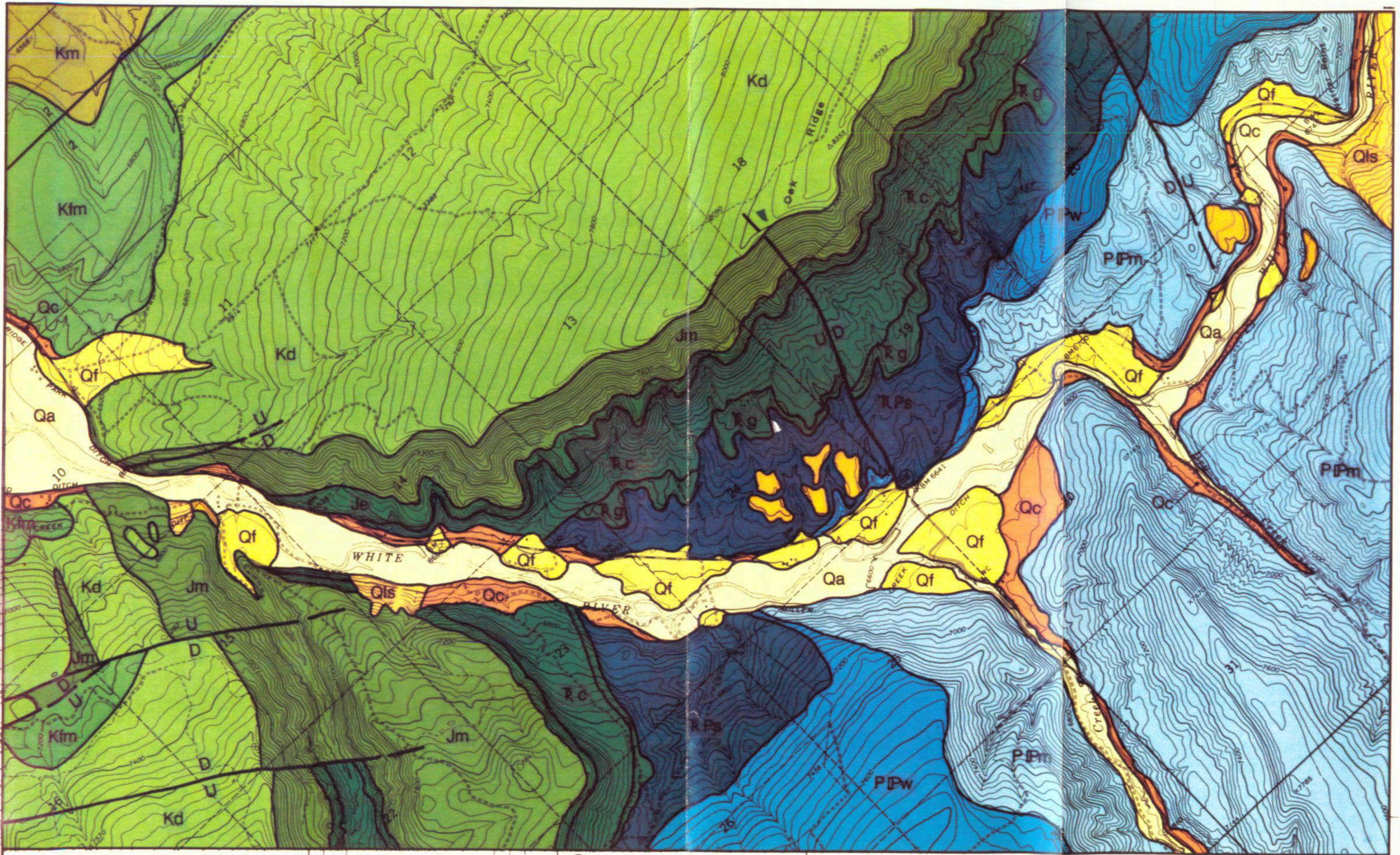
Plate 5



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WHITE RIVER GEOTECHNICAL STUDY
PROJECT GEOLOGIC MAP

IECO NO.
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Plate 6

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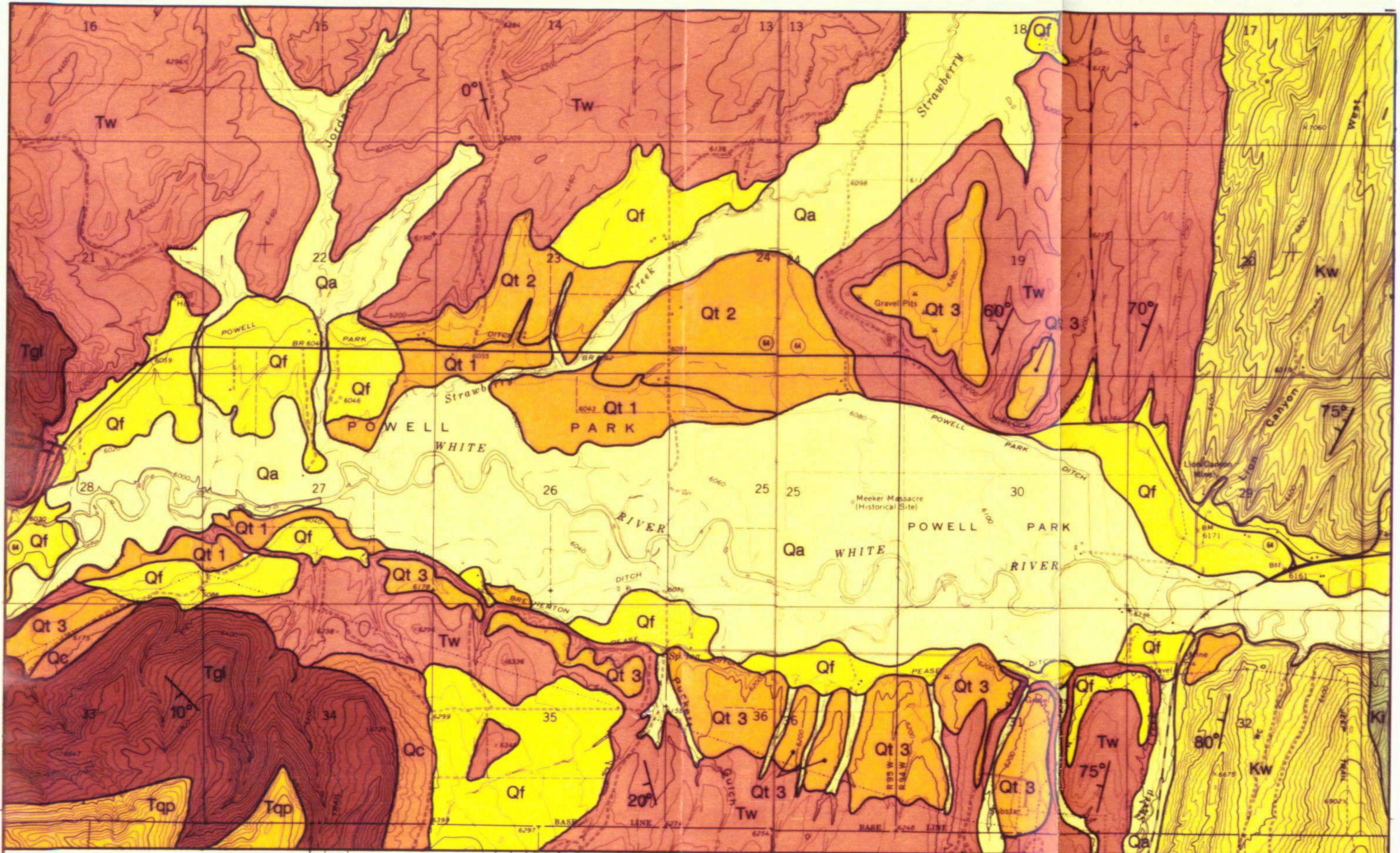

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WHITE RIVER GEOTECHNICAL STUDY
PROJECT GEOLOGIC MAP

IECO NO.
SHEET OF REV
Plate 7

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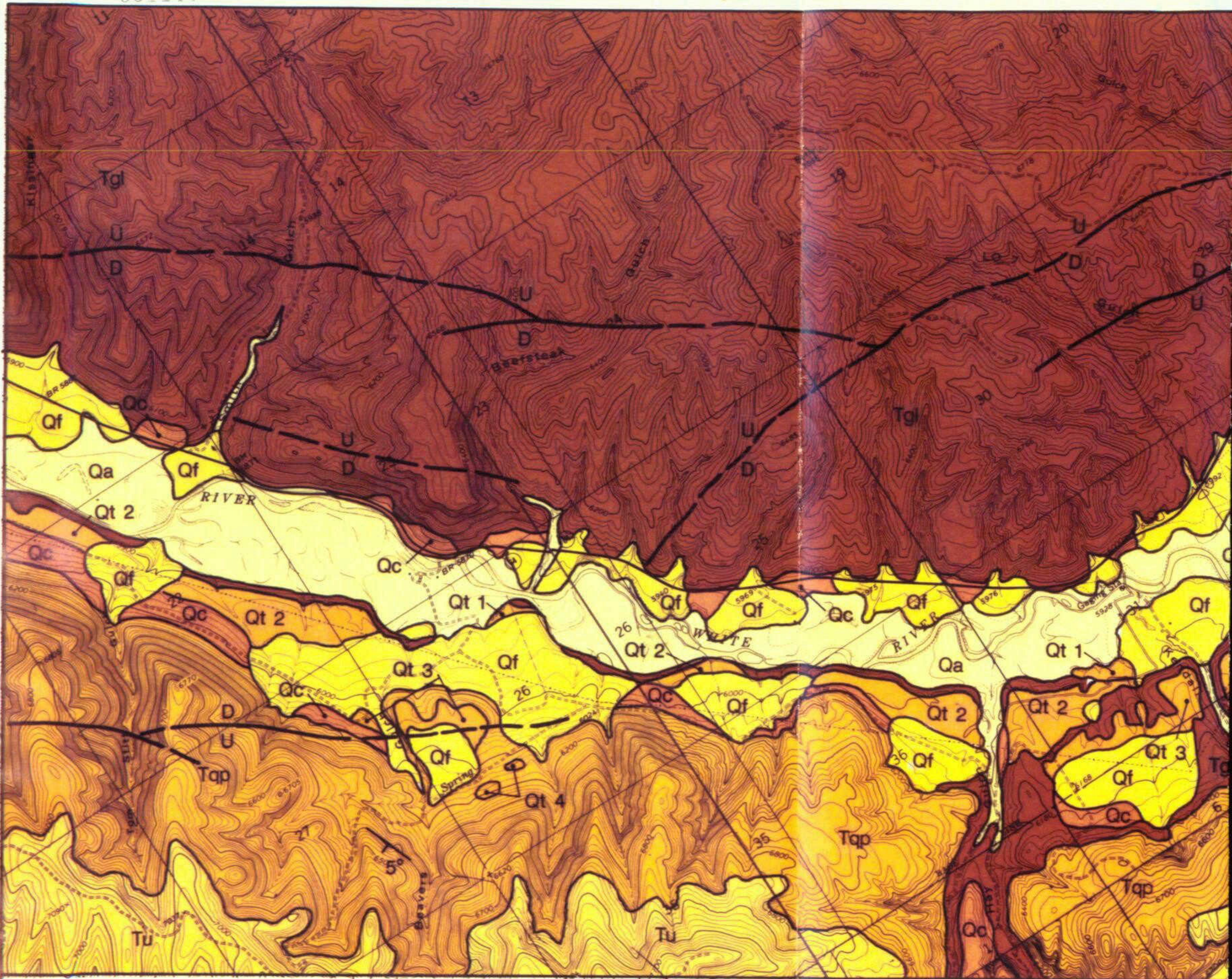

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PROJECT GEOL

4.3 Structural Geology

The eastern or upstream portion of the study area is strongly influenced by the White River Uplift. The Paleozoic-Mesozoic sedimentary rocks strike about N70^{o}E and dip gently from 10^{o} to 20^{o} to the northwest. An exception to this trend is the upper end of the Lake Avery Reservoir where the strike of the mapped units swings around to about N40^{o}W and the dip steepens to 45^{o} to 60^{o} to the southwest. This is due to the influence of the Yellow Jacket anticline located to the northeast. The anticline is an important northwesterly plunging structural feature on the northern edge of the White River Uplift.

The western or downstream part of the study area is structurally situated on the northeastern margin of the Piceance Creek Basin, which is a broad northerly-trending downwarp bounded on the eastern margin by the Grand Hogback. This structural depression is characterized by Tertiary sedimentary formations gently dipping from all sides toward the center of the basin. In the study area, rocks forming the Grand Hogback strike north-south and dip steeply 50^{o} to 70^{o} to the west where the ridge is cut by the White River just west of Meeker. The Tertiary rocks west of Powell Park strike to the northwest and dip gently 5^{o} to 20^{o} to the southwest.

Folding

No major folds are present within the study area, although several gentle parallel folds were mapped in the Weber Sandstone and Maroon Formation in the upper Lake Avery Reservoir area where the regional strike and dip are altered by the Yellow Jacket anticline. Local gentle folding occurs throughout the study area but is not of sufficient significance to warrant mapping.

Jointing

The principal jointing in the region is probably controlled by the tectonic stresses associated with the White River Uplift. Two subvertical joint systems are evident over the entire study area. One set strikes N50^{o}W to N60^{o}W and the other N10^{o}E to N50^{o}E. Minor offsets and slickensides were observed locally along both of these joint systems. An additional principal set of joints is usually found parallel to bedding. The frequency of these joints seems to be controlled by the lithology. Many of the bedding plane joints are open, and provide a path for the migration of ground water. Consequently, these joints are often oxidized and weathered. A fourth set of vertical joints occurs locally striking parallel to the river valley. These joints are caused by stress relief associated with river downcutting.

Faulting

Five faults were identified in the eastern map area. One of these occurs near the confluence of the White River and Miller Creek. North of the White River, a northeast-trending normal fault is evidenced by an offset in the School House Tongue of the Weber Sandstone. In addition, a ten-foot-thick sandstone layer in the State Bridge Formation disappears, and linear trends of small drainages and prominent outcrops of Jurassic-age Entrada sandstone, typically found elsewhere, are disrupted. Folding patterns in the thin sandstone unit in the area of its truncation and the offset in the Weber Sandstone indicate that the downthrown side is to the east. Total displacement is probably greater than fifty feet.

In the vicinity of Warner Point, two adjacent ridges north of the main highway are capped by Weber Sandstone of roughly equal thickness cropping out at different bedding plane projections. The difference between the lowest elevation of sandstone outcrops on the two ridges, looking perpendicular to strike, exceeds the maximum thickness of the unit. Displacement along this northeast-trending normal fault is probably fifty to one-hundred feet, with the downthrown side to the west.

In the westernmost portion of the upstream mapped area, two parallel northwest-trending normal faults offset Triassic through Cretaceous age exposed bedrock formations about one-half mile up Veatch Gulch from its confluence with the White River. The Entrada Sandstone and the Chinle Formation are truncated at the southernmost fault. The block between the faults appears to be downdropped forming a graben. A displacement of about 50 feet is indicated. A third fault, parallel to the first two, is located about one mile north in the hollow just above the road cut at the head of Agency Park. The Dakota Sandstone to the southwest of the fault is downthrown about 50 feet.

Several northwest-trending block faults are shown on the map in the western map area. These are inferred from mapping by others, but were not comfirmed in the field.

Although precise age of faulting is difficult to determine, the faults described probably developed or were reactivated during the Miocene-Pliocene age period of uplift in the region. No evidence of recent movement can be detected in the Quaternary deposits near any of the fault localities.

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CHAPTER 5

AERIAL PHOTOGRAPHY AND TOPOGRAPHIC MAPPING

5. AERIAL PHOTOGRAPHY AND TOPOGRAPHIC MAPPING

Aerial photography was flown at high altitude for the entire project area and topographic maps prepared at a scale of one inch equals 500 feet with contours at 10-foot intervals. Contours were drawn to elevations well above the highest possible reservoir levels in each of the study subareas. The potential damsites were also photographed individually from the air and a topographic map constructed at a scale of one inch equals 100 feet with contours at five-foot intervals.

Key maps showing the area covered by each sheet of the one inch equals 500 foot scale topographic mapping are attached to this report as Appendix A. The aerial photographs and originals of the topography at both scales were submitted to the CWCB under separate cover. If, in the future, originals of the negatives are needed to make additional prints of the aerial photographs, they may be obtained through the following MKE subcontractors:

Scale: 1" = 500'

Towill, Inc.
608 Howard Street
San Francisco, CA 94105

Telephone: 415-982-1758

Scale: 1" = 100'

Nichols Associates, Inc.
770 Horizon Drive
P. O. Box 2327
Grand Junction, CO 81502

Telephone: 303-243-8975

CHAPTER 6

GEOTECHNICAL EXPLORATION

6. GEOTECHNICAL EXPLORATION

6.1 Geologic Mapping

A surficial geologic map of the entire study area was made at a scale of one inch equals 2,000 feet. The objective was to delineate the principal geologic units and the important faults and structural features in the area. The map was compiled by a combination of consulting existing geologic maps, studying aerial photographs, and field mapping. The resulting maps are included in Section 4 above.

In addition, detailed surficial geologic mapping at a scale of one inch equals 100 feet was done at each of the potential damsites. These maps show the principal geologic contacts included in the areal mapping and also include smaller local features important to the engineering feasibility of the sites, such as bedrock outcrops, strike, dip, and joint measurements, and Quaternary surface features such as colluvium, slides and terraces. Maps for each individual site studied are included in Section 10.

6.2 Drilling Program

A subsurface drilling program of auger, casing advancer, and core drilling was carried out under an MKE subcontract to Boyles Brothers Drilling Company of Golden, Colorado.

The drilling program consisted of a number of holes at each potential damsite. The borings were usually initiated with an auger or casing advancer through the soil profile. In this zone, disturbed (split spoon) and undisturbed (Shelby and California) samples were obtained for laboratory testing. Standard penetration tests and constant head and falling head permeability tests were carried out at intervals to give information on the soils bearing capacity and permeability. When inpenetrable material was encountered, either blocks of rock, boulders, or bedrock, the method was changed to diamond core drilling of either NC or NX diameter. A high priority

TABLE 1

SUBSURFACE EXPLORATION SUMMARY

DAM SITES

DESCRIPTION	LAKE AVERY		WARNER POINT		CHOKE CHERRY		VEATCH GULCH		CANYON		POWELL PARK		TOTAL	
	NO.	L.F.	NO.	L.F.	NO.	L.F.	NO.	L.F.	NO.	L.F.	NO.	L.F.	NO.	L.F.
Auger Drilling	14	166.0	2	31.5	--	--	1	5.0	--	--	5	159	22	361.5
Cased Drilling	10	110.5	3	68.5	1	1	3	31.0	3	45.0	8	234	28	490.0
Core Drilling	--	--	8	530.0	1	120	4	289.0	5	379.0	10	465	28	1,783.0
Standard Penetration Tests	27	--	22	--	1	--	--	--	16	--	70	--	86	--
Permeability Test (Soil)	24	--	9	--	--	--	--	--	4	--	28	--	65	--
Packer Tests (Rock)	--	--	27	--	4	--	14	--	16	--	32	--	93	--
Backhoe Pits	11	95.5	12	122.0	8	77	10	65.5	10	94.5	5	52	56	507.0
Seismic Refraction	7	2,275.0	15	4,975.0	10	3,250	11	3,575.0	9	2,925.0	19	6,175	71	23,175.0

was placed on good core recovery. The boreholes penetrated a sufficient depth into sound bedrock until, in the opinion of the MKE geologist on site, no further useful information could be obtained. Following conclusion of a boring, the bedrock portion of the hole was pressure tested with water at 12-foot intervals utilizing a double pneumatic packer.

All boreholes were logged by MKE geologists as they were drilled. The soils were classified by the Unified Soil Classification System (USCS). The core was photographed and stored in wooden boxes. The borehole logs and core photographs are contained in Appendix C. The core is being stored at a ranch belonging to Dave Smith in Meeker.

The address is as follows:

David Smith
1540 W. Market
Meeker, Colorado
Telephone: 303-878-5670

The drilling program was augmented by the excavation of backhoe test pits. The test pits were excavated in potential borrow areas primarily for the purpose of visual inspection of the material and to obtain samples for laboratory testing. Logs of these test pits are included in Appendix D.

The locations of the boreholes and test pits are shown on the plates included in Section 10 for each site. Table 1 is a matrix that summarizes the quantities of drilling and test pit excavation carried out at each site and in total.

6.3 Seismic Refraction

The seismic refraction method of geophysical exploration was used to augment the information obtained from the drilling program. A 12 channel seismograph was used in conjunction with geophones spaced 25 feet apart along a line 300 feet long. A light explosive charge was detonated 25 feet from each end of the line to initiate the measurements. Information was obtained on the depths to strata of different velocities beneath the line. The primary purpose of the seismic refraction program was to determine the depth to the top of sound bedrock between drill holes, and to define any deep channels in the river bottoms.

The locations of the seismic lines are shown in the plans in Section 10 for each site. The results are incorporated into the geologic sections included in the same section of the report. The extent of seismic refraction surveys is summarized by lines and in linear feet by site and in total by the matrix in Table 1.

6.4 Laboratory Testing

Testing of soil and rock samples obtained from the borings and test pits was carried out at the MKE field laboratory in Meeker and at ATEC laboratories in Denver, Colorado. All samples were physically characterized in the field and at the MKE laboratory by visual classification, grain size analysis, moisture content, and plasticity. Larger samples were sent to ATEC for more sophisticated testing such as sodium sulfate soundness, Los Angeles abrasion, Proctor maximum density, consolidation, triaxial shear, direct shear, and laboratory permeability. The results of these tests are given in Appendix E and the quantities of testing are summarized in matrix form for each site and in total in Table 2.

TABLE 2

LABORATORY TESTING SUMMARY

D A M   S I T E S

TEST DESCRIPTION	LAKE AVERY NO.	WARNER POINT AND CHOKE CHERRY NO.	VEATCH GULCH NO.	CANYON NO.	POWELL PARK NO.	TOTAL NO.
Samples Obtained	75	37	2	37	47	198
Sieve Analysis	27	24	2	19	26	98
Atterberg Limits	28	22	0	16	22	88
Specific Gravity	0	1	1	1	1	4
Natural Moisture	23	19	0	15	22	79
Proctor Compaction	8	4	0	4	3	19
Consolidation	7	0	0	0	4	11
Tri-Axial Shear	11	4	0	4	4	23
Direct Shear	1	1	0	1	5	8
Permeability	12	4	0	3	6	25
L. A. Abrasion	0	1	1	0	1	3
Sodium Sulfate Soundness	0	1	1	0	1	3

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FLOOD HYDROLOGY

7. FLOOD HYDROLOGY

Basin hydrology and reservoir operation were not objectives of this study. However, it is necessary to take these factors into consideration in order to determine the general sizing, location, and cost of the spillways that would be required for the storage dams examined.

The spillway cost can be a major component in the total cost of a dam and reservoir project and may therefore be the deciding factor in the selection of the dam type. Previous studies had indicated the potential of roller compacted concrete (RCC) dams at some of the White River damsites. The spillway section for RCC gravity dams can be placed in the center of the dam as an overflow section with little additional cost. An earth dam, however, cannot tolerate overtopping without failure and must either be sized large enough to store most of the design flood or have a separate spillway large enough to pass the flood safely. The spillway section usually cannot be placed over the top of earthfill dams and thus large excavations and costly concrete linings are frequently required around the abutments.

It was an objective of this study to evaluate dams over a range of reservoir sizes for each of the potential dam-sites selected. The spillway requirements are a function of the size of the reservoir at the spillway crest. Although the magnitude of the design flood remains constant at each site, the spillway size will normally be smaller for the larger reservoirs due to partial storage of the flood inflow. The routing effect of the design inflows was taken into account in this study by performing reservoir routings considering design flood hydrographs for the various sites.

Because of the proximity of several damsites in the upper canyon region, one design flood was used as representative for all the sites. A larger flood was estimated to occur at the Powell Park damsites, as the drainage basin size there is approximately 90 percent greater than at the upper canyon sites. The basin above the expanded Avery Dam is much smaller and would generate a flood of significantly lower magnitude. The design flood for all dams examined was selected to be equal to the Probable Maximum Flood (PMF).

The Avery Reservoir and Canyon reservoirs are located upstream of the city of Meeker. For reservoirs in the size range of 50,000 to 300,000 acre-feet, the sudden failure of a dam would pose a significant potential for loss of life in the lower valley. These dams would thus be classified as high hazard dams and as such would have to have spillway facilities that could safely pass the PMF.

A dam at the Powell Park site might possibly be classified as a moderate hazard structure, but the downstream Taylor Draw Reservoir and the Town of Rangely would be endangered by a sudden failure of the dam. For the purposes of this study, therefore, the PMF has been used as the design flood for the Powell Park sites as well.

A PMF had previously been computed for the above mentioned locations as part of the earlier White River Study and the Yellow Jacket Project. While those PMF estimates were preliminary and could change as a result of more detailed final design evaluations, they are adequate to be employed in the generalized consideration of spillway requirements for this study.

Table 3 summarizes the basic characteristics of the PMFs used in this analysis. It represents the upper limits of calculations and would not be larger for final design.

The PMF computations were based on the Probable Maximum Precipitation (PMP) amounts determined using the procedures and data in Hydrometeorological Report No. 49 published by the National Oceanic and Atmospheric Administration (NOAA).

For the mainstem locations (Powell Park and Canyon sites), the flood hydrograph of the PMF was routed through reservoirs of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. Area-capacity relationships were a key factor in this routing process.

In all flood routings, the reservoirs were assumed to be full at the time the PMF occurred. The spillways were all considered to be ungated and the outflow controlled by only the spillway width and reservoir storage characteristics.

For each of the reservoir sizes, a number of spillway widths were selected ranging from 100 to 600 feet and the resulting maximum spillway depths at the spillway crest were determined. Even for the widest spillways examined and for the larger reservoirs, where each foot of reservoir rise provides a great

TABLE 3

PROBABLE MAXIMUM FLOODS

Site Location	Approximate Drainage Basin Area (square miles)	PMF Peak Discharge (cfs)	PMF Peak Volume (acre-feet)
Avery	34	32,000	8,600
Canyon Sites	550	151,000	115,000
Powell Park	1,050	192,000	177,000

volume of storage, the spillway flows per unit of width are large. This measure, called unit discharge, was examined in each case and a criterion established to maintain the unit discharge under 500 cubic feet per second per foot of spillway width. For the embankment dams, the PMF routings were used to select a reasonable spillway width. The resulting maximum reservoir level that would be reached while passing the flood determined the crest elevation of the dam. For the case of RCC dams the same routing computation of the PMF was used to select the spillway width over the dam crest.

A summary of flood routings, inflow hydrograhs, and area capacity tables for each site are contained in Appendix G.

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DESIGNS AND COST ESTIMATES

8. DESIGNS AND COST ESTIMATES

At each site selected, dams were designed to impound three alternative storage volumes. For the mainstem sites these were 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. At the off channel Lake Avery site the sizes were 20,000 acre-feet, 40,000 acre-feet and 60,000 acre-feet.

Embankment dams were designed for all sites investigated except Veatch Gulch where only a roller compacted concrete gravity dam was considered. RCC gravity dam alternative designs were prepared at the Warner Point and Canyon sites.

The dams were all designed to meet accepted safety factors for stability based on the foundation characteristics and construction materials available, and the results of field and laboratory testing of these materials. Results of the stability analyses are summarized in Appendix F. The designs are of appraisal-level quality and are intended to be conservative. Future studies at the feasibility level may, by taking advantage of additional information, be able to improve on these designs and reduce the estimated costs.

8.1 Embankment Dam Designs

All the embankments except Powell Park were designed with similar construction materials and therefore have identical cross sections. The upstream slopes are 2.5H to 1V and the downstream slopes are 2H to 1V. The slopes of the impervious core are 0.75H to 1V upstream and 0.5H to 1V downstream. At Powell Park Dam, upstream and downstream transition zones are provided parallel to the core to accomodate the fine sand derived from required foundation excavation. All embankment dam designs included a 10-foot-wide chimney filter downstream of the core and a drainage blanket between the foundation and the downstream shell. The upstream slope of the dams is protected with a three-foot-thick riprap zone. Crest widths were chosen as a function of dam height in accordance with Table 4:

TABLE 4

EMBANKMENT DAMS: Crest Width vs. Height

Normal Maximum Water Surface (NMWS) Height above streambed (feet)	Crest width (feet)
90-140	30
140-180	32
180-230	34
230-270	36
270-310	38
310-350	40

A cutoff trench to bedrock was designed for all the embankment dams with a width equal to one third of the normal maximum water surface (NMWS) height above streambed or a minimum of 30 feet with side cut slopes of 1H to 1V.

Depending on the quality of the bedrock revealed by the subsurface investigation, a grout curtain was designed to a depth of one third the NMWS height above streambed.

Outlet works were designed to drain the top five feet of the reservoir in five days and to evacuate it to the dead storage elevation within 60 days. The outlet works consists of a single 8-foot diameter concrete-lined conduit under the upstream portion of the dam. The downstream half of the conduit is steel lined. The intake is a gooseneck arrangement with a bellmouth, protected by trash racks situated just above the minimum pool elevation. Releases would be controlled by a hydraulically operated high pressure gate valve at the downstream end of the conduit and protected by an emergency valve located in a valve chamber in the center of the dam. Access to the emergency valve is through a vertical concrete-lined shaft from the crest. The terminal structure consists of a concrete stilling structure located at the downstream toe of the dam. The structure is designed to maintain a minimum tail-water elevation in the event a hydroelectric unit were added to the project. Details of the outlet works design are shown in Plate 9.

Spillways for the dams were designed to safely pass the Probable Maximum Flood (PMF) without overtopping the dam. One foot of freeboard was allowed during the PMF when determining crest elevations. Spillway widths, overflow depths and dam crest elevations were established considering free-flow conditions. No gates were considered in any of the spillway designs. A maximum unit discharge criterion of 500 cubic-feet-per-second-per-foot was adopted for sizing the spillways and a discharge coefficient of 3.4 was used for routing the floods.

All spillway designs for embankment dams are of the side channel type, excavated through the dam abutments. The spillways consist of a concrete-lined approach channel, a concrete ogee weir and a partially or fully lined chute. The location, shape, slope and amount of concrete lining varies with the alternative. The Powell Park Dam spillway discharges into an unlined channel excavated into bedrock and the discharge velocities are stilled naturally. In all the other designs, a flip bucket is provided for energy dissipation. Plate 9 shows some general details of the spillway designs.

8.2 RCC Gravity Dam Designs

All RCC dams were designed with vertical upstream faces and 0.7H to 1V downstream faces to the spillway ogee elevation, above which the downstream face is also vertical. A one-foot-thick sheath of conventional facing concrete is provided on both the upstream and downstream face to protect the RCC from freeze-thaw deterioration and to provide an impervious barrier against seepage at the upstream face. A grout curtain equal to one third the hydraulic height of the dam is provided where the subsurface investigations indicated a need. A drainage gallery is included in all RCC designs connecting to drainage tunnels into the abutments. A drainage curtain,

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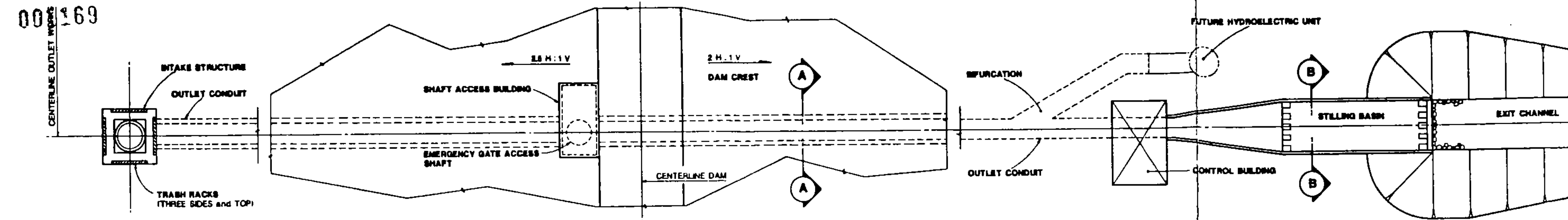
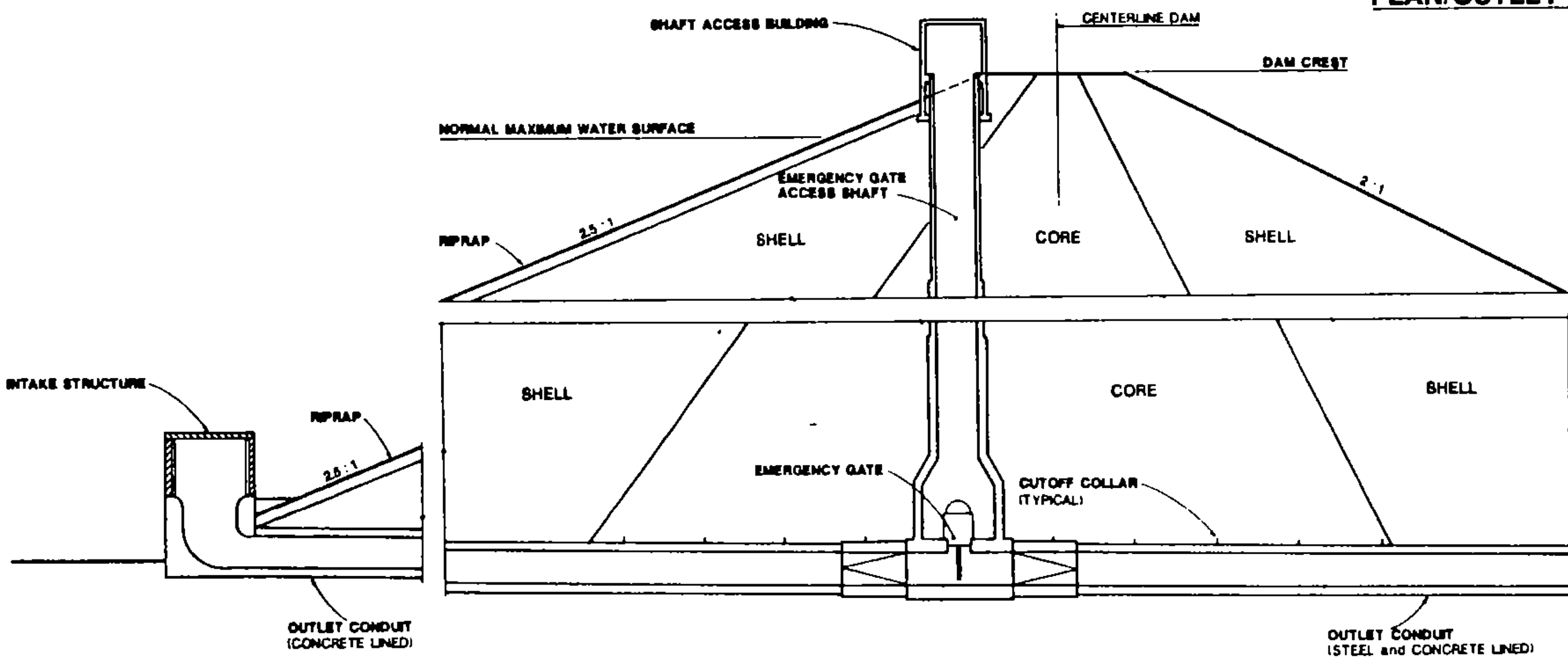
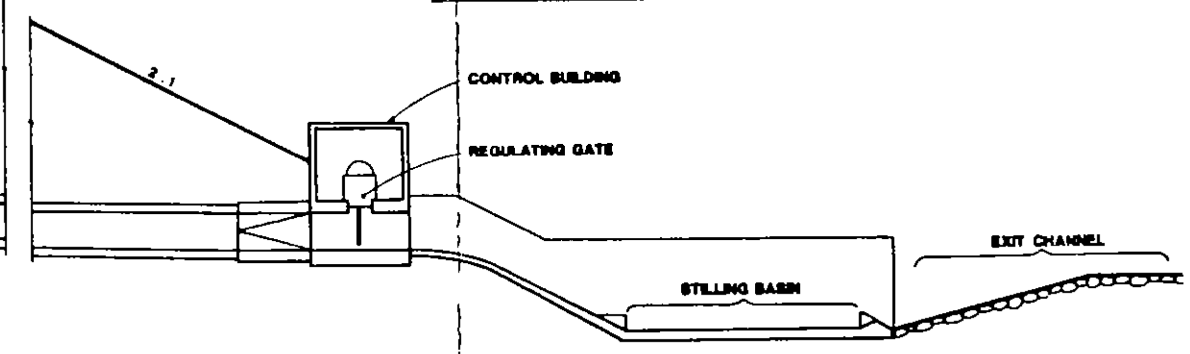
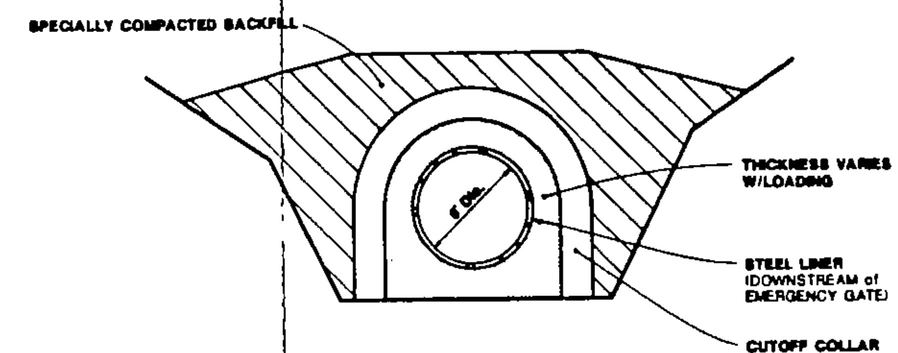


PLAN/OUTLET WORKS

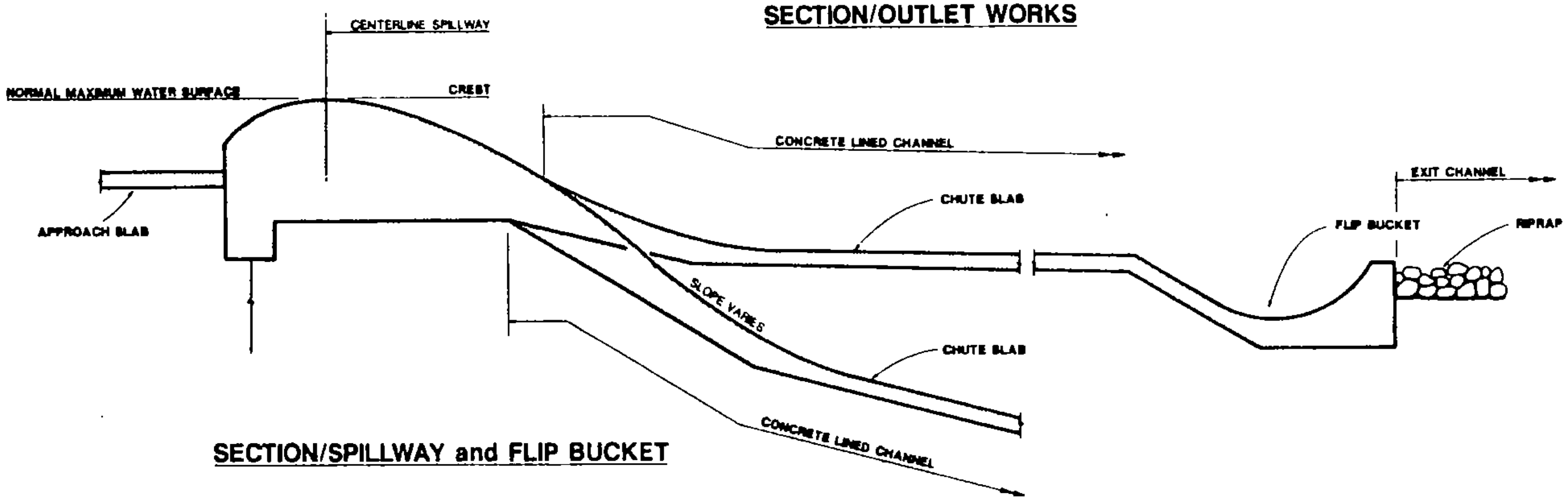




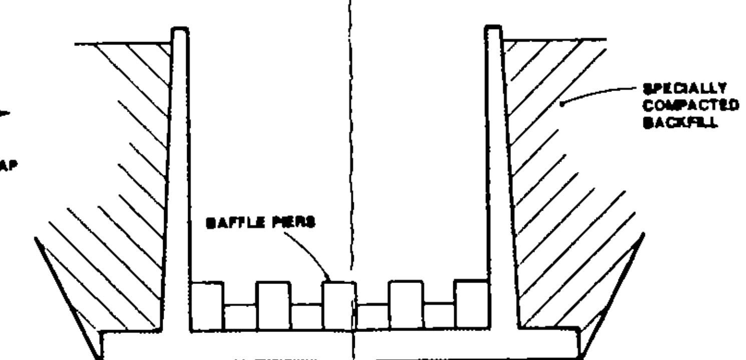
SECTION/OUTLET WORKS



SECTION A–A OUTLET CONDUIT



SECTION/SPILLWAY and FLIP BUCKET



SECTION B–B OUTLET STILLING BASIN

NO	DATE	REVISIONS	BY	CHK	APP'D

MORRISON–KNUDSEN ENGINEERS, INC.
2425 South Colorado Boulevard • Denver, Colorado • 303/757 8586

DESIGNED	DRAWN	CHK	RECOMMENDED
DATE			APPROVED

COLORADO WATER CONSERVATION BOARD

WHITE RIVER GEOTECHNICAL STUDY

OUTLETWORKS and SPILLWAY DETAILS

IECO NO

SHEET OF REV

Plate 9

consisting of drain holes both above and below the gallery and tunnel is also included.

The outlet works consist of an 8-foot diameter steel conduit embedded in conventional concrete in a trench under the dam. The intake to the conduit is through a tower attached to the upstream face of the dam with the inlet at the minimum pool elevation and protected by trash racks. Discharge from the outlet conduit would be controlled by two gate valves in the terminal structure at the downstream toe of the dams. A stilling basin is provided to regulate tailwater in the event a hydroelectric unit were included in the projects.

Spillways are located in the center of the dams and are designed to safely pass the probable maximum flood. Final design, however, might decrease the spillway capacity to something less than the PMF because, in the event of overtopping, the RCC dams would not fail. This could significantly reduce the quantities of RCC required. The crest, chute, and lateral walls of the spillway are of conventional concrete. The energy dissipator is of the flip bucket type constructed of conventional concrete founded on backfill RCC above bedrock.

8.3 Cost Estimates

Cost estimates were prepared considering about 20 items for dam construction in addition to land acquisition, relocations, recreational facilities and related items. Unit prices were estimated based on experience and recent projects of similar magnitude constructed in Colorado, adjusted to 1985 dollars.

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SELECTION OF SITES

9. SITE SELECTION

The approved scope of work for the study dictated that, in addition to Lake Avery, two damsites were to be selected for subsurface investigation in the upper canyon section of the study area between Buford and the Highland Ditch Headgate, and one below Powell Park, upstream of the confluence of Piceance Creek. Therefore one of the first tasks of the study was to select the sites to be studied. Surficial geologic mapping as described in Sections 3 and 4 was completed prior to the site evaluations and was considered in the selection process. It was decided to evaluate the sites over a range of reservoir capacities to assure that geotechnical data was available for the larger reservoir sizes. Earlier hydrologic and simulated reservoir operation studies indicated that mainstem reservoirs of 400,000 acre-feet capacity were the approximate economic optimum to provide near-total regulation of the White River Basin streamflows. It was therefore decided to study all the mainstem sites for potential capacities over the range of 50,000 to 400,000 acre-feet. Area-capacity and cost-capacity curves were made for each location and the sites were ranked according to apparent economic feasibility. The following is a description of the site selection process in each area, the economic ranking of the sites, and the conclusions regarding the sites to be investigated.

9.1 UPPER CANYON SITES

In previous studies of the White River by MKE, the USBR, and others, several potential damsites were identified along the mainstem of the White River upstream of Meeker, between Buford and the Highland Ditch Headgate. It was the objective of this study to select two of these sites for in-depth subsurface foundation investigation and construction materials evaluation. The potential damsites were located along an eight-mile stretch of the river where it cuts through a fairly narrow canyon. For the upper five miles, the river flows west, then turns northwest for the remaining three miles. Plate 1 shows the location of each of the potential dam axes.

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9.1.1 Warner Point

The site farthest upstream in the upper canyon section was named Warner Point. It is located about one mile upstream of a prominent local topographic feature of the same name. The exact location was chosen after comparing the height-capacity relationships of several similar potential alignments in the vicinity. The left abutment slopes about 4H to 1V toward the river and is a dip slope of the Maroon Formation, covered with colluvium and slide debris of unknown depth. Surface mapping did not confirm the existence of the massive deep slumping interpreted by previous studies. However, shallow landslides were mapped based on the absence of in-place bedrock exposures. The river channel is composed of alluvial sand and gravel overlying the Maroon Formation. On the right abutment, a colluvial slope about 75 feet high is fed by a series of vertical cliffs of alternating sandstone and claystone of the Maroon Formation. Barring the existence of deep slumping on the left abutment, the Warner Point site was considered to be geologically sound.

9.1.2 Choke Cherry Dam

The first site downstream of Warner Point was denoted Choke Cherry because the axis passes just downstream of the Choke Cherry Guest Ranch. The site is located about one mile downstream from Warner Point in an area where the river makes several sharp bends. The Choke Cherry site is completely within the Maroon Formation. The abutments are steep slopes formed by outcrops of bedrock, however a wide gently sloping bench occurs on the right abutment above elevation 6,800 that might affect the economic feasibility of the site. The base of both abutments is covered with colluvial debris originating from the outcrops above. The river bottom is composed of alluvial sand and gravel.

9.1.3 Dry Creek Dam

The next site downstream was called Dry Creek. It is located at a bend in the river about one-quarter mile downstream of the confluence of Dry Creek with the White River. The left abutment at the Dry Creek damsite is a steeply sloping ridge of resistant sandstone of the Maroon Formation. The right abutment is a broad alluvial fan leading to a gentle slope composed of siltstone of the Maroon Formation that produces a rolling hummocky topography. The economical maximum height at this site is restricted by the flatness of the right abutment. The river channel is wide at this point and its bed is composed of hard, rounded sand and gravel.

9.1.4 Veatch Gulch

The next potential site lies about three miles downstream of Dry Creek one-half mile to one-quarter of a mile upstream of the confluence of Veatch Gulch with the White River. The site was labeled Veatch Gulch.

The geology at the Veatch Gulch site, consists of massive sandstone cliffs of the Entrada Formation overlying reasonably competent siltstone of the Chinle Formation. If the dam of maximum height were to be constructed, the top of the dam would reach the basal sandstone of the Morrison Formation. The river flood plain is fairly wide and covered with sand and gravel overlying Chinle siltstone.

9.1.5 Canyon Dam

The site at the downstream end of the upper canyon section, located at the head gate for the Highland Ditch, was named Canyon. It lies about one-quarter mile downstream of Veatch Gulch.

The abutments at the Canyon site would be in competent Entrada sandstone, hard Morrison sandstone, and in weaker Morrison shales and sandstone. The river bottom is probably in Entrada and Morrison sandstone. The weak, flat slopes of the upper Morrison and the difference in strength between the Morrison and Entrada would probably preclude this as a site for an RCC dam. However, it might be considered acceptable for an earth dam if suitable impervious material could be located in the vicinity.

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9.1.6 Economic Comparison

Height-capacity curves were developed for each of the sites (Veatch Gulch sites were combined). A maximum storage volume of 400,000 acre-feet was used. Preliminary cost estimates were made for an RCC dam at each of the sites. The RCC section was chosen because of the fairly narrow canyon, the lack of side channel spillway locations, and the possible lack of low cost, impervious borrow material in the vicinity of the dams. The same dam section was used to compute quantities for three reservoir capacities at each site. A cost of $60 per cubic yard was assigned to the RCC for the preliminary estimates to include the dam and appurtenances. The cost estimates were not intended to be extremely accurate, but to provide a common base for economic comparison of the sites. Table 5 shows the ranking of the sites based on the economic analysis from least to most expensive for a 400,000 acre-foot reservoir:

TABLE 5

SITE SELECTION: Upper Canyon Sites--Cost Comparison

Ranking	Dam Height (Feet) for 400,000 ac-ft Reservoir	Cost/acre-foot ($)
1. Warner Point	329	$300
2. Canyon	345	$380
3. Veatch Gulch	348	$405
4. Choke Cherry	369	$535
5. Dry Creek	369	$658

9.1.7 Conclusion

At the time the preliminary comparison was made, prior to subsurface investigation, all the damsites except Canyon were considered suitable for an RCC dam.

Because of the economic ranking and the doubts surrounding the foundation at the Canyon site, it was recommended, and concurred with by the CWCB, that the subsurface phase of the investigation would be focused on the Warner Point and Veatch Gulch sites. It was proposed that the question of possible landslides on the left abutment at Warner Point be resolved by drilling the first hole in that vicinity. If the depth to undisturbed bedrock was considered excessive, the remainder of the investigation would shift to Choke Cherry.

Subsequent drilling at Warner Point revealed a thicknes of alluvial gravel in the river floodplain in excess of 70 feet. In addition, thin, weak layers of weathered siltstone were found to be interbedded with the sound sandstone bedrock of the Maroon Formation in the left abutment. Both of these discoveries cast doubt on the feasibility of an RCC gravity dam at the site. Furthermore the depth of alluvial material might preclude RCC at all the sites in the upper canyon section and therefore invalidate the economic comparison based on an RCC design.

It was decided, in conjunction with the CWCB, that instead of investigating two sites in the upper canyon area, study funds at this level of investigation could be best expended by obtaining subsurface information on as many of the sites as possible. Therefore, the focus of the study was modified. In the upper canyon section only the Dry Creek site was eliminated and subsurface investigations were carried out at Warner Point, Choke Cherry, Veatch Gulch and Canyon.

9.2 POWELL PARK SITES

In the previous studies of the Powell Park site, several dam axes had been examined along an eight-mile reach of the river between Powell Park and Piceance Creek. For the White River Geotechnical Study, a preliminary screening was made of three sites. These were compared based on surficial geologic mapping, reservoir storage characteristics, and approximate embankment costs.

Plate 1 shows the location of the three damsites. The upstream site, located at the downstream end of Powell Park, is called Powell Park No. 1. The middle site, 2.5 miles downstream, was termed Powell Park No. 2. The lower site, located 2.5 miles farther downstream, was named Smith Gulch as it was near that side tributary.

9.2.1 Powell Park No. 1.

The Powell Park No. 1 site is the dam location described in the White River Study of December, 1983. During that study one drill hole was located about 1.5 miles downstream to determine the depth of alluvial material in the valley floor. The site is characterized by a near vertical cliff on the left abutment that rises about 200 feet above the river to approximately the crest elevation of the maximum height dam. The slope is formed by moderately weathered sandstone and shale of the Lower Member of the Green River Formation. The left abutment is capped by terrace gravels and colluvium. The river floodplain is nearly flat and is composed of alluvial sand and gravel and is underlain by variegated claystone and sandstone of the Wasatch Formation. The right abutment slopes at about 1H to 1V and is characterized by benched topography caused by differential weathering of the hard and soft rocks of the Lower Green River Member. Alluvial fans consisting of fine sand emanate from drainages perpendicular to the White River. The fans bury the base of the bedrock outcrops on the right side and extend out over the river alluvium.

9.2.2 Powell Park No. 2

Powell Park No. 2 site, which has had no subsurface exploration, is on the alignment of the Occidental Petroleum storage water right filing. This alignment was based on preliminary designs performed for a reservoir of 75,970 acre-feet. The geology at the site is identical to Powell Park No. 1 except that the left vertical abutment rises only about 100 feet and the encroachment of alluvial fan material on the right abutment is not as extensive.

9.2.2 Smith Gulch

The Smith Gulch site was selected to examine an alternative located downstream of the previously studied sites. A lower site would be desirable because the lower reservoir level would result in less inundation of agricultural land in Powell Park. The geology is similar to the upper two sites except the left abutment is less well defined and the right abutment is a massive landslide.

9.2.4 Economic Comparison

USGS topographic maps were used to construct dam height versus storage capacity curves. An earth dam was used to make approximate embankment volume computations for each size at each site. An arbitrary unit price of embankment of $15 per cubic yard was used to compute an approximate total embankment cost. No separate designs or cost estimates were made for spillways, outlet works or other features. Cost versus storage capacity comparisons showed the Powell Park No. 1 site to be lower in cost throughout the entire range of reservoir sizes. At the 200,000 acre-foot size, Powell Park No. 1 was approximately one-half the cost of Powell Park No. 2 and about one-third the cost of a dam at the Smith Gulch site. Table 6 is a ranking of the three sites based on estimated cost per acre-foot of storage and indicates the height of dam necessary to impound a reservoir of 400,000 acre-feet.

9.2.5 Conclusions

There is no major geologic difference between the Powell Park No. 1 and Powell Park No. 2 sites. The Powell Park No. 1 site appeared to be significantly lower in cost than the other two sites to the degree that other factors such as foundation conditions, spillway requirements, or outlet works would not result in cost changes of such magnitude to reverse the relative cost position. A potential disadvantage of the Powell Park No. 1 site would be greater inundation of agricultural lands in the broad Powell Park valley just downstream of Meeker. It was recommended, and agreed by CWCB, that detailed subsurface exploration would be carried out at the Powell Park No. 1 site.

TABLE 6

SITE SELECTION: Powell Park Sites--Cost Comparison

Dam site	Dam Height (Feet) for 300,000 acre-foot reservoir	Cost/acre-foot ($)
Powell Park No. 1	191	$178
Powell Park No. 2	230	$343
Smith Gulch	242	$543

SITE INVESTIGATIONS

10. SITE INVESTIGATIONS

10.1 LAKE AVERY DAM

10.1.1 History

Lake Avery Dam was constructed by the Colorado Division of Wildlife in 1961. Lake Avery, also known as Big Beaver Reservoir, is located on Big Beaver Creek, a tributary to the White River, about three miles downstream from Buford. The existing dam is 80 feet high and impounds a reservoir of approximately 6,000 acre-feet, with a spillway elevation of 6985. The reservoir was built solely for recreational purposes although an outlet was included during construction for agricultural releases. The possibility of raising the dam to provide active storage for agricultural, industrial, and municipal use was first studied by the USBR in 1970 in their study of the Upper Colorado River Basin. It was concluded that economical storage could be provided and, in 1972, the USBR drilled four boreholes along the centerline of the proposed dam. One hole was drilled on the right abutment, one just downstream from the existing dam, and two on the left abutment. The two holes on the left abutment showed a thickness of pervious gravel and boulders in excess of 100 feet. The top of bedrock on that abutment was shown to be only 10 feet above the existing reservoir's normal maximum water surface elevation. Based on these findings the USBR determined the dam raising to be infeasible since a positive cutoff in excess of 100 feet deep would have to be constructed.

In 1982, during the Yellow Jacket study by IECO, geologic mapping in the area of the left abutment raised hopes that a narrow ridge of bedrock might exist at a higher elevation at the southern or downstream edge of the abutment. A seismic refraction survey was conducted of the left abutment, which showed that the gravel was at least 100 feet thick over the entire area extending 5,000 feet northeast of the dam, and actually thickened to more than 180 feet. The seismic refraction study did, however, show a mantle of fine soil of variable thickness that might possibly serve as an impervious blanket overlying the gravel. The appraisal level studies prepared for the Yellow Jacket Study called for raising the existing dam and blanketing the left abutment. This approach was considered feasible, for reservoir capacities up to 64,000 acre-feet. Water would be delivered to Lake Avery through a gravity pipeline from the North Fork of White River, with the intake located about four miles upstream.

The White River study by IECO in 1983 included an enlarged Lake Avery as a potential upstream storage site to be used in conjunction with mainstem regulation and storage downstream. No further geotechnical evaluations were done in conjunction with that study.

10.1.2 Reservoir Characteristics

Drawings for the enlarged Lake Avery for additional storage capacities of 20,000 acre-feet, 40,000 acre-feet, and 60,000 acre-feet as well as the intake pipeline are shown on Plate 10. It was assumed that the existing Lake Avery capacity would be maintained as a minimum pool and not available for useable active storage in any enlargement project. The largest reservoir would inundate approximately 235 acres of the existing reservoir and about 220 acres of irrigable bottom land along Big Beaver Creek. The remaining 285 acres of land flooded by the new reservoir would be gently rolling, sage-covered terrain adjacent to the present Lake Avery and steeper sage-covered and rock-strewn slopes upstream in the narrower valley. Most of the land impacted belongs to the State of Colorado.

Because the present Lake Avery is maintained as a recreation facility by the Division of Wildlife, the reservoir is kept as full as possible at all times. In the event a new reservoir were built, the purpose would be to provide storage of spring flows for release in late summer during dry years. This type of operation of the reservoir would cause the water surface to fluctuate significantly during the year.

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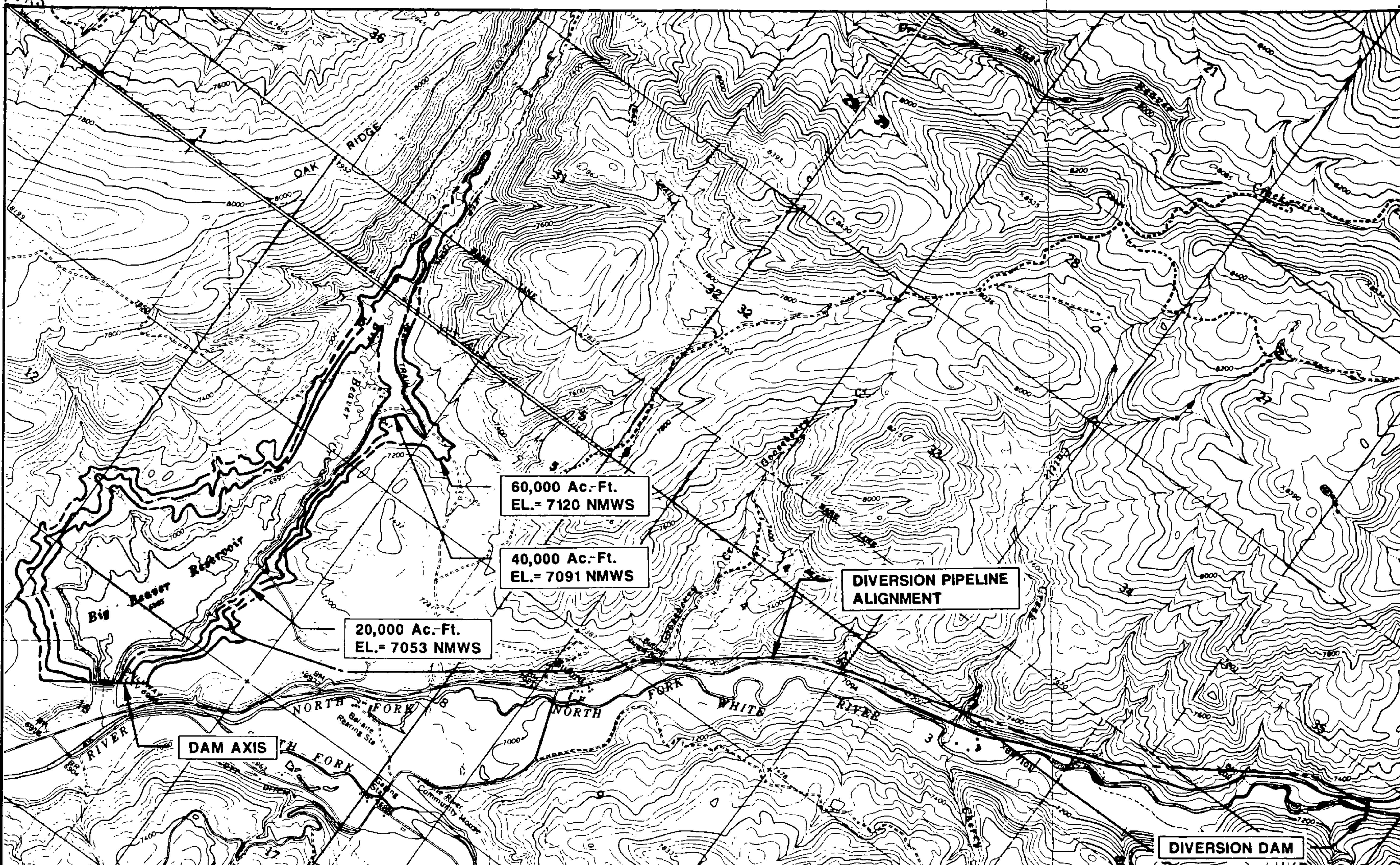

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WHITE RIVER
GEOTECHNICAL STUDY
RESERVOIR AREAS
LAKE AVERY DAM

IECO NO
SHEET OF REV.
Plate 10

10.1.3 Site Geology

Lake Avery and the proposed intake pipeline are located near the southwest limb of the Yellow Jacket anticline on the northeast edge of the White River Uplift. Geologic mapping of the area is shown in Plates 4 and 5, and of the damsite in Plate 11. The oldest exposed rocks are of the Minturn-Eagle Valley sequence. The interbedded mudstones, siltstones, and sandstones of the Maroon Formation and Weber Sandstone crop out along portions of the Lake Avery shoreline, on the ridges northwest of the lake, and along the White River Valley. Small drainages and steep, unstable hillslopes are underlain by less resistant mudstones and siltstones of the Maroon Formation. Dissected ridges west of Lake Avery are defined by shallow dip slopes on one side and steeper, more irregular and unstable slopes on the other.

Strike and dip measurements taken on bedding planes of the Weber and Maroon Formations are in agreement with the regional pattern, with beds striking northeast and dipping to the northwest at 15^o to 30^o angles. Several measurements taken on resistant beds west and northwest of Lake Avery, however, show dips to the southeast and southwest. When plotted, these variations in dip direction reveal two relatively tight folds within the mapped area. The largest fold is an anticline plunging to the northeast near Lake Avery, gradually changing to a north-north-east trend up Big Beaver Creek. A smaller synclinal fold with a similar direction of plunge exists on the southeast limb of the anticline. The anticlinal and synclinal axes appear to converge near the northwest boundary of the map area. No evidence was found to justify faulting as a mechanism for development of the geologic setting in this area.

A small remnant of a basalt flow is present northeast of Lake Avery. The extent of the dense, dark brown basalt of Miocene-Pliocene age is defined by small, highly weathered outcrops, an abundance of basalt debris on hillslopes, and the presence of a dark brown finely-textured soil.

Locally interbedded with the basalt is the Miocene-age Brown's Park Formation. Outcrops of this gray, fine-to medium-grained, well-sorted, loosely-consolidated, fluvial sandstone and siltstone are found in several places along the Lake Avery shoreline. Outcrops show low-angle cross bedding, with individual beds generally less than two inches thick. Strike and dip measurements taken on bedding planes reveal north-northwest strikes and northwest and southwest dips at 50 to 70 degrees. In this area, lack of sufficient outcrop exposure precludes identification of any structural trends through this formation. The strike and dip of cross-beds mentioned above indicate a westerly direction of streamflow during deposition.

The contact of Brown's Park Formation with adjacent formations is poorly defined. A noticeable break in slope above the shoreline is a reasonable estimate of the contact with overlying terrace gravels, since unconsolidated terrace gravels would tend to erode more quickly. Although the contact with basalt is concealed, the Brown's Park Formation probably underlies or is interbedded with the lower part of the basalt, based on their age relationship. This fluval sandstone and siltstone lies unconformably over less resistant beds of the Maroon Formation.

Deposits of Quaternary age cover the study area extensively. Aerial photographs reveal three distinct fluvial terraces, excluding the modern White River floodplain. These deposits consist mainly of cobble-to sand-sized rounded, poorly consolidated clasts of multiformational origin covering bedrock to varying depths. Although no age-determination techniques have been applied to these deposits, material found in most mountain valley stream terraces is derived from erosion of Pleistocene glacial moraines.

The most extensive fluvial terraces (Qt) in the map area increase in thickness to the northeast from the left abutment of Lake Avery Dam, and to the northwest from the right abutment. The terraces eventually wedge out against bedrock formations. Maximum thickness is approximately 200 feet. The top of the terraces are characterized by slightly uneven terrain, with gravels exposed only at the margins. Auger-hole drilling, test pit excavations and seismic refraction revealed a 10 to 20-foot layer of silty sand overlying the gravel, which gradually thins out near the margins. The porous texture of the clayey material, lack of well-defined stratification, and the variation in thickness indicates that the deposit is of eolian origin.

Successively younger terraces are found at lower elevations in the map area. The step-like appearance of terraces is explained by partial obliteration of higher terraces by a meandering river, with deposition of gravels preceding a period of more active stream downcutting and deposition of younger terraces.

Other Quaternary-age deposits in the area include colluvium, landslides, and modern stream alluvium. Colluvium, consisting of angular fragments of all sizes and composition, forms poorly sorted, unconsolidated deposits at the bases of slopes and is transported by slope-wash and slope-creep processes. Although colluvium is found nearly everywhere, thick deposits forming distinct landforms occur only along the paved highway and at the bases of steeper slopes of ridges west of Lake Avery. In the area used for borrow material during construction of the existing dam, a prominent north-facing scarp partially outlines a landslide.

10.1.4 Foundation Investigation

The objective of the subsurface investigation at Lake Avery was to confirm the thickness and character of the soil mantle overlying the terrace gravels on the left abutment as detected by the seismic refraction work during the Yellow Jacket Project. The zone was to be evaluated as potential foundation for an earth dam and as a source for impervious construction material. The saddle area on the right abutment was to be investigated as being a possible gravel filled channel, and as a potential spillway location for the reservoir enlargement.

A 500-foot exploration grid was laid out on the left abutment extending 4,500 feet northeast of the damsite. A total of ten auger holes and ten cased rotary/percussion holes were drilled, augmented by ten backhoe excavated test pits. Plate 11 shows the locations of the drill holes and test pits.

Generally the entire area is covered with a stiff brown clay of low to moderate plasticity. Thickness ranges from less than two feet at the upstream side of the left abutment to 25 feet in the northeast corner of the investigation area. East of a point about 3,000 feet northeast of the existing dam, the clay is underlain by up to 15 feet of stiff, tan, silty clay that visually displays a porous structure. Both the clay and silt overlie the alluvial terrace gravel. Sections illustrating the relationship of soil units, terrace gravel and bedrock as detected by the present subsurface exploration and previous investigations for the Yellow Jacket Project and by the USBR are shown in Plates 12a through 12e.

"In-situ" penetration tests and permeability tests showed the materials to be of adequate strength and of low permeability as a foundation for an embankment dam. However, visual observation in the test pits often indicated the soils to be of low in-place density due to a system of small voids and lack of consolidation. This was especially true in the case of the clayey silt deposits.

Results of the foundation testing are contained in Appendix E and summarized in Table 7.

A terrace deposit that occurs in the low saddle on the right abutment just to the west of the existing dam was investigated with four hand auger holes, one manual test pit, and two crossing lines of seismic refraction. They revealed a thickness of ranging from five to 15 feet of reddish-brown clay characterized by a seismic velocity of less than 2,000 feet per second (fps), overlying sand and gravel with a velocity of 2,300 to 3,200 fps. The gravel overlies bedrock of the Maroon Formation to a depth as great as 62 feet. The bedrock has a velocity of 6,000 to 13,300 fps. The gravel bedrock contact rises sharply at the downstream edge of the abutment where it is mapped at approximately elevation 7140.

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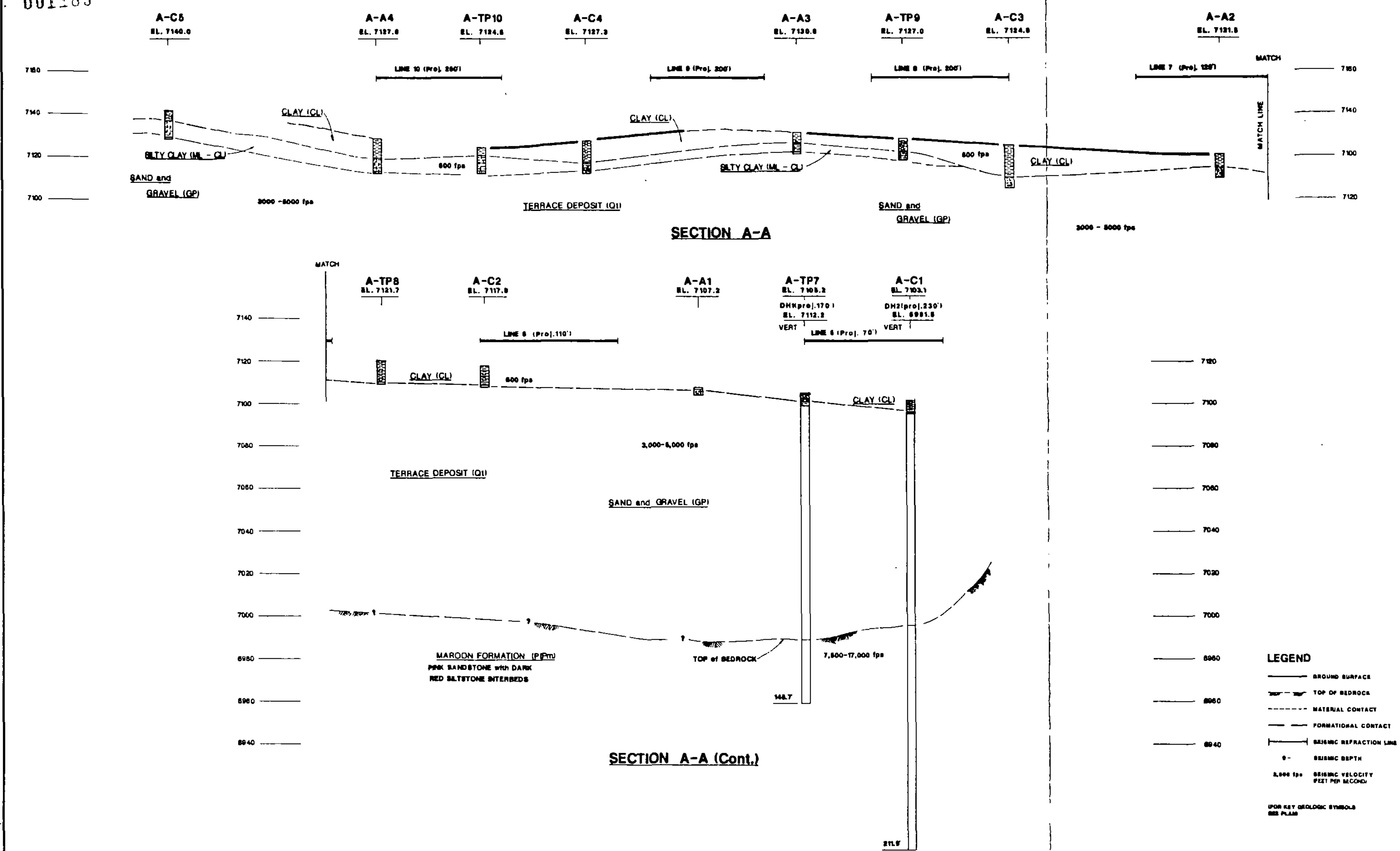

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WHITE RIVER GEOTECHNICAL STUDY

GEOLOGIC SECTIONS
LAKE AVERY DAM

SHEET OF REV

Plate 12A

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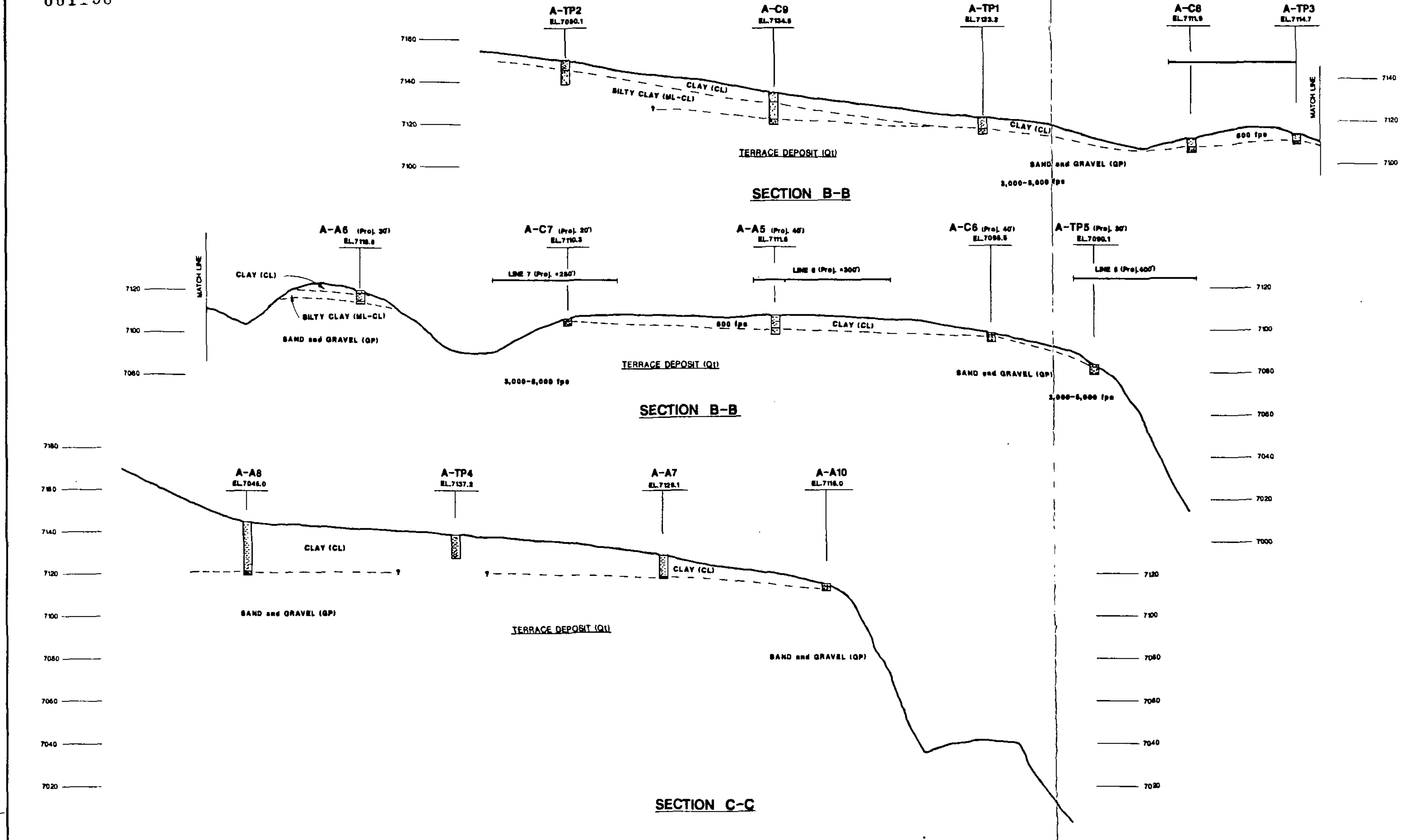

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WHITE RIVER GEOTECHNICAL STUDY

GEOLOGIC SECTIONS

LAKE AVERY DAM

SHEET OF REV

Plate 12B

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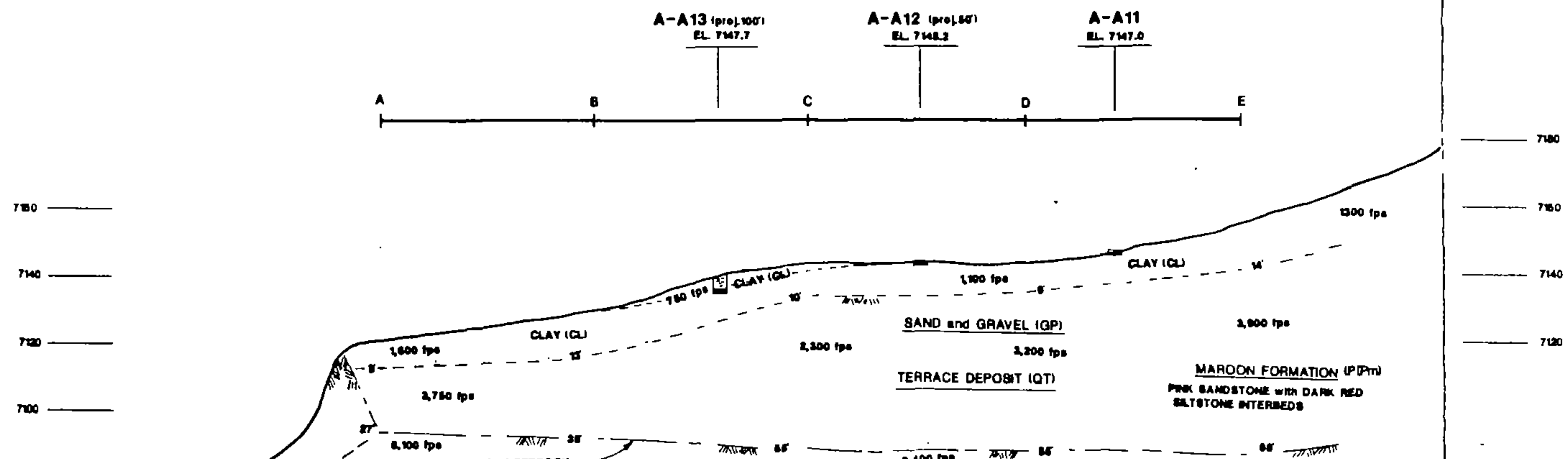
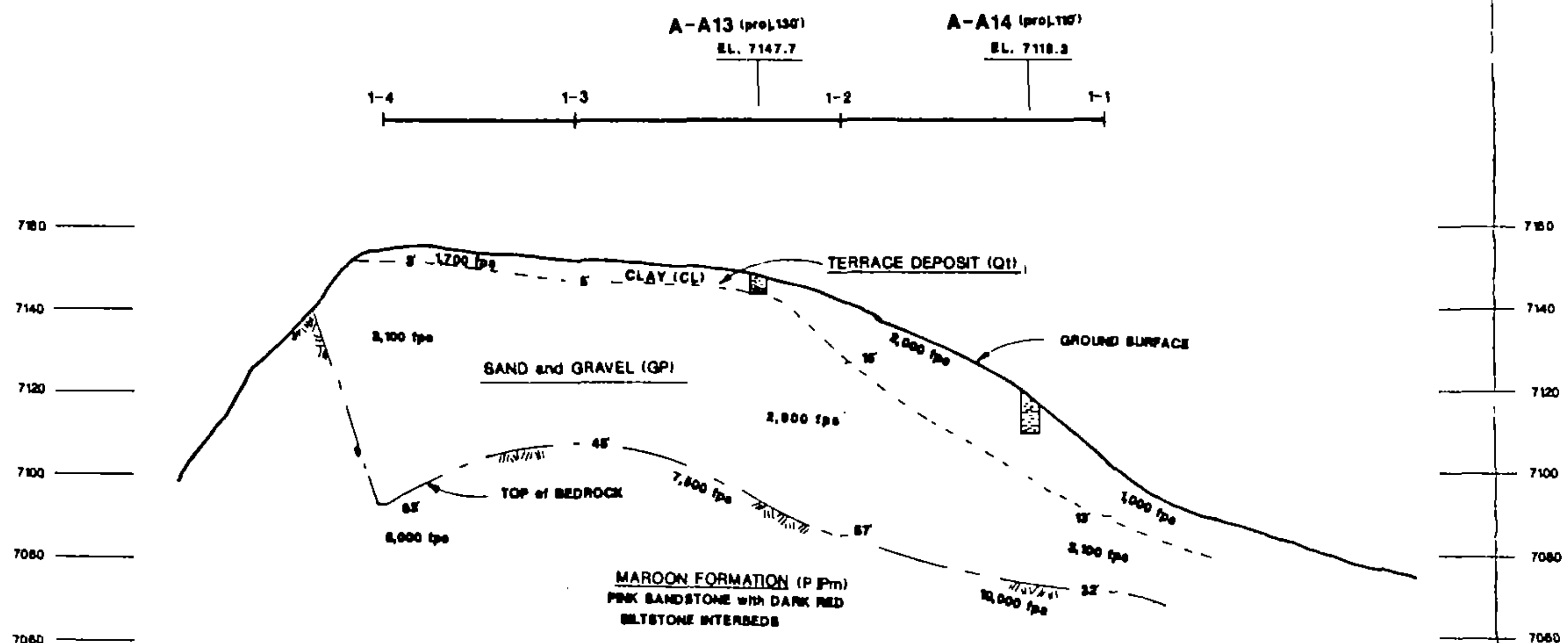


SECTION D-D



SECTION E-E

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WHITE RIVER GEOTECHNICAL STUDY
GEOLOGIC SECTIONS
LAKE AVERY DAM

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SHEET OF REV
Plate 12C

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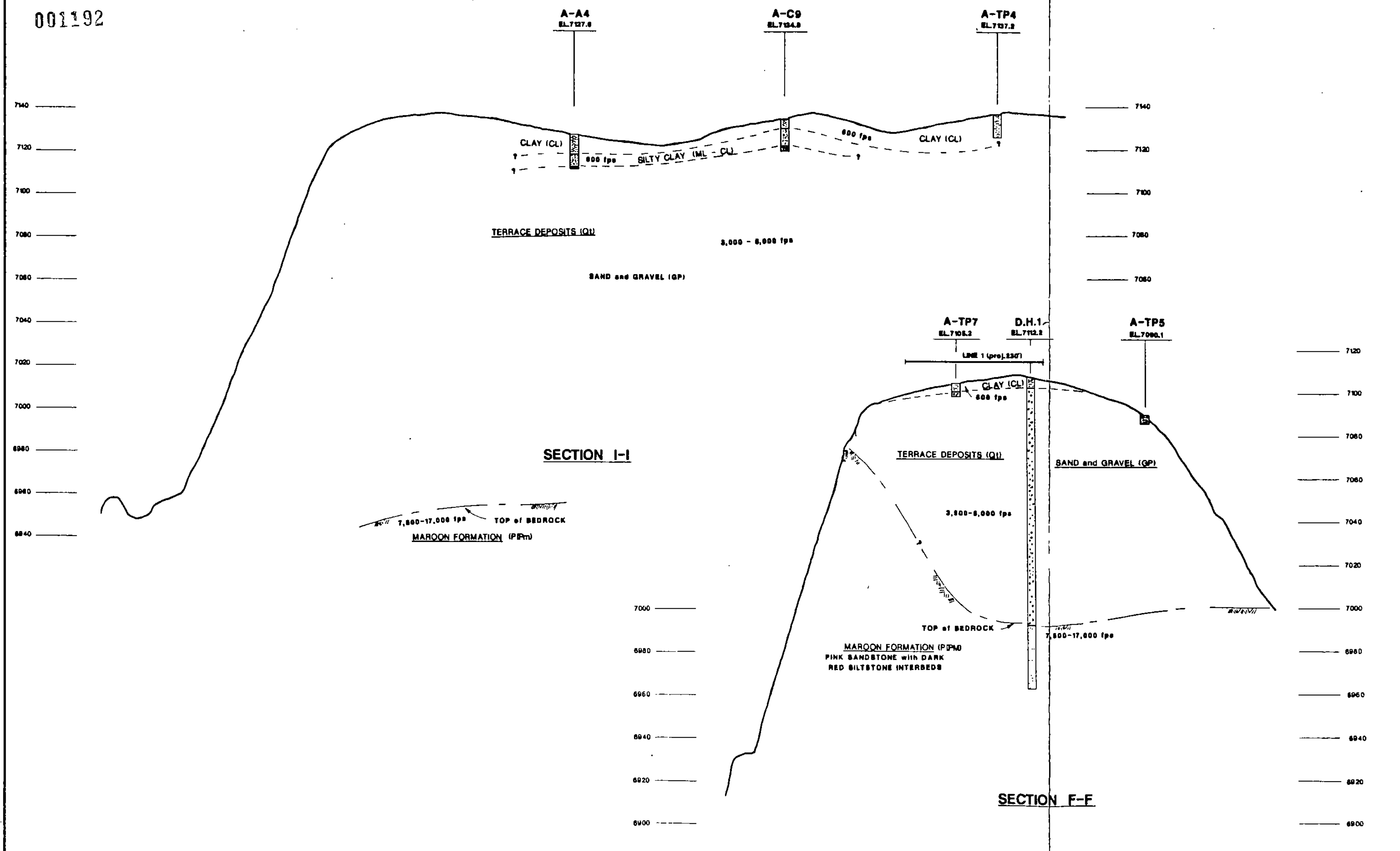

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GEOLOGIC SECTIONS
LAKE AVERY DAM

ECO NO.
SHEET OF REV
Plate 12D

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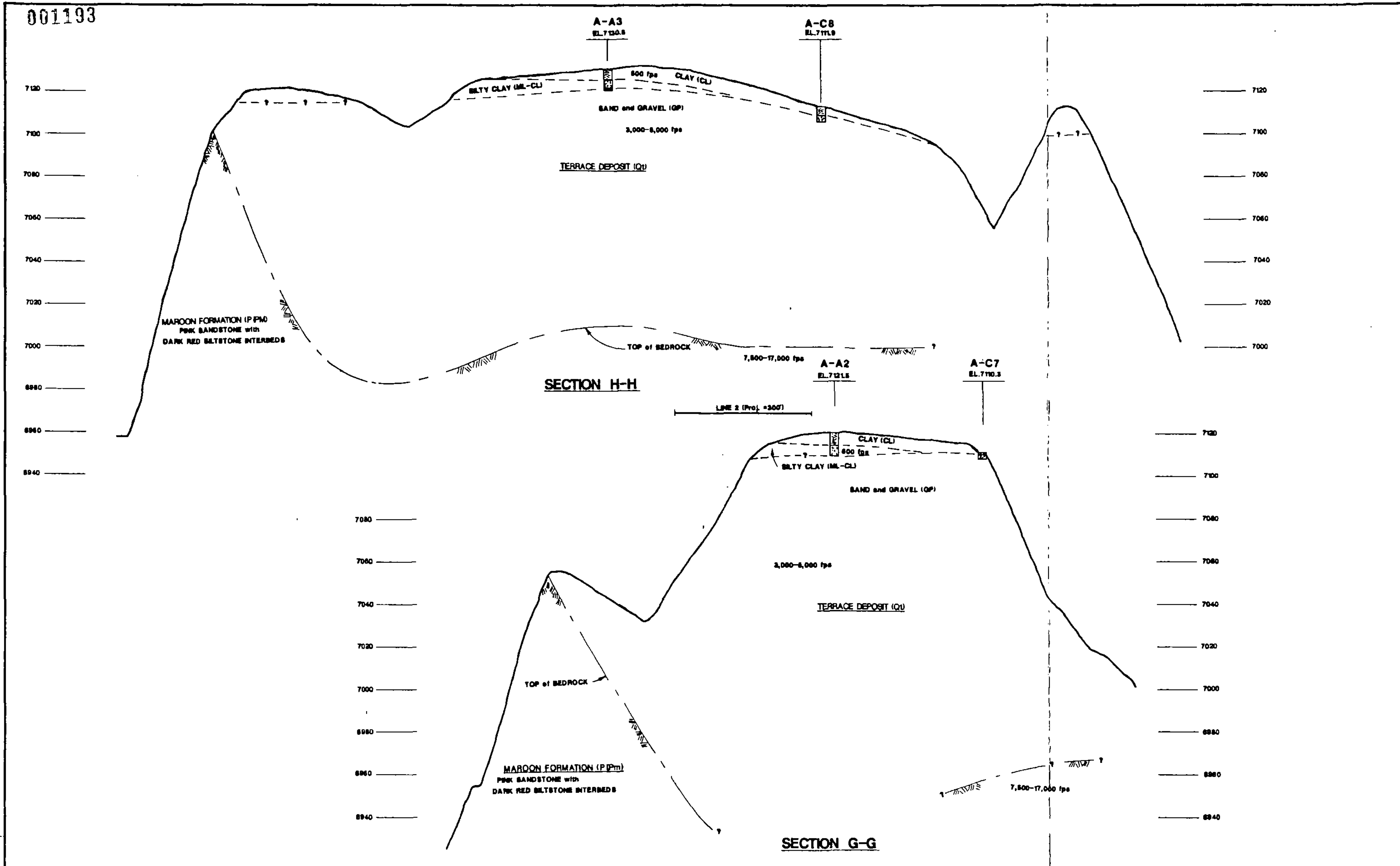

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WHITE RIVER GEOTECHNICAL STUDY
GEOLOGIC SECTIONS
LAKE AVERY DAM

IECO NO.
SHEET OF REV
Plate 12E

TABLE 7

LAKE AVERY: Foundation Test Results

	DESCRIPTION		
Test Result	Clay (CL)	Silt (ML)	Sandy Gravel (GP)
Plasticity Index	17	7	NP
"In Situ" Density	108 pcf	103 pcf	No Test
"In Situ" Moisture	18%	20%	No Test
Shear Strength	C=12 psi, $\phi=29^{\circ}$	C=5 psi, $\phi=15^{\circ}$	C=0, $\phi=40^{\circ}$
Permeability	3.2×10^{-6} cm/sec	2.2×10^{-6} cm/sec	1×10^{-2} cm/sec

10.1.5 Construction Materials Investigation

The subsurface investigation revealed that construction materials are readily available near the damsite. The clean sandy gravels on the left side of the reservoir occur in abundance and are of sufficient quality and quantity to serve as embankment shell, and after processing as concrete aggregate, filter and drain material. The clays and silty clays are suitable for embankment core and impervious blanket material and are available in adequate volumes on both sides of the reservoir. The area along the existing access road on the left abutment is the most uniform source for these materials.

Laboratory testing was conducted on both the silty clays and clays to determine their physical characteristics and confirm their suitability as construction materials. The test results are contained in appendix E and are summarized in Table 8.

TABLE 8

LAKE AVERY: Construction Materials Test Results

	DESCRIPTION		
Test Result	Clay (CL)	Silty Clay (ML-CL)	Sandy Gravel (GP)
Plastic Index	17	7	NP
Optimum Moisture	20%	19%	N/A
Optimum Density	108 pcf	105 pcf	145 pcf
Shear Strength	C=12 psi, $\emptyset=29^{0}$	C=5 psi, $\emptyset=15^{0}$	C=0, $0=40^{0}$
Permeability	3.2×10^{-6} cm/sec	2.2×10^{-6} cm/sec	No Test

10.1.6 Engineering Geology

The foundation bedrock at Lake Avery is the Maroon Formation consisting of sound layers of resistant red-to-pink micaceous sandstone separated by thin interbeds of weaker dark red claystone and mudstone. The beds dip approximately 15 degrees northwest. The bedrock will be the foundation for the raised dam on the entire right abutment and up to elevation 7010 on the left abutment, 10 feet above the existing embankment. The rock mass as a whole is sound, and the only concern in terms of stability would be the shear strength of the relatively weaker interbeds. This is not considered a problem, however, due to the low unit stresses imparted by the relatively low embankment dam. The principal set of regional vertical joints striking to the northwest cut the foundation in an upstream downstream direction. A grout curtain will be necessary to cut off seepage flow through these joints as well as through the bedding plane joints.

The design outlined in Section 10.1.7 and as shown in Plates 13 and 14 calls for blanketing the upstream side of the left abutment with impervious material. The blanket would have to be tied into the top of relatively impervious bedrock. The Maroon Formation can be traced along the shoreline of the left abutment for a distance of about 2000 feet, at which point it either dips below the normal reservoir surface or is covered with colluvial gravel and soil from above. The bedrock in the form of Brown's Park Formation sandstone can be detected again about 2,000 feet further along the shoreline from where it disappeared. Determination of the top of bedrock and the permeability of the bedrock along the left abutment contact with the impervious blanket should be a priority of any future investigative effort.

Geology does not directly affect the engineering aspects of the remainder of the design. A positive cutoff key to bedrock would have to be provided through the alluvial gravel in the valley floor. A grout curtain would probably be necessary in all the areas where the impervious core is in contact with bedrock. The cutoff trench need only be excavated about 10 feet into the exposed bedrock of the abutments. The foundation for the spillway and chute located in the saddle on the right abutment would be in sound sandstone. The slopes of the new reservoir, like the present reservoir, would be so gentle that no slope stability problems are anticipated.

10.1.7 Design and Cost Estimate

Based on the foundation and materials information gathered from the geotechnical investigation, appraisal-level designs and cost estimates were prepared for replacing Lake Avery Dam with new facilities having storage capacities of 20,000 acre-feet, 40,000 acre-feet and 60,000 acre-feet. The new dam would be of the embankment type. Seepage through the pervious alluvial terrace gravel deposits on the left abutment would be controlled by impervious blanketing of the upstream slope of that abutment. The cost estimates also include diversions and conveyance facilites for transporting water from the White River to the new reservoir.

The dam and blanket designs are shown in plan on Plate 13 and in section on Plate 14. Construction drawings of the existing Lake Avery Dam are contained in Appendix I. The three sizes of embankment would have crest elevations, heights above present creek bed, and normal maximum reservoir surfaces as shown in Table 9.

TABLE 9

LAKE AVERY: Embankment Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)	NMWS Elevation (ft msl)
20,000	7065	145	7053
40,000	7103	183	7091
60,000	7130	210	7120

Foundation preparation would consist of partial or complete removal of the existing embankment dam, spillway, and sheet pile cutoff wall. The estimates prepared for this report assumed complete removal. A positive cutoff trench would be excavated approximately 50 feet deep to bedrock through the alluvial sand and gravel of Big Beaver Creek. Excavation in the bedrock portion of the abutments would be limited to a 10-foot-deep cutoff trench in the area of core contact. The upstream and downstream shells would be founded on in-place alluvial gravel and weathered bedrock. Grouting of the fractured bedrock would be necessary in all dam foundations where bedrock is exposed in the bottom of the core trench. The highest dam alternative would require embankment on the left abutment to be founded on terrace deposits, and the upper mantle of potentially collapsible transported material would be removed to the top of the pervious gravel. The foundation for the impervious upstream blanket would be prepared by trimming and shaping the natural upstream slopes depending on the topography and the requirements for gravel in the shell of

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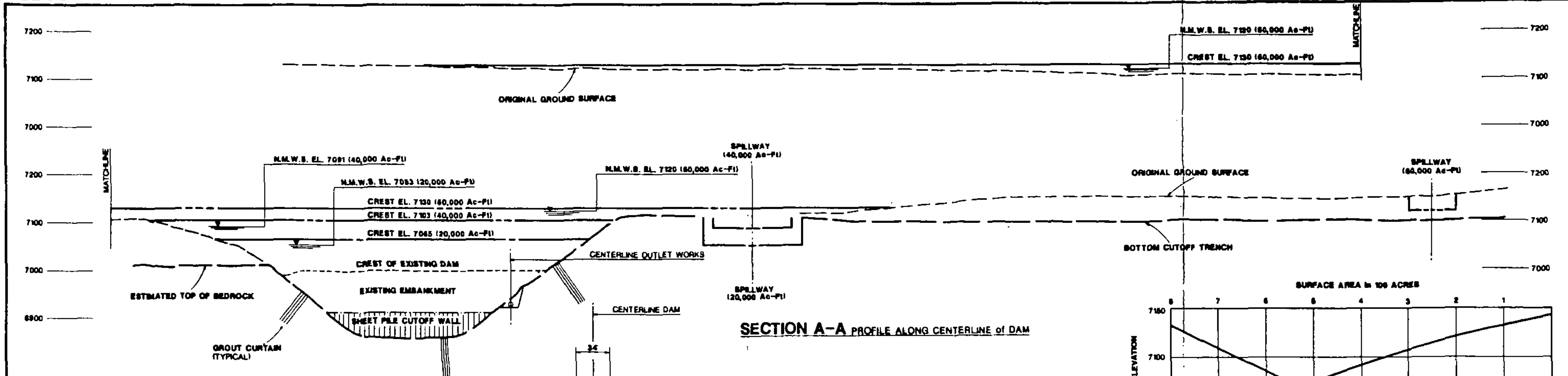
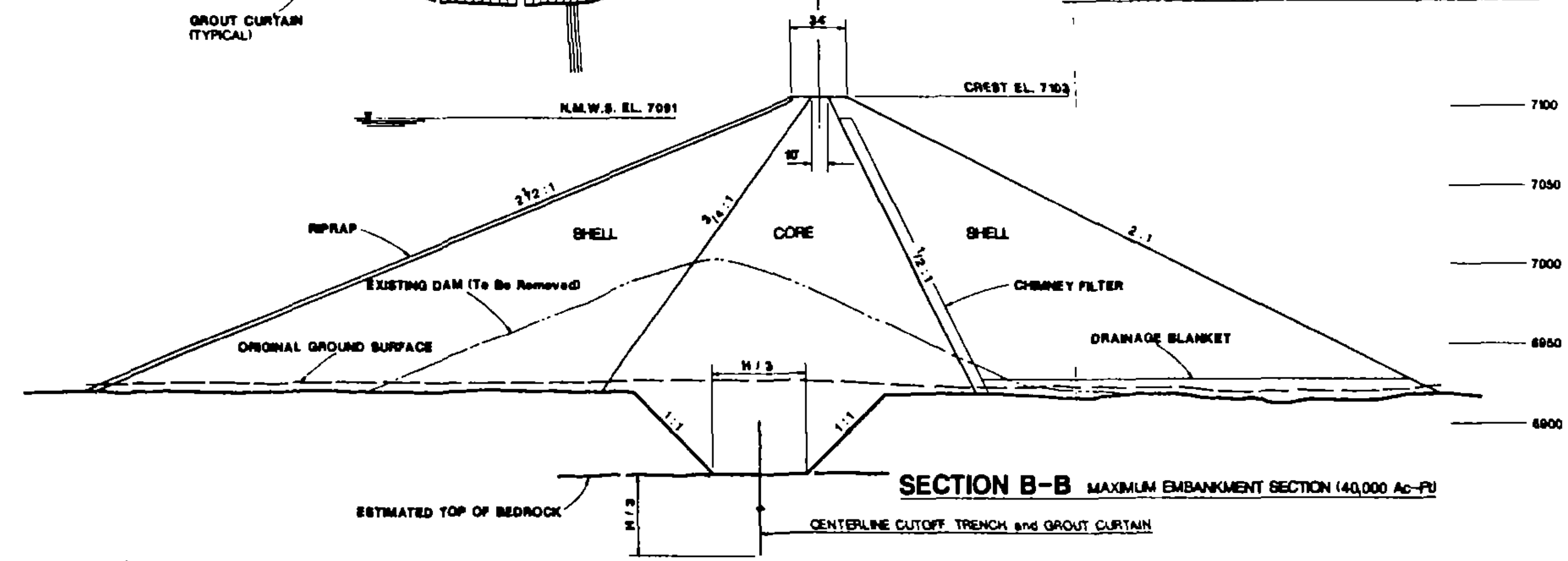
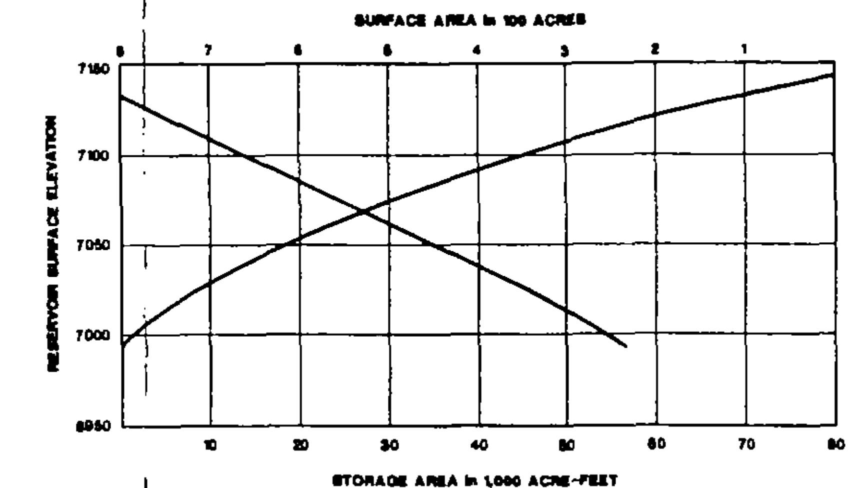


SECTION A-A PROFILE ALONG CENTERLINE of DAM



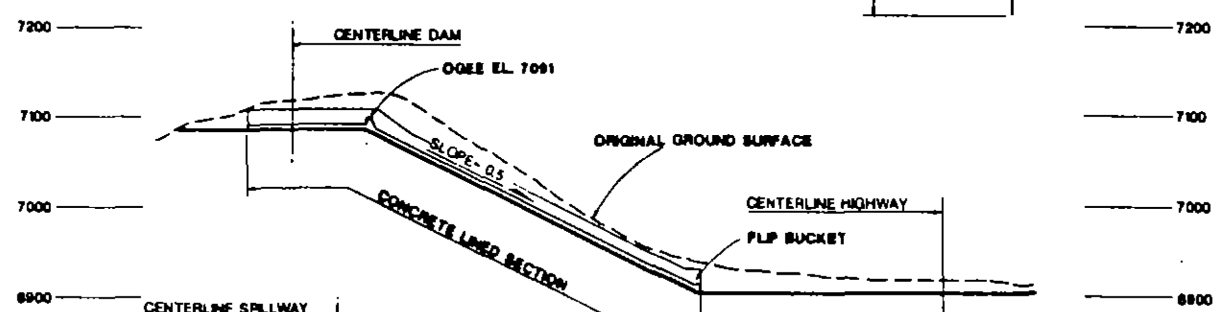
SECTION B-B MAXIMUM EMBANKMENT SECTION (40,000 Ac-Ft)



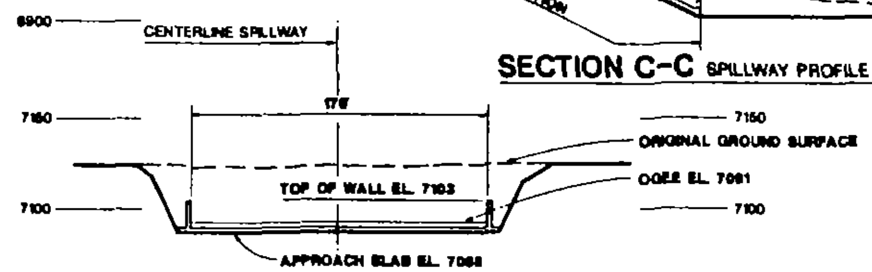
RESERVOIR AREA/CAPACITY

TABLE of ESTIMATED QUANTITIES

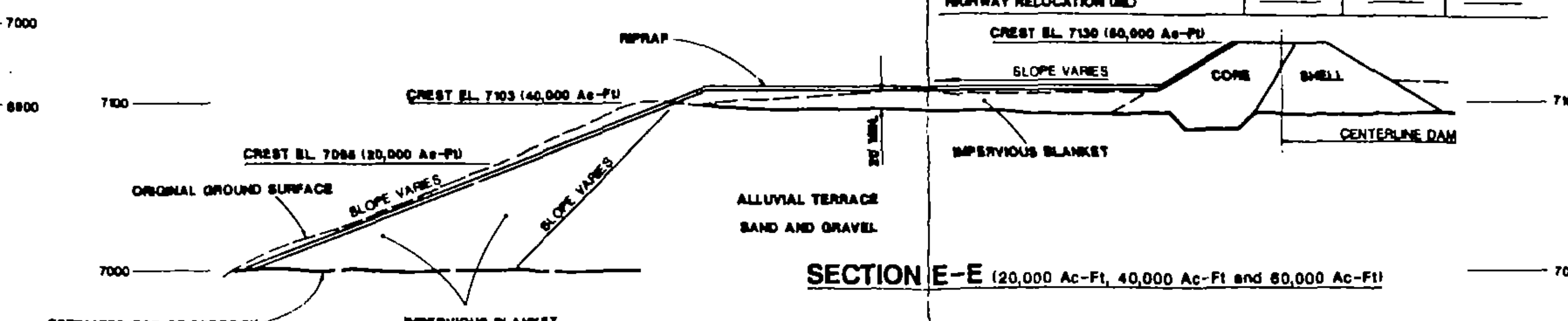
ITEM	RESERVOIR SIZE in ACRE FEET		
	20,000	40,000	60,000
COMMON EXCAVATION (Cu. Yd.)	1,343,890	1,779,879	2,498,678
ROCK EXCAVATION (Cu. Yd.)	223,893	134,118	148,572
CORE (Cu. Yd.)	968,264	1,957,895	3,305,498
SHELL (Cu. Yd.)	715,883	1,378,406	1,854,743
RIPRAP (Cu. Yd.)	84,381	130,817	326,248
GROUTING (Bk.)	1,722	2,198	2,557
STRUCTURAL CONCRETE (Cu. Yd.)	6,410	7,800	8,895
LAND ACQUISITION (Ac.)	509	664	782
HIGHWAY RELOCATION (Mi.)	—	—	—



SECTION C-C SPILLWAY PROFILE (40,000 Ac-Ft)



SECTION D-D SPILLWAY CREST (40,000 Ac-Ft)



SECTION E-E (20,000 Ac-Ft, 40,000 Ac-Ft and 60,000 Ac-Ft)

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WHITE RIVER GEOTECHNICAL STUDY
EMBANKMENT DAM/SECTIONS
LAKE AVERY DAM

ECO NO
SHEET OF REV
Plate 14

the dam. For the present design the upstream slope was cut at a 1H to 1V slope to provide sufficient material for the dam and to provide a wide contact zone for the impervious blanket with the bedrock.

The embankment dam would be a zoned earthfill conforming to the design outlined in Section 8.1. The impervious core material could be derived completely from borrow sources in the terrace deposits on either abutment as described in Section 10.1.5. The shell zones would be constructed of rock fill and gravel derived from required excavation of the dam, blanket foundation, and spillway chute. The impervious blanket would have a variable slope depending upon the natural terrain but for the present design was considered to have an upstream slope of 3H to 1V. For the highest alternative, where the blanket would cover the horizontal top of the left abutment and tie into the in-place impervious cap, the minimum thickness of blanket was estimated at 10 feet. The upstream slope of the blanket and horizontal blanket would be protected by riprap derived from rock excavated for the spillway chute.

The outlet and spillway designs are described in Section 8.1. Typical details of the designs are shown in Plate 9. For each of the three alternative enlargements the spillway is cut into the right abutment. The approach channel and chute downstream of the ogee weir would be concrete lined. At the toe of the chute, flow would be dissipated by a flip bucket before it rejoined the White River.

The cost estimates include facilities for diverting water from the mainstem White River into the enlarged Lake Avery Reservoir. The design consists of a small diversion dam upstream at a location that varies with the size of the enlarged reservoir. A pipeline with a variable diameter, also contingent on capacity requirements, would be laid along the north side of the White River Valley. The pipeline would discharge into a stilling structure at the reservoir. The designs are not included in this report but were taken from the White River Study done by IECO in 1983. The cost estimates were adjusted to 1985 rates.

The detailed cost estimates for the Lake Avery expansion are included in Appendix H and are summarized in Table 10.

TABLE 10

LAKE AVERY: Embankment Dam Costs

	EXPANDED STORAGE CAPACITY (Acre-Feet)		
	20,000	40,000	60,000
Estimated Cost	$15,741,000	$30,730,000	$45,493,000

10.1.8 Findings and Conclusions

The subsurface investigation on the left abutment confirmed the existence of a cap of relatively impervious transported material overlying the terrace gravels identified by previous investigations. The thickness of the impervious mantle ranged from two feet at the west end of the abutment near the existing dam to twenty-five feet at the eastern end of the abutment. The material in the western half of the abutment is a brown clay of low plasticity. In the eastern half of the area studied, the clay is underlain and inter-tongued with a light brown mottled silty clay.

It is not recommended that the silty clay be left in place as a foundation material because of its porous and collapsible nature. The clay, on the other hand, appears to have sufficient strength to tie into an impervious embankment. In the case of a zoned embankment, however, the shell zone should be founded on top of the underlying gravel.

The quality and thickness of the impervious cap should be sufficient to provide an effective impervious barrier if left inplace. It is recommended, however, that in areas where the natural blanket is thinner it should be augmented by an additional thickness of the same material borrowed from an adjacent location.

A sufficient quantity of impervious fill is available on the left abutment to provide material for the core of the embankment and for the upstream impervious blanket.

Exposed bedrock was mapped above the waterline of existing Lake Avery for a distance of 2000 feet upstream of the dam. At that point it is covered with alluvial terrace material from above and cannot be traced further. After treatment, the exposed rock would provide an acceptable surface to tie in the impervious blanket. Future studies, however, should confirm the elevation of the top of bedrock at the upstream contact for the entire length of the blanket.

Seismic refraction on the right abutment showed the terrace deposit to be up to 50 feet in thickness. Even though it occurs at a higher elevation than the terrace on the left abutment, it would be exposed to reservoir water with the highest enlargement alternative. Test pits excavated on top of, and upstream of, the terrace show it to be covered with five to 15 feet of impervious clay. This clay covers the entire abutment and extends down the upstream slope into the reservoir. Therefore, it appears that no additional blanketing would be necessary on the right abutment.

Geologic mapping along the alignment of the proposed pipeline revealed that the right-of-way would be almost entirely on side slopes composed of the Eagle Valley-Minturn Formation. As discussed earlier in Section 4, the formation is composed of weak sandstone and siltstone, and includes soluble minerals such as gypsum and anhydrite. Therefore, there is a potential along the alignment for development of sink holes below the pipline and for slope failure. At several points upstream of Buford, the alignment would cross landslide debris originating from the edge of the basalt flows occurring at higher elevations. These areas also could offer potential for slope instability. Any future, more detailed study of the Lake Avery Project should include geotechnical work at the diversion damsite and along the pipeline alignment.

Since the reservoir capacities considered at Lake Avery do not coincide with those studied at the other sites, the cost estimates have been expressed as units of cost per acre-foot of storage in order to draw economic comparisions. The cost per acre-foot at the three storage capacities considered was fairly constant, ranging from a high of $787/acre-feet for the 20,000 acre-foot alternative to $758/acre-feet at the 60,000 acre-foot capacity. These storage costs, however, do not compare favorabaly to the cost per acre-foot of storage at any of the other sites. The most expensive 50,000 acre-foot alternative was the RCC dam at Veatch Gulch at $704/acre-feet or eight percent less than the 60,000 acre-foot alternative at Lake Avery. The most economical 50,000 acre-foot reservoir was the RCC dam at Warner Point at $469/acre-feet or 62 percent less expensive than the Avery alternative.

10.2 WARNER POINT DAM

10.2.1 History

Warner Point is a locally well-known topographic feature formed by a sharp bend in the river, in a narrow, steep stretch of the White River about two miles long. Several dam alignments could be located along this reach of the river.

Warner Point was first identified as a potential damsite by the USBR in their reconnaissance of the upper Colorado River unit in the early 1970's. No subsurface exploration or layouts were prepared. Geologic reports based on surficial mapping, however, indicated possible foundation instability on the left abutment, or south side of the river, in the nature of massive slumping.

During the mid- to late 1970's, the south side of the river in the vicinity of Warner Point was subdivided for vacation homes. Geologic and foundation reports associated with that development also indicated extensive landslides in the area.

In 1982 IECO selected Warner Point Dam as one of the alternative mainstem storage sites in the Yellow Jacket Project. Again, no subsurface exploration was done and foundation investigation was limited to a cursory site reconnaissance and to reviewing available literature. It was suggested, based on this reconnaissance, that the landslide problem, identified in previous work, might not be as extensive as first thought and that the site should not be ruled out without further geotechnical evaluation.

The Yellow Jacket Study showed that a single large upstream storage reservoir with a capacity of about 65,000 acre-feet would be adequate to develop the YJWCD water rights. The dam alignment was moved just less than one mile upstream of USBR Warner Point alignment to a location where the mimimum size dam would be required to provide the desired storage volume. Area-capacity curves and appraisal-level designs and cost estimates were prepared in conjunction with the study. The YJWCD filed for storage rights at the revised axis known locally as Tru-Sport.

The White River Study carried out by IECO in 1983 again identified Warner Point Dam and Reservoir, alone or in conjunction with other storage facilities, as being one of the more economic options available for White River water development. Another general geologic review was made in conjunction with that study. The only new information was furnished by a local driller, who reported the alluvial gravel in the river bottom to be approximately 25 feet deep based on several water wells drilled in the vicinity.

As part of the White River Optimum Sizing Report, prepared informally by IECO in 1984, Warner Point was again identified as one of the most economic storage sites in the upper White River Basin. This report also indicated reservoir capacities as large as 500,000 acre-feet could be employed to provide increased yields with incremental increases in storage.

Subsequent to that report, the State requested that the USBR make a geologic reconnaissance of the large mainstream sites identified by IECO. The USBR's cursory survey again indicated that the Warner Point site was not feasible due to massive slumping on the left abutment. Therefore, one of the primary goals of the White River Geotechnical Study was to confirm or deny the effect of the landslide activity on the feasibility of a dam at the site.

10.2.2 Reservoir Characteristics

The Warner Point Reservoir for storage volumes of 50,000 acre-feet, 150,000 acre-feet, and 300,000 acre-feet, is shown on Plates 15a and 15b. Depending on the storage capacity selected, the reservoir could extend as far upstream as Buford, and with the largest alternative, would submerge the existing Lake Avery Dam and Reservoir. Approximately 1600 acres of the flooded area would be privately owned land presently irrigated and planted in hay or utilized for livestock grazing. The remainder of the inundated land is owned by the State, primarily to the north of the river, and by private parties to the south. This land is steeper grazing land characterized by sage, grass, and scrub oaks. The Sleepy Cat Guest Ranch and Bel-Aire Fish Hatchery, as well as several ranch houses and out buldings, would be flooded by almost any scenario.

The reservoir would operate as an emergency storage and upstream regulating facility for the White River. Water derived from spring runoff would be stored and released in late summer of dry years for agricultural, municipal, and industrial uses. There is a possibility that the dam could have some potential for generating hydroelectric power in a run-of-the-river scheme.

10.2.3 Site Geology

The Warner Point Dam site lies completely within the Maroon Formation comprised of hard, thick layers of red-to-pink micaceous sandstone interbedded

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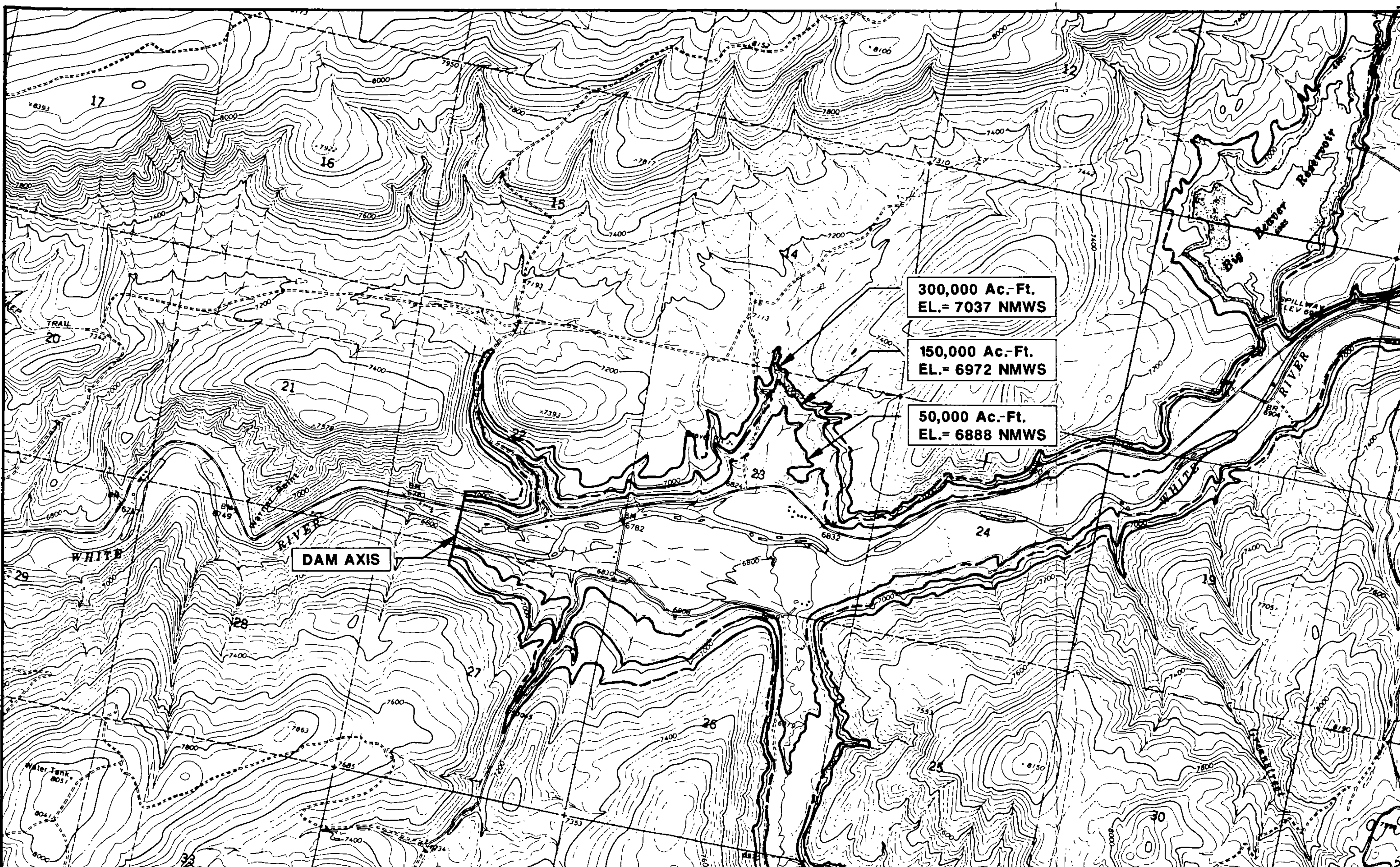

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RESERVOIR AREAS
WARNER POINT DAM

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Plate 15B

with weaker, thin layers of deep red mudstone, siltstone, and shale. Plates 4 and 5 are geologic maps of the area and Plate 16 shows the surficial geology of the damsite. Bedding at the damsite strikes N70°E and dips 10° to 20° to the northwest. Because the river at this point runs roughly parallel to the strike of the bedding, two distinct types of topography have been produced on either side of the river.

The right abutment, or north side of the river, is characterized by sparse vegetation and steep stair-stepped cliffs formed by differential weathering of the sandstone and weaker rocks as they dip into the hillside. The resistant sandstone layers form near-vertical cliffs, whereas weathering of the weaker siltstones and mudstones, produce gentle slopes or steps. The slope instability created by the weathering of the weak zones causes large blocks of sandstone to topple down the slope and accumulate at the base as talus or colluvium.

The left abutment is a dip slope of about 15° toward the river. The higher parts of the abutment are covered with scrub oak and pines, sage, and grass, whereas the steeper slope near the river is densely vegetated with large evergreen trees, and a rich ground cover of ferns and bushes. Surficial geologic mapping revealed in-place Maroon Formation bedrock at the crests of several strike ridges, probably representing more resistant sandstone layers, on the upper part of the abutment. Outcrops were also mapped in two of the northeasterly-trending, deep linear drainages perpendicular to the river valley and in the major drainage just upstream of the damsite. The topography of the upper left abutment appears to have been produced by differential weathering of the thin, weak zones producing long, gentle dip slopes which, due to the northern exposure, have weathered to some depth. The lower part of the abutment appears to have suffered some landslide activity, probably caused by slippage along the weaker bedding plane units parallel to the slope. This has produced an accumulation of debris on the moderate slope just above the final steep slope to the river. In-place outcrops were mapped in only two places along the left abutment adjacent to the river, but based on the steepness of the slope and the size of the sandstone blocks visible along the river, it does not appear that the base of the left abutment is a landslide. There is no evidence in the vicinity of the damsite for massive rotational type landslides that might make construction of a dam infeasible.

No major faulting is apparent in the immediate vicinity of the damsite. Several northeasterly-trending lineations parallel to the normal faults mapped downstream appear to be associated with regional jointing patterns. Several gentle folds are noticeable to the north of the river, but are of such small magnitude that mapping is unwarranted.

Three principal sets of subvertical joints were mapped in the outcrops of the right abutment, two of which correspond to the regional jointing described in Section 4. The principal sets strike approximately N55°W and N10°E. The joints are moderately spaced, and are often stained with iron oxide and partially filled with calcite. Occasionally, small offsets and slickensided surfaces are evidenced in the subvertical joints, probably associated with recent stress redistribution caused by slope development. The third set of vertical joints occurs intermittently, parallel to the valley, and probably represents local stress relief caused by erosion of the valley and consequent unloading. A fourth major joint set is parallel to bedding. Most of these joints are closed, but occasionally open, oxidized, or partially filled joints can be mapped, especially at lithologic contacts, evidencing the passage of water.

Recent deposits at Warner Point are characterized by the landslide on the left abutment and colluvium at the base of the abutments on either side of the river, which may reach depths of up to 50 feet. These are composed of angular blocks of sound sandstone surrounded by a matrix of silty sand. The valley floor is formed of alluvial sand and gravel composed of sound, well-rounded, poorly-graded cobbles and grains derived from Precambrian granite, gneiss, and quartzite generally gray in color, black-to-purple basalt and some sub-rounded red sandstone derived locally from the Maroon Formation.

Nearly the entire Warner Point Reservoir would cover alluvial gravels on the valley floor and gently-to steeply-sloping bedrock of the Maroon Formation at the water line. Terrace deposits of alluvial sand and gravel of several ages occur along both the north and south sides of the reservoir. Minor occurrences of weakly consolidated sandstone of the Brown's Park Formation were also mapped in the reservoir. Of great importance to the feasibility of a reservoir at Warner Point is the occurrence of the evaporitic facies of the Eagle Valley-Minturn unit mapped in the upper extremes of the reservoir near Buford, at the confluence with the South Fork of the White River, and up Elk Creek. Several sink holes were observed, both in the alluvial gravels of the valley floor and in the slopes where the unit crops out.

10.2.4 Foundation Investigation

The subsurface foundation investigation at Warner Point Dam consisted of eight rotary bore holes drilled along the centerline of the proposed dam. Seismic refraction surveys were carried out on the centerline upstream and downstream of the dam, and on the left abutment. The locations of the borings and seismic lines are shown in plan on Plate 16, and the subsurface information is represented by sections on Plate 17. Borehole logs and photographs of the core are contained in Appendix C.

The borings were initiated on the left abutment with the intent of evaluating the apparent landslide problem brought up previously by others and noted during the surficial mapping. Hole W-C1 was located on the crest of an east-west striking ridge, at approximately elevation 7055. The surface was strewn with large boulders of red sandstone from the Maroon Formation that had been disturbed, but due to their linear orientation along the ridge, were thought to be nearly in place. The hole was drilled vertically and penetrated 15 feet of micaceous silty sand, deep red in color and of low plasticity, before encountering in-place, hard, sandstone bedrock of the Maroon Formation. The material was logged as residual soil derived from in-place weathering of a siltstone interbed in the Maroon Formation. There was no proof, however, that the material was undisturbed, as no bedding could be identified and small fragments of hard sandstone were recovered. Therefore,

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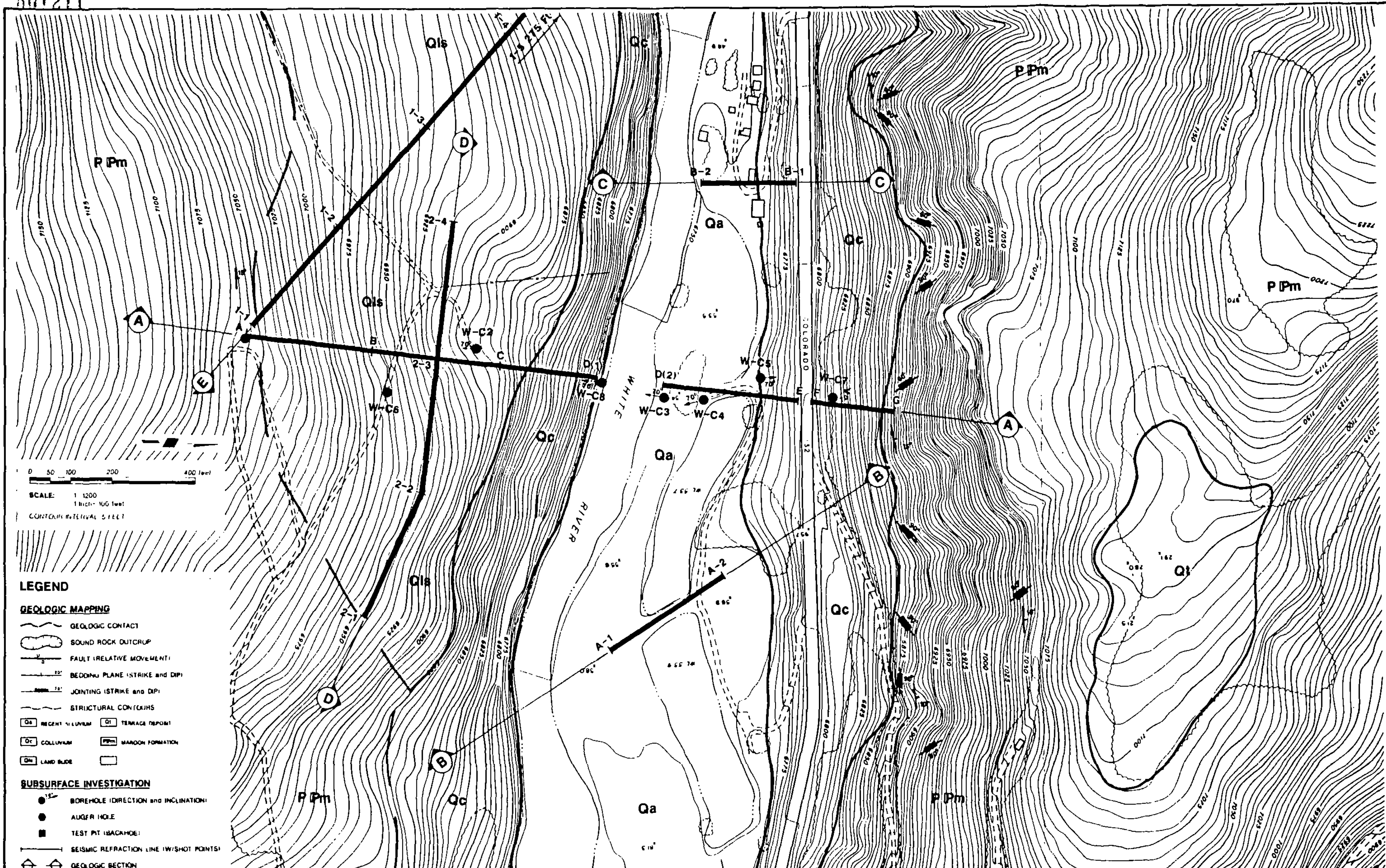

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GEOLOGIC SECTIONS
WARNER POINT DAM

FILE NO
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Plate 18

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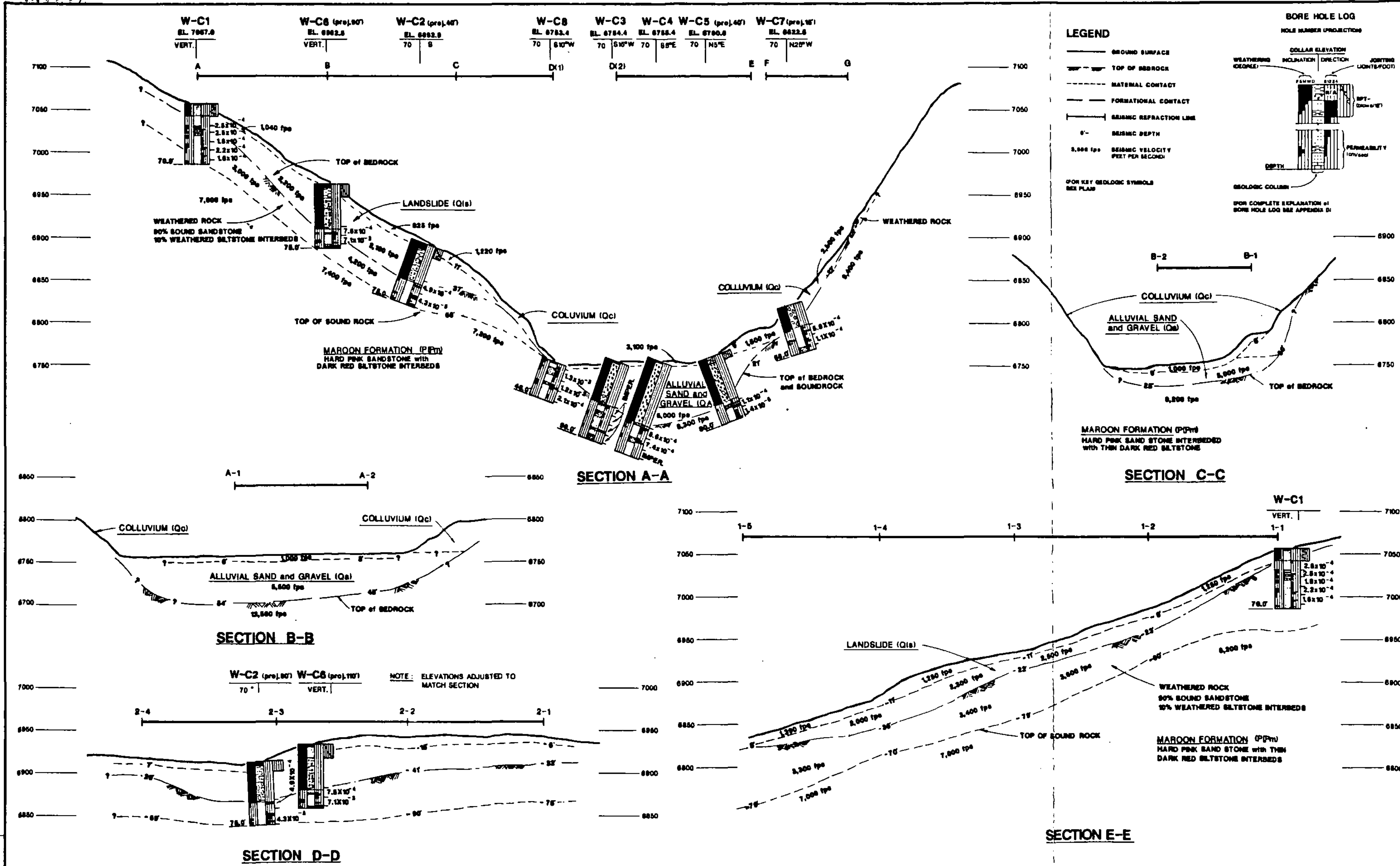

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GEOLOGIC SECTIONS
WARNER POINT DAM

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Plate 17

the upper zone could be considered as transported slide debris or colluvium. In any case, the upper 15 feet would probably have to be removed to provide an adequate foundation for any type of dam. Standard penetration test blow counts in this zone averaged about 20 per foot except where the sampler was in contact with a hard rock fragment. Sound, hard, pink-to-red sandstone interbedded with weak, weathered layers of deep red, moist, siltstone bedrock of the Maroon Formation was recovered between 15 feet and the bottom of the hole at 70 feet. The core displayed well-defined bedding planes that were consistent with the strike and dip in the general vicinity, or N70°E, 15°NW. Weathering occurred in siltstone interbeds up to a few feet thick. Along bedding plane joints in the fractured and highly weathered zones, the core was often broken and washed by the drilling and core recovery and RQD values suffered accordingly. The primary set of joints detected was at a high angle, with calcite filling, and showed a strike of about N55°W. Water pressure tests showed a constant permeability for the bedrock of approximately 2.0×10^{-4} centimeters per second (cm/sec), due to dilation of open joints under pressure.

Holes W-C6 and W-C2 were located in the suspected landslide area on the left abutment at approximate elevations 6950 and 6900, respectively. W-C6 was vertical and W-C2 was angled 70° into the abutment. Both holes showed from five to six feet of organic sandy silt topsoil overlying approximately 48 to 54 feet of transported debris. The zone consisted of large angular blocks of pink sandstone to three feet in diameter in a moist, dark red, sandy clay matrix. The orientation of the bedding planes in the sandstone blocks varied, proving the material was not in place but rather had been disturbed and mechanically transported. The two holes were drilled to a final depth of 75 feet. The in-place bedrock below the debris zone was similar to that found in hole W-C1, and is sound, hard micaceous sandstone of the Maroon Formation, with interbeds up to a few feet thick of moist, deep red, highly weathered siltstone. The jointing pattern was also similar in orientation; however, the high-angle joints were generally open rather than calcite filled. A second set of subvertical joints striking northeast was also detected, corresponding to the regional joints mapped on the surface. Foundation rock permeabilities ranged from 7.0×10^{-3} cm/sec to 4.3×10^{-4} cm/sec, with the water take apparently due to the high angle open joints.

Borehole W-C8 was drilled at an angle of 70^{o} to a depth of 46 feet into the left abutment, on the bank of the river at approximate elevation 6760. The core recovered was in-place red sandstone and siltstone from the Maroon Formation. There was no evidence that the rock had been disturbed. As in the other holes on the left abutment, there were a few highly weathered zones parallel to bedding, corresponding to siltstone interbeds; however, these layers were thinner below nine feet than in the holes higher up the abutment. High-angle joints were much less pronounced in this hole, and open and closed bedding plane joints predominated. Permeability to 30 feet was 1.2×10^{-3} cm/sec, tightening to 2.1×10^{-4} cm/sec in the bottom 12 feet.

The river valley alluvium was penetrated by boreholes W-C3, W-C4, and W-C5. The holes were angled 70^{o} into the nearest abutment and drilled to depths of 98, 118, and 80 feet, respectively. They penetrated alluvial sand and gravel, representing reworked glacial outwash composed of well-rounded, gray-to-pink granite, gneiss, and quartzite, and black-to-purple basalt. Hole W-C5 at the base of the right abutment showed a thickness of seven feet of colluvial debris overlying the alluvium. The two holes nearest each abutment revealed about a ten-foot-thick zone of large, pink, angular sandstone blocks derived from the Maroon Formation between the alluvial gravel and the underlying bedrock which occurred at a depth of about 50 feet. The blocks are probably the remnants of ancient landslide debris contemporaneous with river downcutting.

Borehole W-C4, located in the center of the valley, passed directly from alluvium to weathered bedrock at a depth of 72 feet. The bedrock penetrated in all three borings was sound pink sandstone with darker red siltstone interbeds and an occasional lense of gray limestone. Weathering of the interbeds and bedding plane joints was much less than in the foundation rock of the left abutment, as was the frequency of jointing. Like hole W-C8, the primary joints logged were open and closed bedding plane joints with a few high-angle joints striking northwesterly. Water losses in these holes were very low, showing permeabilities from 7.4×10^{-4} cm/sec to impermeable, within the accuracy of the test.

Only one hole was drilled on the right abutment, as the relatively competent Maroon Formation crops out above approximately elevation 6900. The lower part of the slope is covered by a skirt of colluvium. Borehole W-C7 was drilled at about elevation 6815 and angled 70^0 into the abutment to determine the depth of the transported material. The hole penetrated 28 feet of sound, angular sandstone boulders surrounded by dark, moist, silty sand. The orientation of the bedding in the boulders varies, indicating they are disturbed and transported. Bedrock is sound, hard sandstone of the Maroon Formation with siltstone interbeds. Weathering along bedding planes, however, is much less pronounced than on the left abutment, to the final depth of the hole at 58 feet. Only one highly weathered zone was detected, at a depth of 40 feet, where core was lost.

The seismic refraction survey correlated well with the boreholes along the centerline's left abutment as surveyed by line A-D and perpendicular to it, by line 2-1 to 2-4. They showed a soil layer ranging from 5 to 10 feet thick with a velocity of less than 1250 fps. The slide debris registered a velocity of about 2100 fps and ranged from 20 to 54 feet in thickness, thinning toward the drainages upstream and downstream of centerline. The Maroon Formation bedrock with the highly weathered siltstone interbeds showed a surprisingly low velocity of only 3000 to 4200 fps and ranged in depth from 15 to 85 feet from the surface. The velocity in the unweathered bedrock was also relatively slow at about 7200 fps.

The right abutment seismic line E-G showed the colluvial debris to exhibit low velocity, less than 1500 fps. Colluvium directly overlies sound bedrock with a velocity of about 9400 fps at a depth up to 33 feet and as shallow as 21 feet at the highway cut.

The lines across the alluvial valley floor D-E, A1-A2 and B1-B2 showed the gravel to have a velocity of about 5000 to 5500 fps and a thickness of 54 feet. The gravel overlies sound bedrock with a velocity of 8300 to 9500 fps. No deeply incised channels were detected by the seismic survey.

Line 1-1 to 1-5 along a possible spillway alignment on the left abutment was similar to the left abutment centerline profile in zones and velocity with the slide debris ranging from 29 to 39 feet in depth and the weathered bedrock zone extending as deep as 100 feet.

10.2.5 Construction Materials Investigation

The exploration for construction materials at Warner Point was carried out with backhoe excavated test pits. The locations and logs of the test pits are shown on Plate 18.

Sand and gravel of suitable quality and quantity for RCC and conventional concrete aggregate are available in the alluvial valley floor and in terrace deposits just upstream of the dam on the right side of the river. These deposits include the county gravel pit. The gravel would also be suitable shell material, and when processed, filter and drain material for an embankment dam.

In the event an embankment dam was determined to be the most economic design, a side channel spillway would generate a large quantity of rockfill of suitable quality for use as riprap and in the shell of the dam. Excavation of landslide debris would probably produce some waste as large blocks, but the bulk of the material, would be a silty sand with fragments of sound sandstone that could also probably be used in the shell zone. No test pits were excavated to investigate the materials from required excavation other than in the alluvial gravel.

The borrow area for impervious core material for a potential embankment dam was explored in a gentle valley about one mile to the north of the site. The

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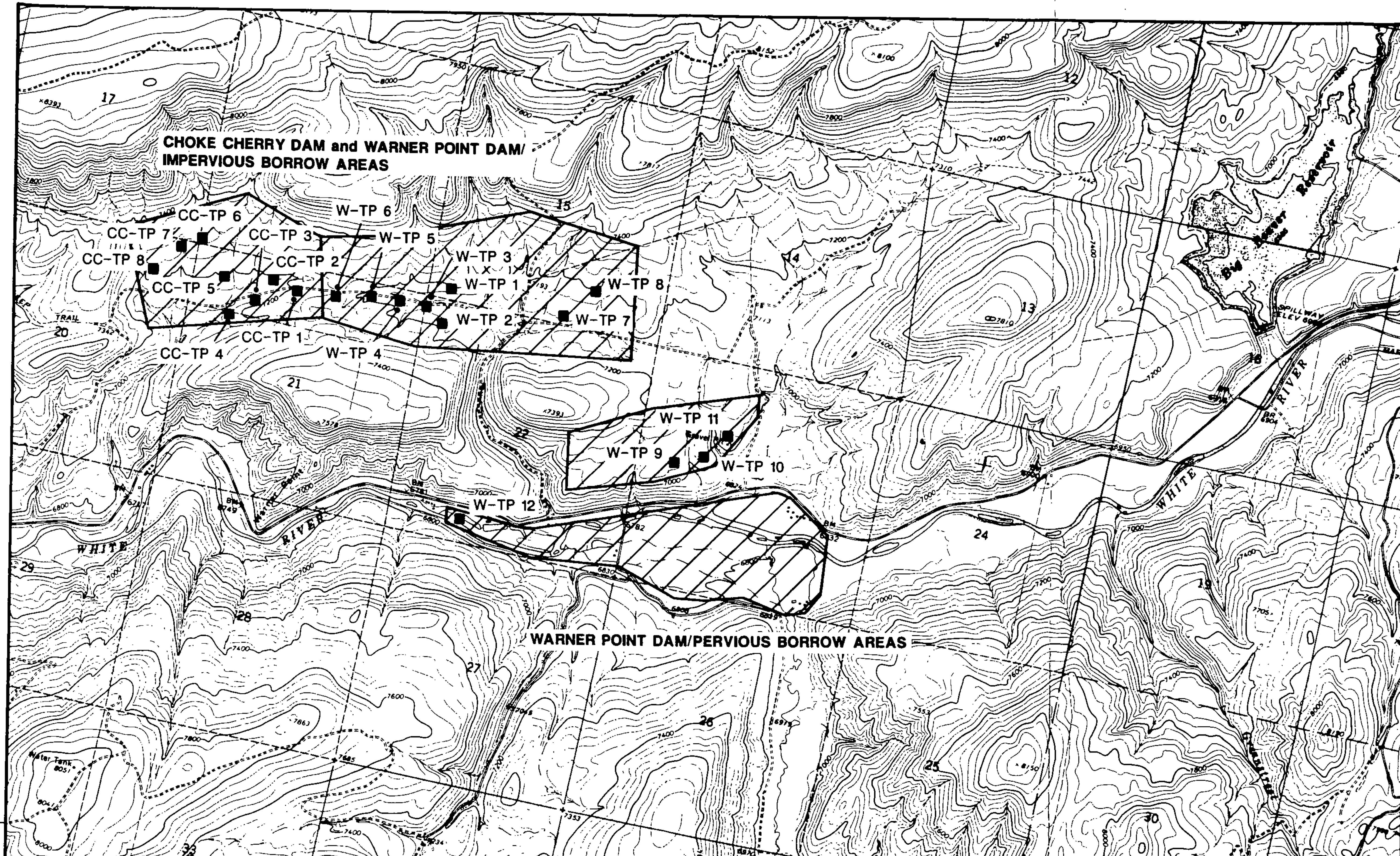

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WHITE RIVER GEOTECHNICAL STUDY
CONST. MATERIALS/PLAN
WARNER PT. & CHOKE CHERRY

ECO NO.	
SHEET OF	REV.
Plate 18	

valley parallels the White River at an elevation about 450 feet above the main valley floor for a distance of about 2.5 miles. The western portion of this borrow area could also serve as a source for impervious material for the Choke Cherry Dam. The test pits in the area revealed a deep red, sandy clay of low plasticity derived from weathering and erosion of the State Bridge and Chinle Formations, transported to the lower elevations of the valley. The deposit ranges in depth from eight feet to 12 feet, the maximum depth attainable with the backhoe. Samples of the material were obtained from the backhoe pits and tested in the laboratory. The materials strength and permeability characteristics are favorable for use as the impervious core of an embankment dam. Test results are included in Appendix E and summarized in Table 11.

10.2.6 Engineering Geology

The surficial geologic mapping and subsurface investigation has shown that Warner Point is a technically feasible damsite from the geological standpoint. The suspected landslide deposits on the left abutment that were feared would affect feasibility do, in fact, exist. However, the thickest occurrence detected either by drilling or seismic refraction was 54 feet. The lateral extent of the landslide deposits is also limited to the lower portion of the abutment and do not occur below the elevation of the present valley floor. The landslide appears to be an accumulation of ancient surficial slips along weak weathered interbeds of siltstone and shale, parallel to the dip slope toward the river. There is no sign of recent movement, nor is there evidence of large scale mass slumping that might affect the bedrock at depth. The slide debris would have to be completely removed for either an embankment or RCC design.

The left abutment shows an interesting weathering feature. That is the separation of thick layers of sound sandstone by weathered interbeds of siltstone and claystone to a maximum depth of about 70 feet along the dam axis and as deep as 100 feet downstream on the left abutment along the alignment of a possible side channel spillway. The rock is massive and sound over 90 percent of its volume but the weaker 10 percent could affect the foundation stability. Any future geotechnical work at the site should include obtaining

TABLE 11

WARNER POINT: Construction Materials Test Results

Test Result	DESCRIPTION Clay (CL) and Clayey Silt (CL-ML)	Gravel (GP)
Plasticity Index	6	NP
Natural Moisture	13%	N/A
Optimum Moisture	13%	N/A
Optimum Density	116 pcf	145 pcf
Shear Strength	C=11 psi, $\phi=27^{o}$	C=0, $\phi=40^{o}$
Permeability	3.8×10^{-7} cm/sec	1.0×10^{-2} cm/sec

undisturbed samples of these weathered zones in order to determine their strength characteristics and effect on foundation stability. Lacking detailed strength parameters for this foundation zone, it was assumed that the weathered rock zone in question could be left in place for an embankment dam, but would have to be removed for the RCC gravity dam design.

The top of bedrock under the alluvial gravel cover in the valley section can be taken as adequate foundation. On the right abutment, a 10-foot-deep cutoff trench would be recommended for the embankment dam, and 10 feet of rock should be removed over the entire footprint for the RCC dam.

Due to the well developed set of open bedding plane joints and lithologic contacts in the Maroon Formation, and considering the frequency of subvertical joints striking northwest, a grout curtain should be anticipated along the entire length of the dam. High grout takes could be expected, but an effective cutoff should be attainable. Slope stability should only be of concern in the side channel spillway cuts. Permanent slopes in the alluvial gravel, colluvium, and slide debris will probably be stable at 1.5H to 1V. Slope stability in the weathered rock cuts will depend on the dip of the weak siltstone interbeds and their shear strength. On the left abutment spillway arrangements, the alignments are roughly parallel to the principal set of subvertical joints and the weak bedding plane joints will dip toward the channel at about 15 degrees. These two discontinuities in conjunction with a third northeasterly striking set of joints could necessitate some slope stabilization measure such as rock bolts, shotcrete, and/or post-tensioned tendons.

On the right abutment, toppling of large sandstone blocks released by the subhorizontal bedding and stress relief joints parallel to the valley can be expected to continue at a slow rate in the future. This should not be a problem except where it might affect access to the crest of the dam or the rock cut for a spillway channel. In these areas, rock bolting and shotcreting should be adequate to stabilize the slabs.

Temporary excavation of the cutoff trench for an earthfill dam or the footprint for a gravity dam in the riverbed alluvium should be stable at slopes as steep as 1H to 1V, provided the exit gradient of ground water infiltration can be controlled.

The stability of south-facing slopes within the reservoir does not appear to be a problem. Even though these slopes are often steep and sparsely vegetated, the bedding of the bedrock dips away from the reservoir. The most severe instability that could be anticipated would be an occasional slab separating from the cliffs. The northerly-facing south side of the reservoir could present some slope instability with rapid drops in reservoir level. This problem could occur as a reactivation of the ancient landslides immediately upstream of the reservoir, or three to four miles upstream on the thickly wooded slopes where the Maroon Formation is underlain by the Eagle Valley-Minturn Formation and overlain by poorly consolidated Browns Park Sandstone and slump debris at the edge of the Tertiary basalt flows. If a reservoir were constructed, instrumentation of these areas would probably be in order although the reservoir slope stability is not expected to be a problem because of the flatness of the slopes.

A bigger concern regarding the adequacy of a future reservoir at Warner Point is the existence of sink holes in the reservoir area up Elk Creek and near Buford. There, the reservoir would be underlain by the Eagle Valley-Minturn Formation. The evaporitic facies of the Eagle Valley Formation consists of soluble salts, gypsum and anhydrite that, when exposed to reservoir head, could conceivably dissolve and form solution channels that would exit somewhere downstream of the dam. Surficial mapping of the Eagle Valley, however, has shown the evaporitic facies to be discontinuous and to be intensely folded. Therefore, the probability of the existence of a continuous bed extending over a long distance is low. The shortest seepage path from upstream to downstream of the dam, through the outcrops of the Eagle Valley-Minturn unit mapped, would be from the lower part of Elk Creek to Miller Creek across from Moog Gulch, a distance of about five miles. It is therefore considered highly unlikely that seepage of this nature would occur. If plans for implementing the Warner Point Project are carried further in the future, the potential for reservoir seepage should be one of the primary items to be evaluated.

10.2.7 Design and Cost Estimate

Based on the foundation and materials information gathered from the geotechnical investigation, appraisal-level designs and cost estimates were prepared for dams at the Warner Point site with storage capacities of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. Preliminary comparisons between embankment and roller compacted concrete gravity sections showed the two alternatives to be competitive, therefore designs and cost estimates were prepared for each capacity with both concepts.

10.2.7.1 Embankment Dam

The embankment dam designs are shown in plan on Plate 19 and in section on Plate 20. The three sizes of embankment would have crest elevation heights above present river bed and normal maximum reservoir surfaces as as given in Table 12.

Foundation preparation would consist of complete removal of all slide and colluvial debris. It would be necessary to excavate a cut-off trench through the pervious alluvium of the valley floor to the top of bedrock and 10 feet into weathered rock where exposed on the left and right abutments. The upstream and downstream shells would be founded on in-place alluvial gravel and weathered rock on the abutments. A grout curtain is necessary along the entire length of the dam due to the jointed nature of the rock.

TABLE 12

WARNER POINT: Embankment Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)	MMWS Elevation (ft msl)
50,000	6917	162	6888
150,000	7001	246	6972
300,000	7066	311	7037

The embankment would be a zoned earthfill conforming to the design described in Section 8.1. Impervious core would be derived completely from the borrow area described in Section 10.2.5. The shell zones would be constructed of slide debris, rock, and gravel produced by required excavation of the dam foundation and spillway, and complemented by borrowed gravel as necessary. The upstream slope would be protected with riprap, also derived from rock excavated for the spillway chute.

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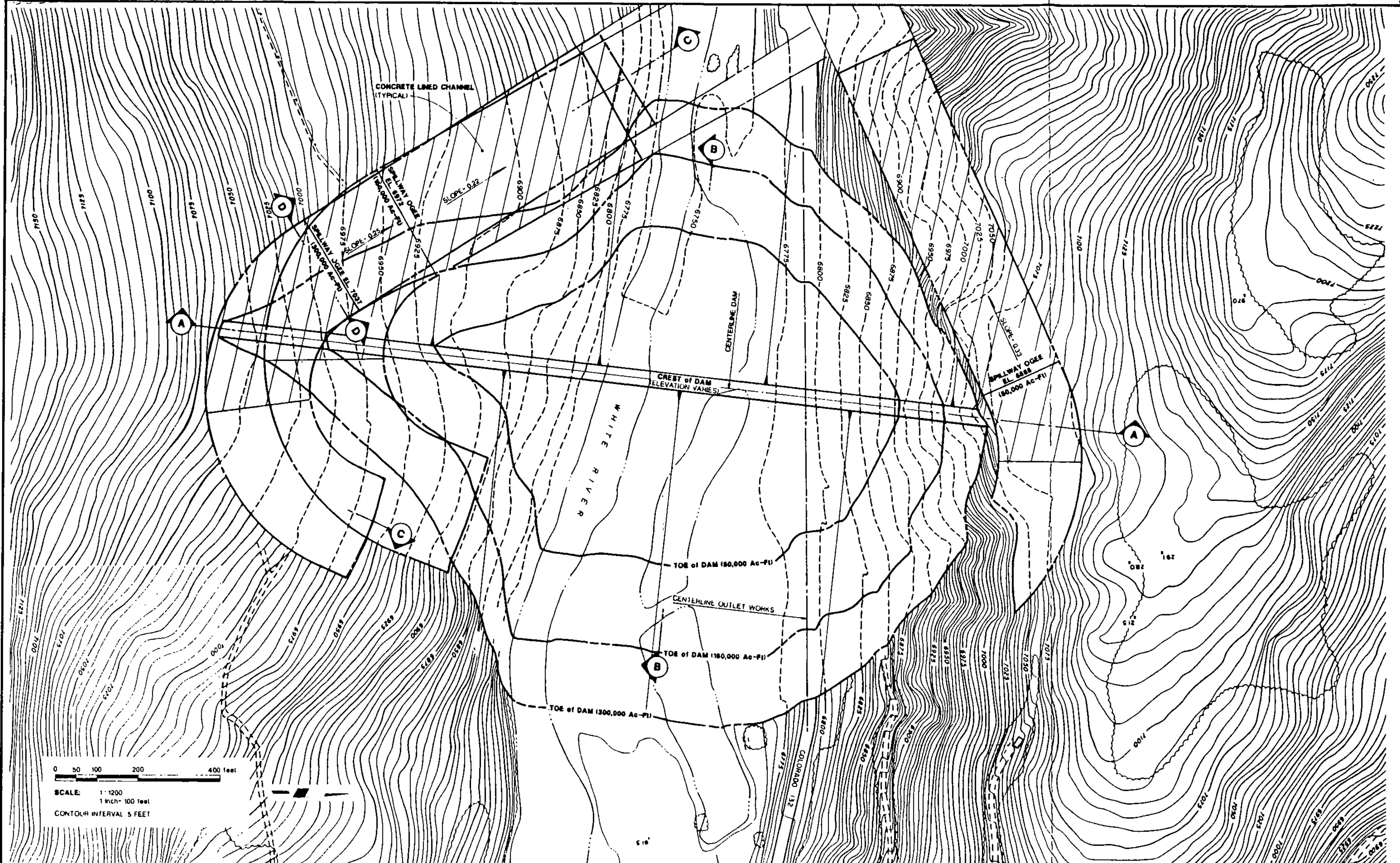

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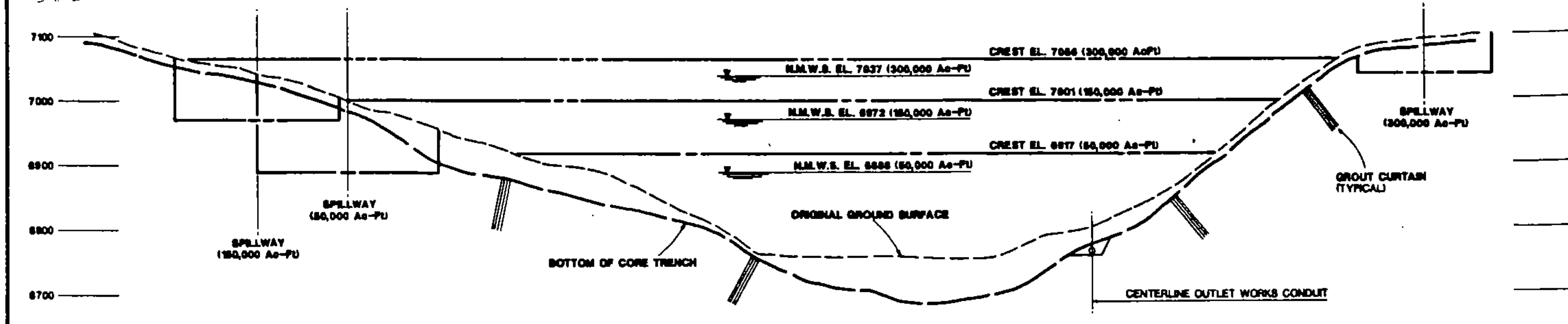
WHITE RIVER GEOTECHNICAL STUDY

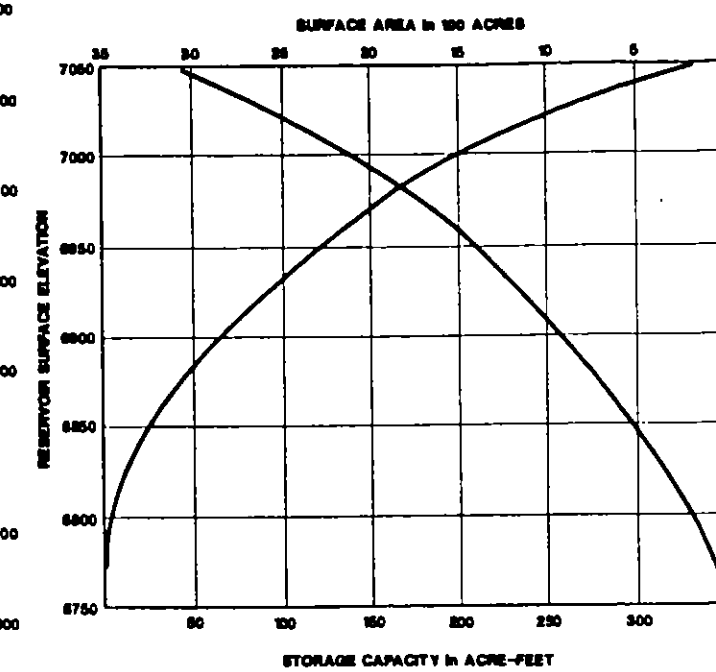
EMBANKMENT DAM/PLAN
WARNER POINT DAM

IECO NO
SHEET OF REV
Plate 19

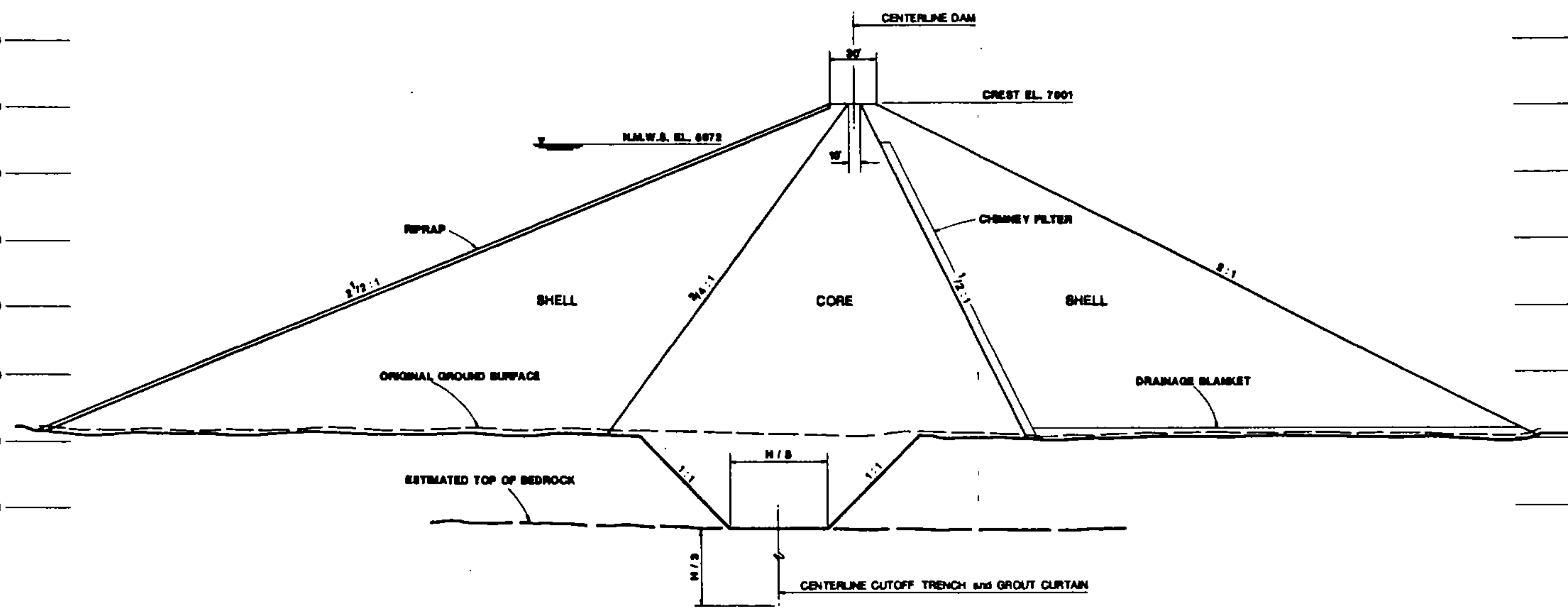
001224



SECTION A-A PROFILE ALONG CENTERLINE OF DAM



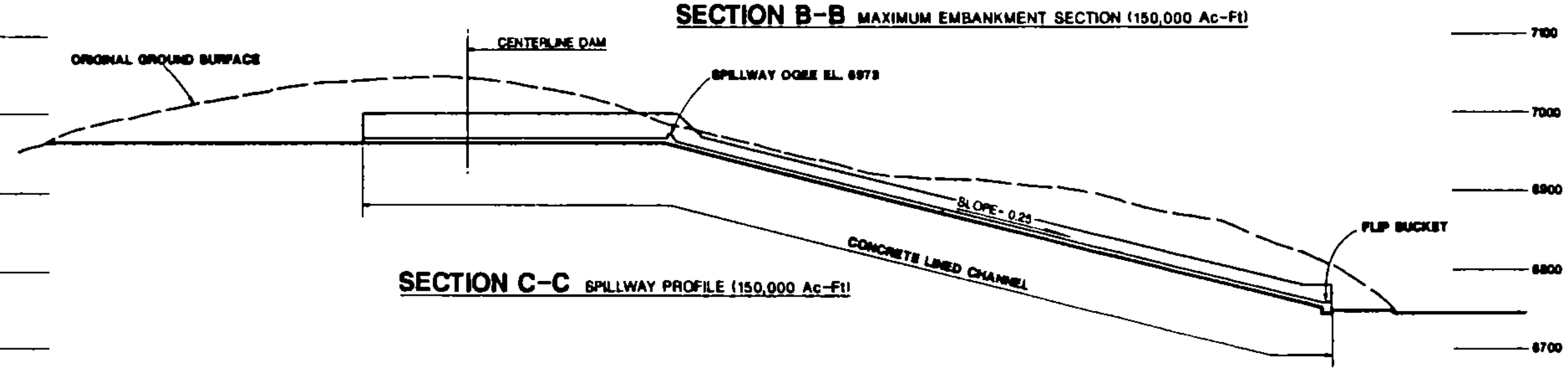
RESERVOIR AREA/CAPACITY



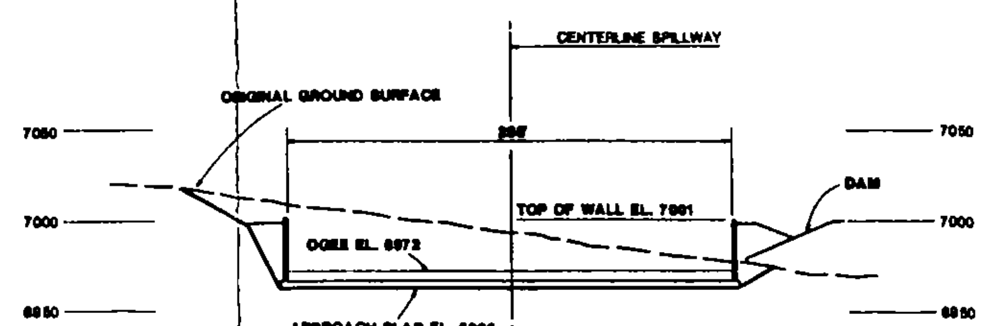
SECTION B-B MAXIMUM EMBANKMENT SECTION (150,000 Ac-Ft)

TABLE of ESTIMATED QUANTITIES

ITEM	RESERVOIR SIZE in ACRE FEET		
	50,000	150,000	300,000
COMMON EXCAVATION (Cu. Yd.)	1,089,330	1,493,060	1,450,327
ROCK EXCAVATION (Cu. Yd.)	854,832	493,895	1,234,203
CORE (Cu. Yd.)	583,537	1,468,219	2,487,942
SHELL (Cu. Yd.)	1,243,070	3,639,706	6,133,245
RIPRAP (Cu. Yd.)	33,536	66,631	99,016
GROUTING (Sk.)	2,025	4,025	5,957
STRUCTURAL CONCRETE (Cu. Yd.)	22,665	26,045	21,145
LAND ACQUISITION (Ac.)	1,055	2,226	3,276
HIGHWAY RELOCATION (Mi.)	5.3	7.3	8.1



SECTION C-C SPILLWAY PROFILE (150,000 Ac-Ft)



SECTION D-D SPILLWAY CREST (150,000 Ac-Ft)

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EMBANKMENT DAM/SECTIONS
WARNER POINT DAM

IECO NO
SHEET OF REV.
Plate 20

The outlet works and spillway designs for the embankment dam alternative are described in Section 8.1. Typical details of the designs are shown in Plate 9. The spillway channels are cut into the left abutment for the lower two alternatives and the right abutment for the highest dam spillway. The discharge energy would be dissipated by a flip bucket.

The detailed cost estimates for the embankment alternative at Warner Point are included in Appendix H and are summarized in Table 13.

10.2.7.2 RCC Dam

The designs for the RCC gravity dam alternative are shown in plan on Plate 21 and in profile on Plate 22. The three sizes of dam would have normal maximum water surfaces equal to those of the embankment dam, but the crest elevation and height above stream bed would be as shown in Table 14.

TABLE 13

WARNER POINT: Embankment Dam Costs

	STORAGE CAPACITY (Acre-Feet)		
	50,000	150,000	300,000
Estimated Cost	$31,421,000	$38,876,000	$58,415,000

Foundation preparation would consist of removal of all transported soil and debris, as well as weathered and highly fractured bedrock. It was assumed for the estimates that excavation would extend 10 feet into exposed bedrock on the right abutment, to the top of bedrock in the river, and to the top of sound bedrock on the left abutment. Excavation on the left abutment would extend as

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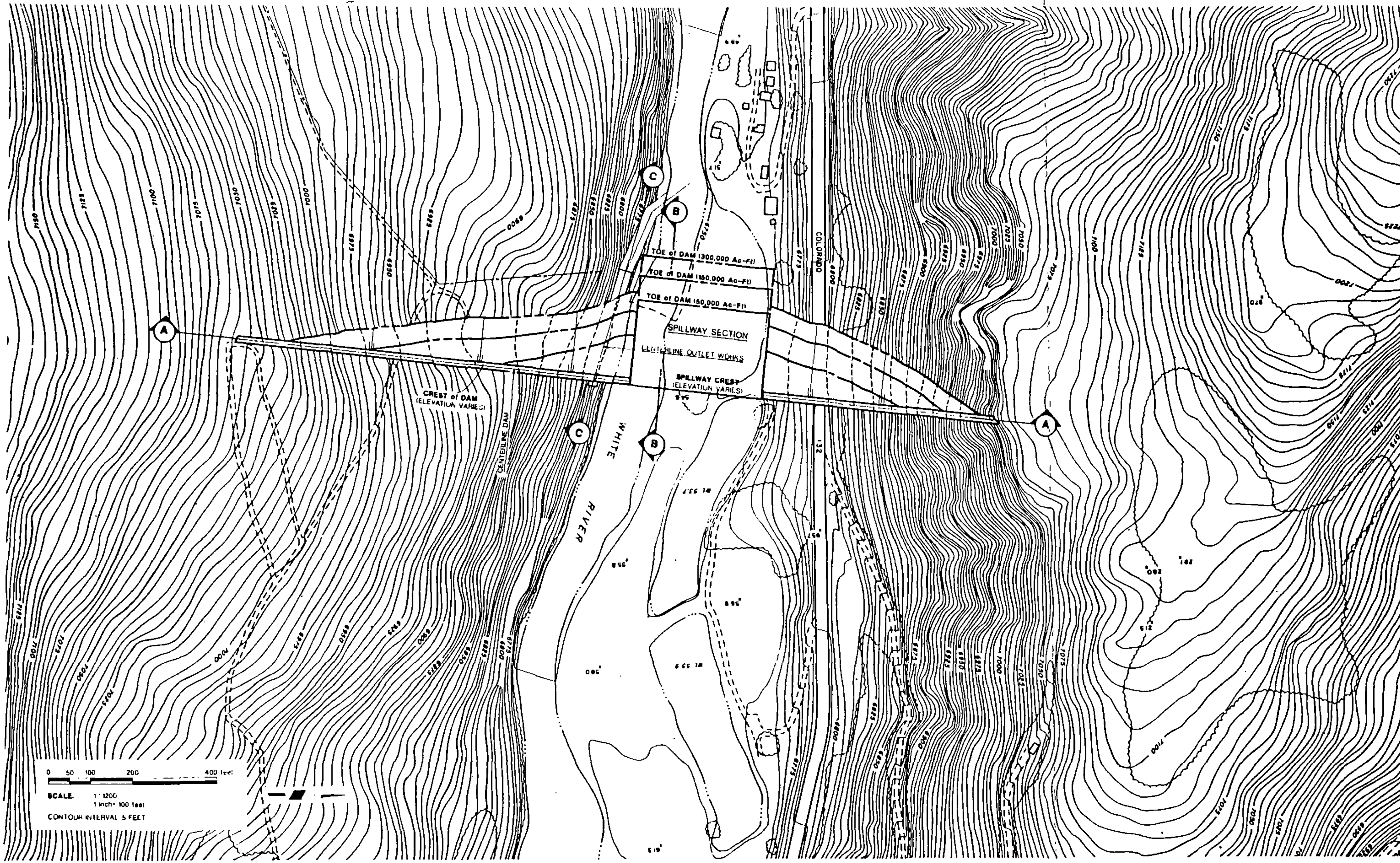

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WHITE RIVER GEOTECHNICAL STUDY
RCC DAM/PLAN
WARNER POINT DAM

MKE NO.
SHEET OF REV.
Plate 21

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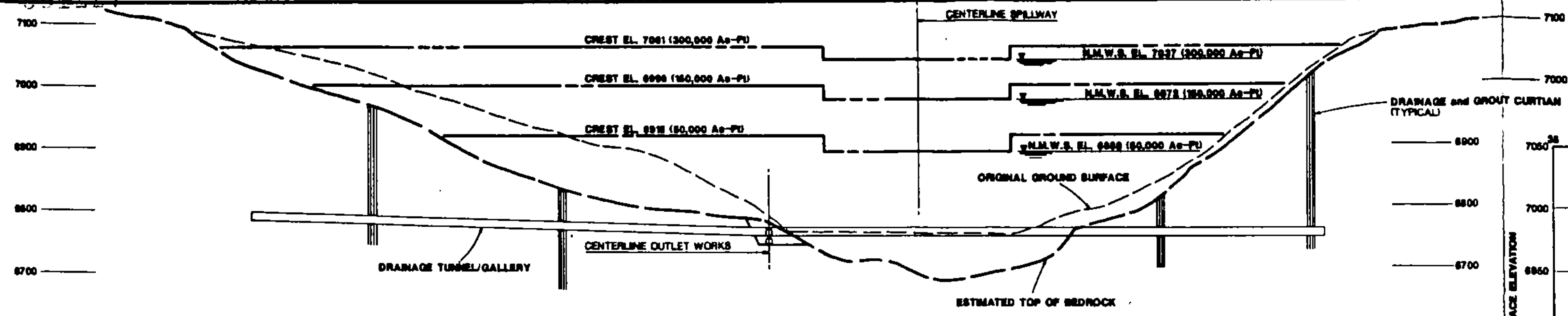
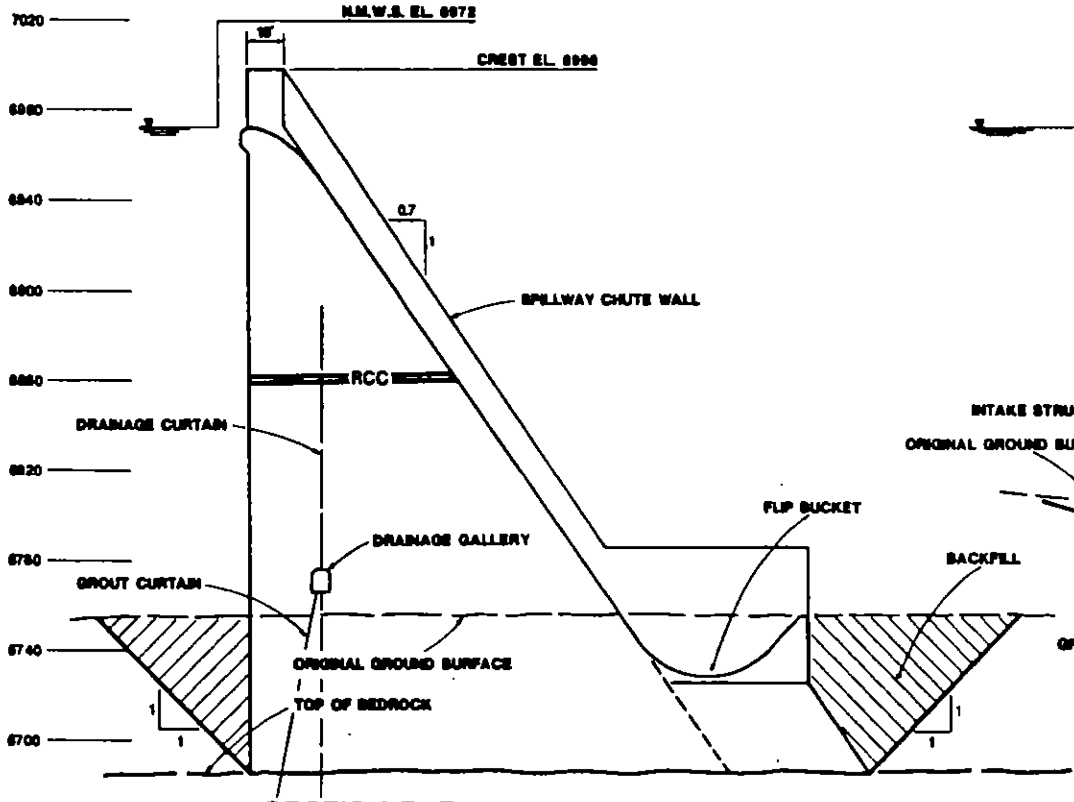
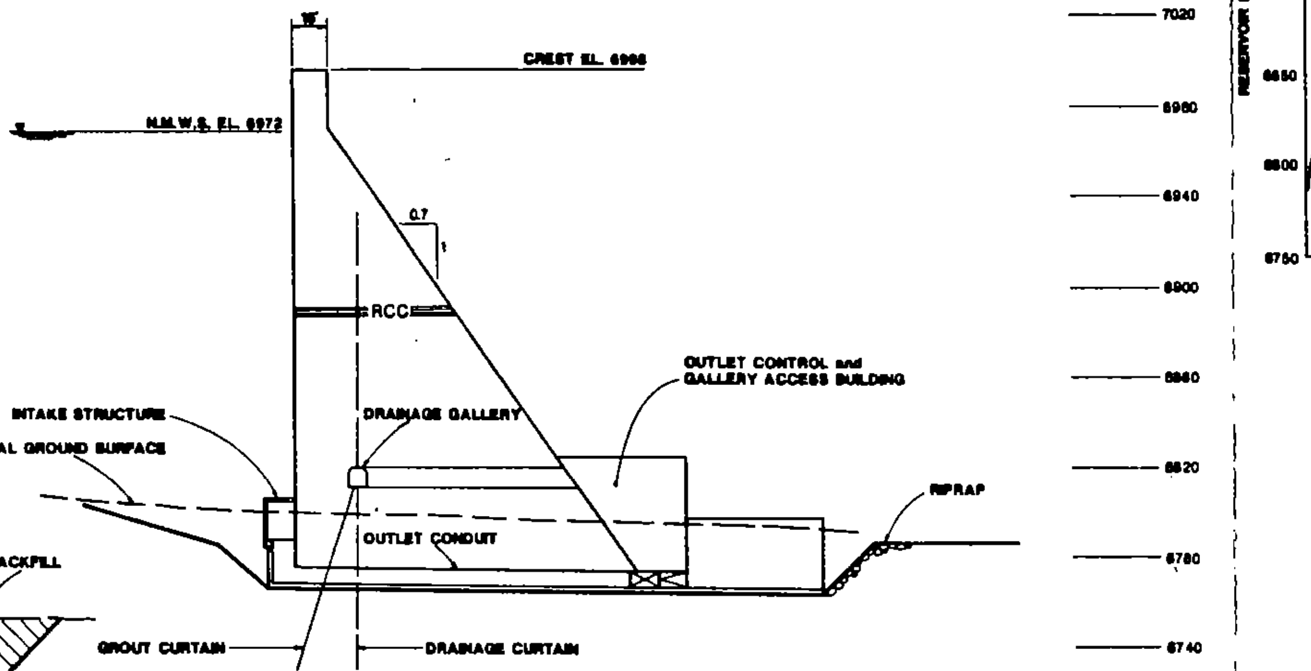


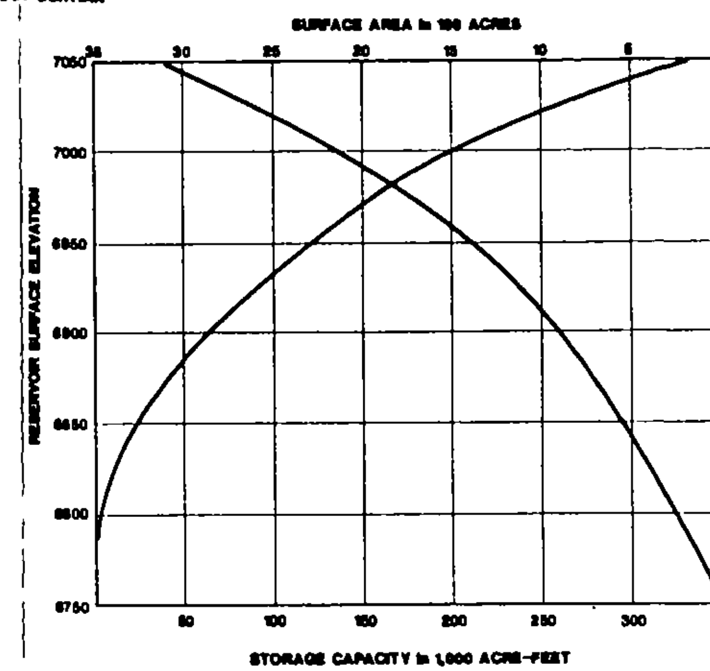
SECTION A-A PROFILE ALONG CENTERLINE of DAM



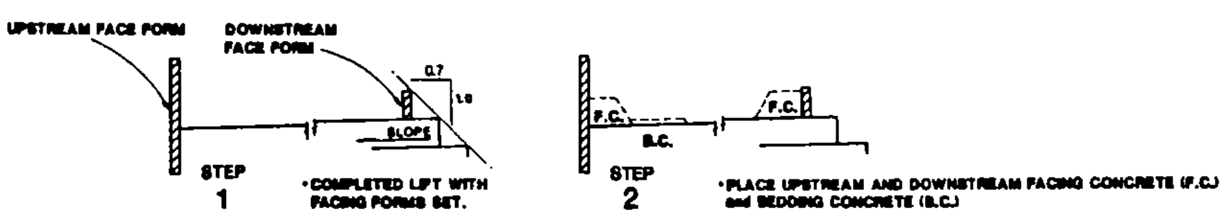
SECTION B-B SPILLWAY (150,000 Ac-Ft)



SECTION C-C OUTLET WORKS (150,000 Ac-Ft)



RESERVOIR AREA/CAPACITY



PLACEMENT SEQUENCE FOR RCC and FACING CONCRETE

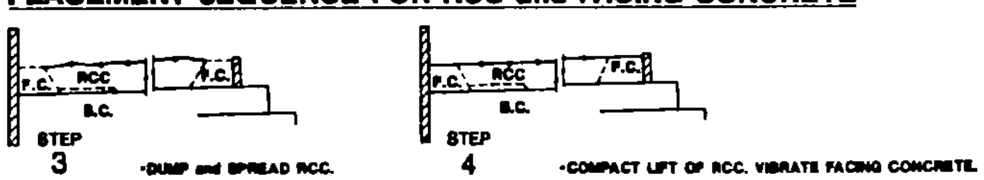


TABLE of ESTIMATED QUANTITIES

ITEM	RESERVOIR SIZE in ACRE FEET		
	50,000	150,000	300,000
COMMON EXCAVATION (Cu. Yd.)	285,460	483,400	597,220
ROCK EXCAVATION (Cu. Yd.)	18,430	63,040	124,630
RCC (Cu. Yd.)	391,890	918,970	1,527,500
GROUTING (Lf.)	2,025	4,025	6,587
STRUCTURAL CONCRETE (Cu. Yd.)	15,140	15,140	15,140
LAND ACQUISITION (Ac.)	1,055	2,226	3,276
HIGHWAY RELOCATION (Mi.)	5.3	7.5	8.1

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WHITE RIVER GEOTECHNICAL STUDY
RCC DAM/SECTIONS
WARNER POINT DAM

IECO NO.
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Plate 22

TABLE 14

WARNER POINT: RCC Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)
50,000	6915	160
150,000	6998	243
300,000	7061	306

deep as 75 feet in some places to remove the thin weathered zones of siltstone. It is possible, however, that a comprehensive sampling and testing program of the weathered rock zone during final design might reveal that the rock is sufficiently stable to provide an adequate foundation for the gravity dam. If this were the case, then the excavation, RCC, and backfill quantities would be significantly reduced. A grout curtain will be necessary along the entire length of the dam.

The RCC dam outlet works and spillway conform to the design described in Section 8.2.

The detailed cost estimates for the RCC alternative at Warner Point are included in Appendix H and are summarized in Table 15.

10.2.8 Finding and Conclusions

The most significant finding of the geotechnical investigation at the Warner Point damsite was that the landslides on the left abutment are surficial, extending to 50 feet in depth and do not appear to affect the feasibility of the project. The slide material would have to be completely removed to provide an adequate foundation for any dam design.

TABLE 15

WARNER POINT: RCC Dam Costs

	STORAGE CAPACITY (Acre-Feet)		
	50,000	150,000	300,000
Estimated Cost	$23,467,000	$41,327,000	$59,982,000

The alluvial gravel in the valley bottom is thicker than had been previously expected. The depth to bedrock ranges from 50 to 75 feet below the valley floor. This significantly affects the economics of the project, especially the RCC alternative.

The upper bedrock zone underlying the left abutment consists of a sequence of thick hard sandstone layers separated by thin weathered siltstone beds to a depth of 75 feet. The zone has been characterized as weathered rock by the seismic refraction survey, but in actuality the majority of the rock is sound except for the weathered zones that dip about 15^{o} toward the river. For the design and cost estimate of this report, the weathered rock was assumed to be an adequate foundation for the embankment dam, but unacceptable for the RCC alternatives. Any further geotechnical investigation at the Warner Point site would have to determine the continuity and strength of the weathered beds by sampling and testing. A detailed stability analysis of the embankment or gravity dam design would be necessary. It can be concluded from the present subsurface investigation, however, that a suitable foundation for either an embankment or gravity dam exists at a reasonable depth at the Warner Point site.

Adequate construction materials are located within close proximity for either an embankment or RCC dam. Alluvial sand and gravel for use as shell material in an embankment dam or for RCC aggregate is available in the river valley and in terraces along the right side of the reservoir. Impervious core material for an embankment design is available less than one mile north of the site.

The slopes of the reservoir created by a dam at Warner Point are considered to be stable, although further studies and surface instrumentation is indicated on the left side of the reservoir upstream of Elk Creek. The biggest question regarding the Warner Point Project is the significance of outcrops of the evaporitic facies of the Eagle Valley-Minturn Formation within the reservoir. This aspect of project feasibility should be the subject of future, more detailed studies. It is felt that the evaporitic deposits are irregular, discontinuous layers; and it is, therefore, unlikely that a single bed occurs that would connect outcrops within the reservoir with a daylight point more than five miles downstream.

The estimates show that except for the 50,000 acre-foot reservoir, the cost of an embankment dam is about equivalent to the RCC gravity dam design. The 50,000 acre-foot RCC dam is 25 percent less expensive than the embankment at a similar size because of the costs associated with constructing the spillway on the left abutment. This differential might disappear, however, if the centerline were shifted a short distance downstream to a more favorable alignment for a low dam.

The Warner Point Dam compares very favorably from the economic standpoint with the other projects studied on the White River. It ranks as the least expensive site for the 50,000 acre-foot and 150,000 acre-foot storage capacities and second to the Powell Park Dam for the largest 300,000 acre-foot reservoir. Conclusions regarding the relationship of one site to another are discussed in Section 11.

10.3 CHOKE CHERRY DAM

10.3.1 History

The Choke Cherry site was identified by a USBR geologist during his cursory site reconnaissance in 1984. The site is located at the lower end of the narrow steep section of the river in the vicinity of Warner Point. No subsurface information was available but surficial observations indicated that the left abutment at the site was free of any apparent massive landslides.

10.3.2 Reservoir Characteristics

Plates 23A and 23B shows the location of the Choke Cherry Dam and the area that would be flooded by reservoirs of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. The Choke Cherry reservoir would have many of the same characteristics as the Warner Point reservoir, as it occupies the downstream end of the same narrow stretch of the upper canyon. The steep gradient through that reach of the river results in a higher dam being required than at Warner Point to store an equal volume of water. Therefore, the extent of the reservoir and inundated areas would be almost identical to that described in Section 10.2.2 for Warner Point. The Choke Cherry Reservoir would, however, flood the steeper canyon with its potentially unstable slopes on the south side of the river just downstream from Warner Point. This area has been subdivided into privately owned lots as small as one acre.

10.3.3 Site Geology

Plates 4 and 5 are geologic maps covering the Choke Cherry area and Plate 24 shows the surficial geology at the damsite. The Choke Cherry site is located entirely within the Maroon Formation, as is Warner Point. However, at Choke Cherry the river does not run parallel to the strike of the bedding which is generally N70°E with a dip of approximately 15° to the northwest. The left abutment is steep and sparsely vegetated, with a few outcrops of in-place

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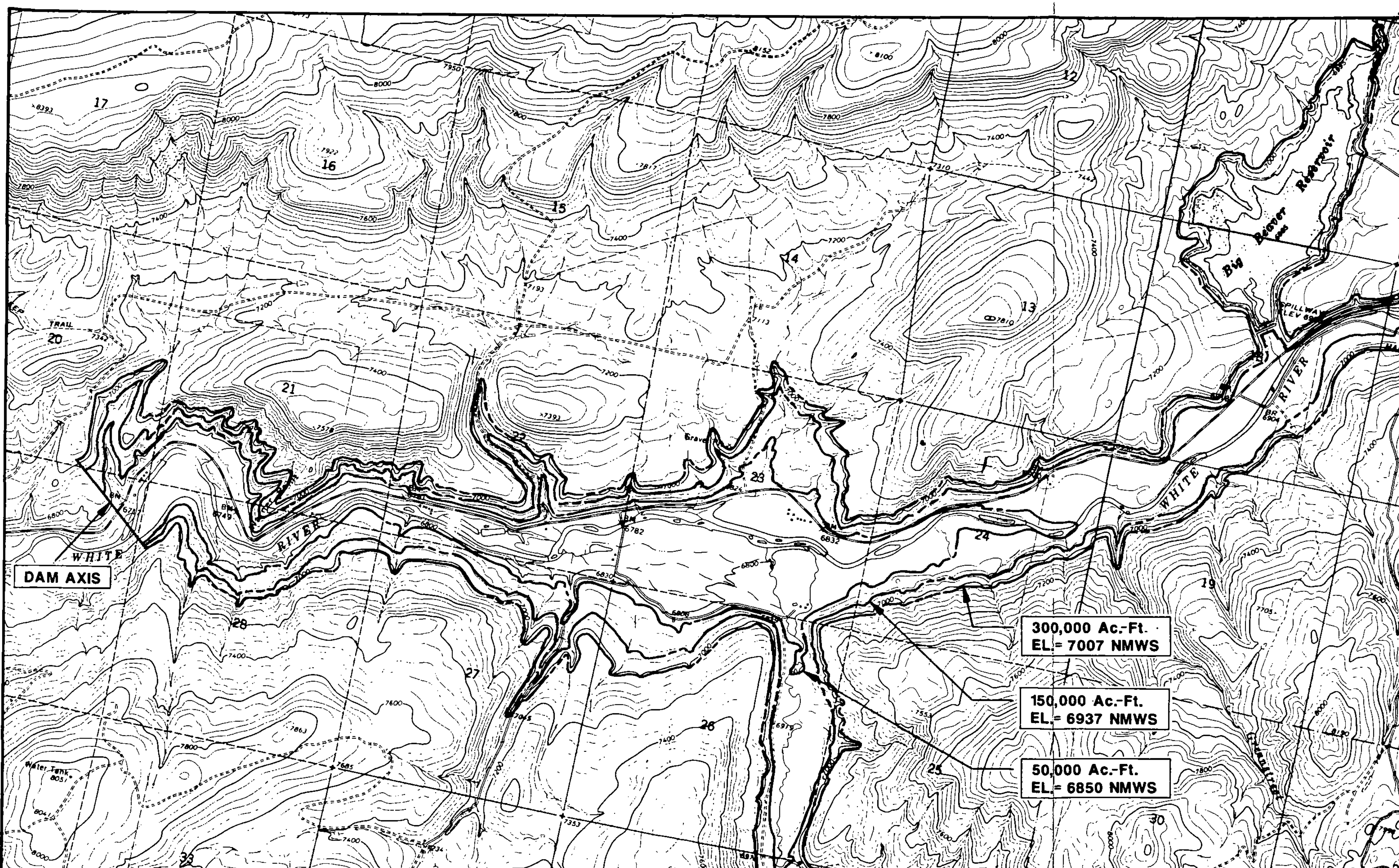

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WHITE RIVER GEOTECHNICAL STUDY RESERVOIR AREAS
CHOKE CHERRY DAM

IECO NO
SHEET OF REV.
Plate 23A

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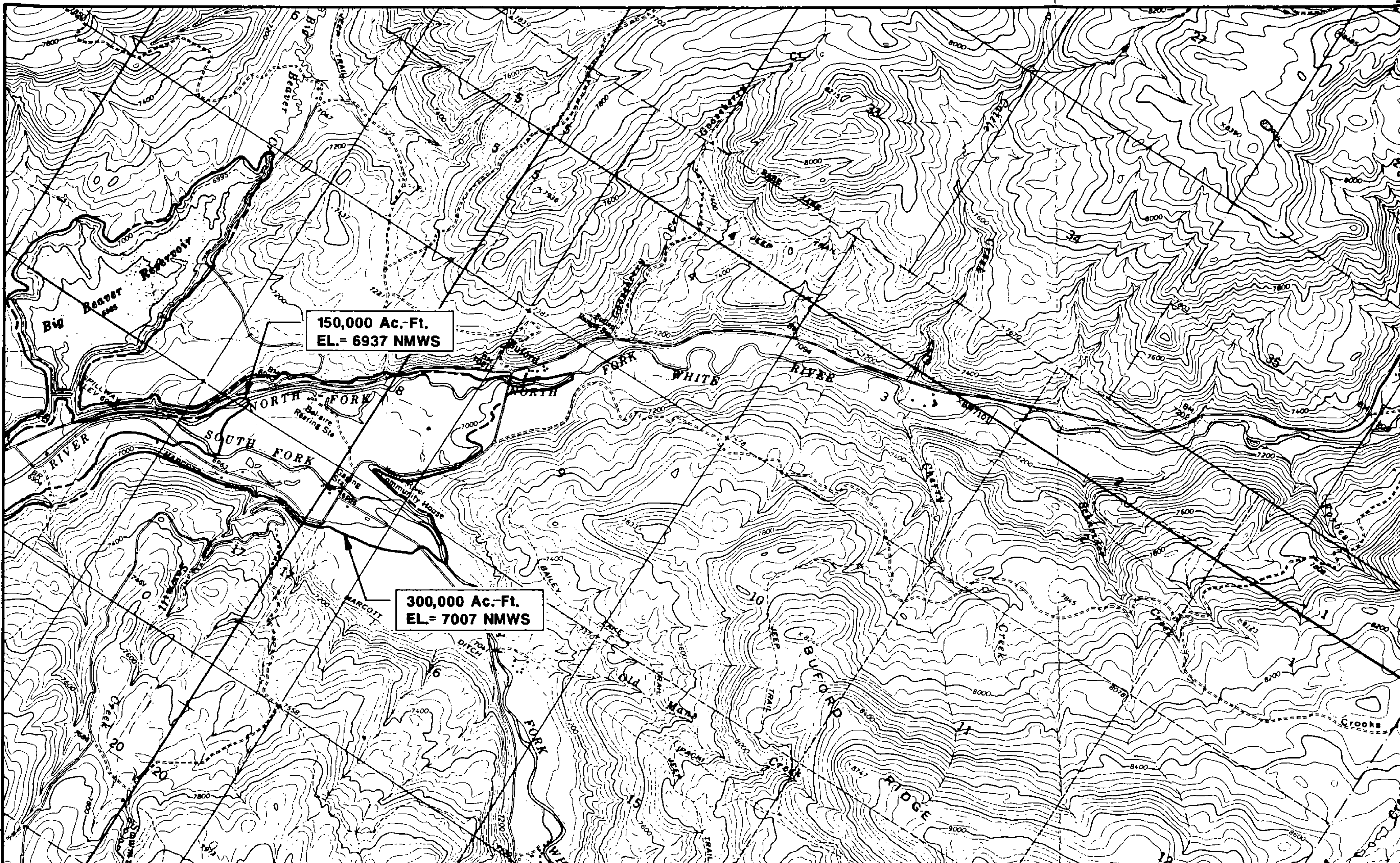

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WHITE RIVER
GEOTECHNICAL STUDY
RESERVOIR AREAS
CHOKE CHERRY DAM

IECO NO.
SHEET OF REV.
Plate 23B

bedrock mappable on the higher elevations. Most of the slope is covered by slope wash and the base is buried in colluvial debris to a depth of up to 50 feet.

The lower part of the right abutment is formed by steep outcrops of gently dipping sandstone of the Maroon Formation. A short distance up the abutment, however, the steep slope gives way to a wide, gently sloping bench covered with terrace deposits of sand and gravel. Further up the abutment, the topography is typical of the Maroon Formation, with a series of steep cliffs and benches culminating in a strike ridge formed by the resistant Weber Sandstone.

Several gentle folds were mapped in the Maroon Formation in road cuts near the Choke Cherry site, but they represent only minor ripples in the overall structure at the site. A normal fault striking to the northeast was mapped just downstream of the damsite on the right abutment, but does not appear to impact site feasibility.

Jointing of the bedrock at Choke Cherry is regional and identical to that described in Section 10.2.3 for Warner Point. Three principal subvertical sets were mapped, two of tectonic origin striking N55^{o}W and N25^{o}E, and the third set is caused by stress-relief parallel to the valley. Gently dipping joints also occur, frequently parallel to the bedding.

The valley bottom is formed by alluvial sand and gravel, representing glacial outwash composed of well-rounded gray granite, gneiss, and quartzite, black-to-purple basalt, and some minor subrounded cobbles of red sandstone from the Maroon Formation. The alluvium is partially covered at the base of each abutment by colluvial debris composed of blocks of hard red sandstone in a silty sand matrix.

The reservoir formed by a dam at Choke Cherry would back up water over the same geologic units described at Warner Point except the upstream limits of the reservoir would not reach the evaporite units and sink holes near Buford.

The reservoir, however, would flood the base of the landslide deposits mapped on the south side of the river between Warner Point and Choke Cherry, and this might be of concern to future slope stability within the reservoir.

10.3.4 Foundation Investigation

After the initial site screening of the upper canyon sites, no further studies were scheduled for Choke Cherry because of its poor economic comparison with the other sites. However, after the initial drilling at Warner Point revealed some unexpected conditions, it was decided to investigate Choke Cherry as well.

The subsurface investigation at Choke Cherry Dam was limited to one rotary borehole and seismic refraction surveys along the centerline, across the alluvial valley floor downstream of the dam, and along a potential spillway alignment on the right abutment. Three drill holes had been planned at Choke Cherry, but based on the results of the first hole, the remaining two holes were transferred back to Warner Point Dam where they had been scheduled originally. The location of the boreholes and seismic lines are shown in plan on Plate 24 and in section on Plate 25.

Hole CC-C1 was drilled vertically, approximately in the center of the valley floor just downstream of the proposed centerline of the dam. The hole penetrated 71 feet of alluvial sand, gravel, and boulders before encountering bedrock of the Maroon Formation. The bedrock consists of reddish brown-to-pink interbedded sandstone and siltstone, slightly weathered and moderately-to-highly fractured. Most of the joints were open, oxidized bedding plane joints, a few filled with calcite. Most of the high-angle joints detected showed a wide range of strike generally to the northeast, with a few corresponding to the principal regional set to the northwest. Water pressure tests showed the rock to be of low permeability in the 10^{-5}cm/sec range.

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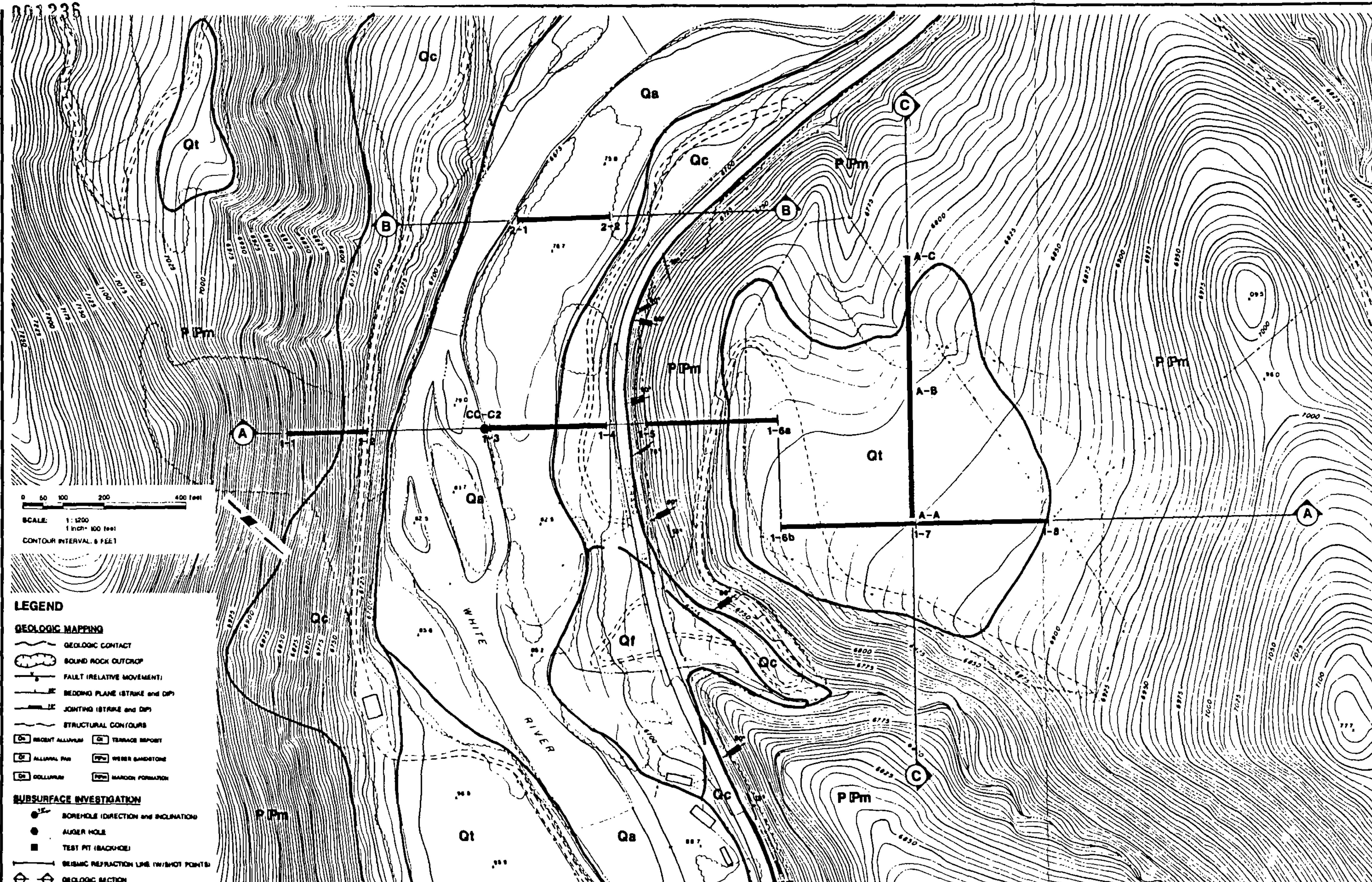

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WHITE RIVER GEOTECHNICAL STUDY

GEOLOGIC MAP/LOCATION PLAN

CHOKE CHERRY DAM

SHEET OF REV

Plate 24

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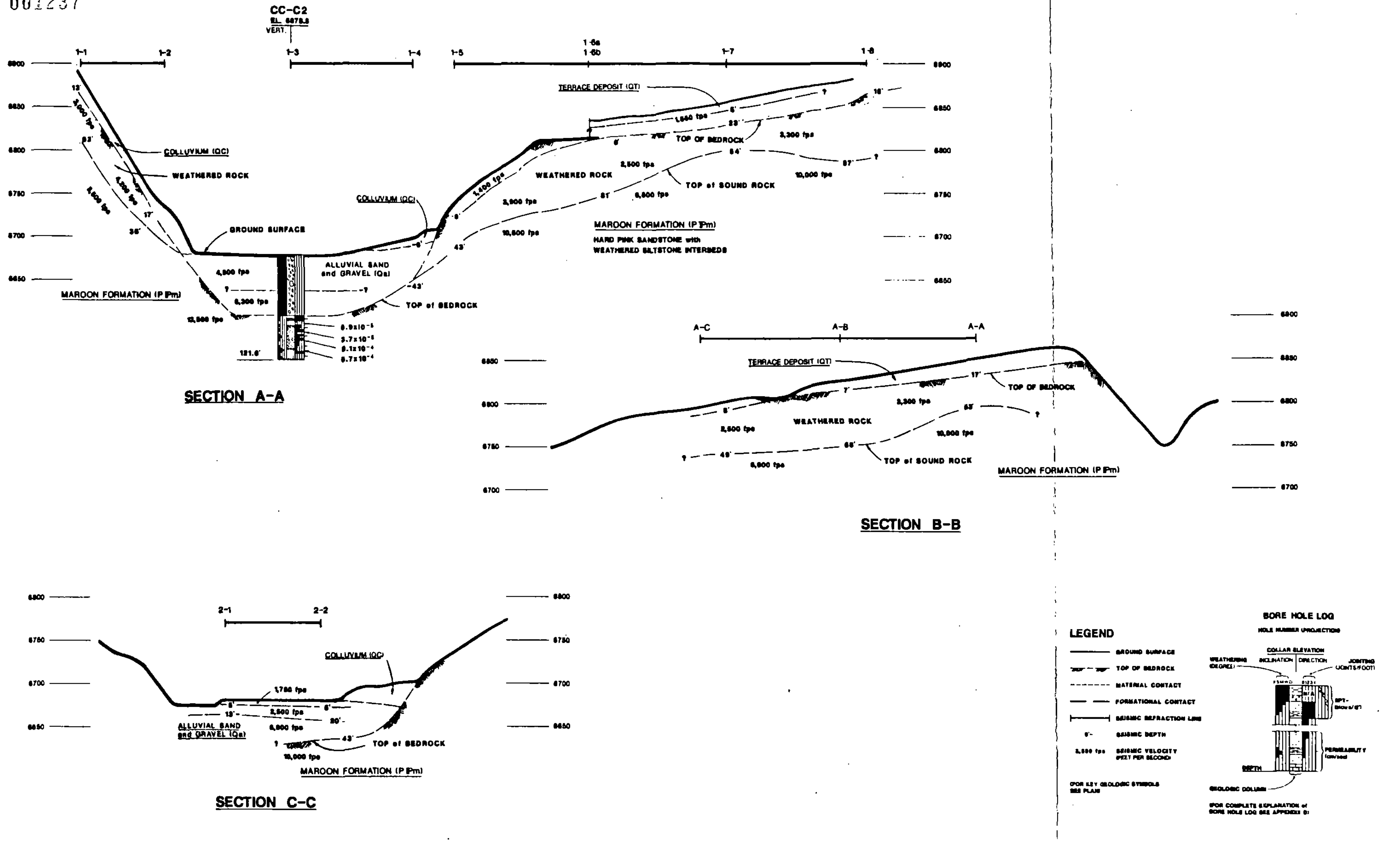

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WHITE RIVER GEOTECHNICAL STUDY
GEOLOGIC SECTIONS
CHOKE CHERRY DAM

IECO NO
SHEET OF REV
Plate 25

At 95 feet, the boring passed into hard, dark gray limestone, moderately jointed, with a network of small oxidized solution cavities below 105 feet. Surficial geologic mapping often detected a similar limestone layer at the base of the Maroon Formation which was mapped as the contact with the underlying Eagle Valley-Minturn complex that occasionally displayed evaporitic facies. Although the limestone is porous, it is not highly permeable; the pressure tests registered 6.7 to 8.1×10^{-4} cm/sec. There was some evidence of washing of filling from the solution channels during the test.

The limestone layer is underlain by a fine, weak, gray sandstone unit, very poorly cemented, and in some cases reduced to a cohesionless oxidized sand, orange in color. Interbedded in this unit are thin coal seams and purple siltstone stringers. The hole was terminated at 121 feet due to the poor foundation characteristics of the rock and the potential of encountering anhydrite and gypsum in the Eagle Valley-Minturn Formation.

Seismic line 1-1 to 1-2 on the left abutment detected a layer of colluvium or slope wash with a velocity less than 1700 fps from five to 13 feet thick overlying bedrock. The bedrock is apparently weathered with a velocity of only 3000 to 4000 fps. Although no holes were drilled in this area, this velocity corresponds to that of the weathered rock in the left abutment at Warner Point, which was 90 percent sound sandstone interbedded with highly weathered siltstone. Sound rock with a velocity of 11,500 fps was detected at a depth of 36 feet at the base of the slope and 85 feet deep further up the slope.

The gently sloping right abutment was investigated with seismic lines 1-5 to 1-6A and 1-6A to 1-8. No boreholes were drilled to correlate the data. The seismic lines showed the terrace deposit to be from five to 23 feet thick, with a velocity of less than 1500 fps. The overburden is underlain by weathered rock, probably of the same nature as described above for the left abutment, with a seismic velocity of 2500 to 3900 fps. Sound bedrock with a velocity of 6800 to 10,500 fps was located at a depth ranging from 43 feet near the river to 87 feet up the right abutment.

Seismic lines 1-3 to 1-4 and 2-1 to 2-2 in the valley bottom detected a change in velocity in the alluvium at a depth ranging from 20 to 39 feet. This zone is represented by velocities in the 2500 to 4800 fps range which generally corresponds with the alluvial sands and gravels logged at other sites. Between this zone and the bedrock, which underlies the valley at depths up to 74 feet, there is an alluvial layer width velocity ranging from 6900 to 8300 fps. It probably consists of large boulders, although borehole CC-C2 did not penetrate any. The bedrock velocity was 10,000 to 12,500 fps.

10.3.5 Construction Materials Investigation

It was hoped that Choke Cherry would be an RCC damsite, but the depth of alluvial gravel and the quality of the foundation preclude that alternative. Materials of acceptable quality and quantity for an embankment dam, however, were found in close proximity to the dam.

Excavation of a side channel spillway would produce a moderate quantity of weathered and sound rock acceptable for riprap and for use in the shell zone. River gravel is also available for use as shell material and could be processed to produce concrete aggregate and filter and drain material. No test pits were excavated or samples taken of the required excavation or in the alluvial gravels.

A source of impervious core material was located less than one mile north of the site. The borrow area is the western end of the gentle, perched valley, parallel to the White River described in Section 10.2.5 and shown in Plate 18 as a borrow area for Warner Point Dam. The material is a red sandy clay of low plasticity derived from weathering, erosion, and transport of siltstone and claystone of the State Bridge and Chinle Formations. Several test pits were excavated by backhoe in the area, showing a depth of usable material from eight to 12 feet, the maximum depth achievable with a backhoe. Samples were obtained from the test pits for laboratory testing. Test results are included in Appendix E and summarized in Table 11 of Section 10.2.5.

10.3.6 Engineering Geology

The single boring at the Choke Cherry Dam Site indicates the foundation to be of questionable quality. Although the foundation bedrock immediately underlying the alluvial gravel in the valley floor at 71 feet is sound sandstone and siltstone of the Maroon Formation, the contact with the unpredictable and potentially dangerous Eagle Valley-Minturn Formation lies only about 25 feet below. Although the borehole at Choke Cherry did not recover soluble rock such as gypsum and anhydrite, it did penetrate about 20 feet of porous limestone and below that several zones of weathered, unconsolidated sand. Although it is certainly possible to design an embankment dam at the site, it is considered dangerous due to the proximity of the foundation to those undesirable rock units.

The left abutment at Choke Cherry is free of the landslides present at Warner Point, but is covered with approximately 10 feet of slope wash and colluvium overlying weathered rock. The weathered rock zone is probably, similar to Warner Point, consisting of about 90 per cent sound sandstone interbedded with thin weathered layers of siltstone. This weathered rock would provide a suitable foundation for an embankment dam.

A positive cutoff to bedrock would have to be excavated to a depth of at least 75 feet across the valley floor. The temporary cut-off trench slopes would be stable at 1H to 1V if seepage inflow were adequately controlled during construction. The river beds and gravel would provide adequate foundation for the shell of the embankment dam. The shell zones could be founded on the terrace deposits of the right abutment, but a positive cut-off trench up to 23 feet deep would be required to the top of bedrock. A shallow cutoff trench would be provided into the outcropping bedrock further up the abutment foundation depending on local weathering of the weak siltstone interbeds.

A grout curtain would be essential at the Choke Cherry site along the entire length of the dam. Special attention would have to be given to grouting the foundation under the river valley where the grout holes would penetrate the upper part of the Eagle Valley-Minturn Formation.

Excavation of the spillway channels on the right abutment does not appear to be a problem in terms of slope stability. The channel alignments trend toward the southeast and would be perpendicular to the principal set of vertical joints. The bedding dips into the right abutment and would probably not present a problem. Any stress relief joints that have developed parallel to the valley would not be a factor because the channels are laid out well back into the abutments where rebound has probably not occurred.

The questions of reservoir slope stability and permeability more serious at Choke Cherry than at Warner Point. The Choke Cherry reservoirs cover essentially the same area upstream as the Warner Point reservoirs, but in addition, would inundate the steep canyon section between the two dams. The reach of river immediately upstream would include the large area mapped as landslides on the south side of the river directly opposite Warner Point. These ancient slides could be reactivated by a rapid drop in reservoir elevation that would produce a build-up in pore pressure in the saturated slopes. A rapid massive landslide into the reservoir in such close proximity to the dam could cause a wave that could overtop the embankment and result in erosional failure.

The reservoir seepage problem at Choke Cherry would be similar to Warner Point except for the potential for a shorter seepage path through solution channels in the foundation directly under the dam. Even though no readily soluble rock was found in the one bore hole that was drilled, it is possible that such rock does occur nearby. Furthermore, although the drill hole showed a protective cap of Maroon Formation bedrock over the potentially soluble zone, it is conceivable that a short distance away a tongue of evaporites might occur at a higher elevation or the river may have cut a deeper channel. Either one of these possibilities would provide direct access of reservoir water through the soluble foundation to an exit immediately downstream of the dam. The potential for failure posed by both the reservoir stability and seepage problems should discourage any further consideration of the Choke Cherry Dam site.

10.3.7 Design and Cost Estimate

Based on the foundation and materials information gathered from the geotechnical investigation, appraisal-level designs and cost estimates were prepared for dams at the Choke Cherry site with storage capacities of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. Preliminary comparisons between embankment and roller compacted concrete gravity sections showed the gravity dam to be not economically competitive with the embankment. This was due to the extent of the weathered rock zone that would have to be removed from the right abutment. Therefore, design and cost estimates were prepared only for the embankment.

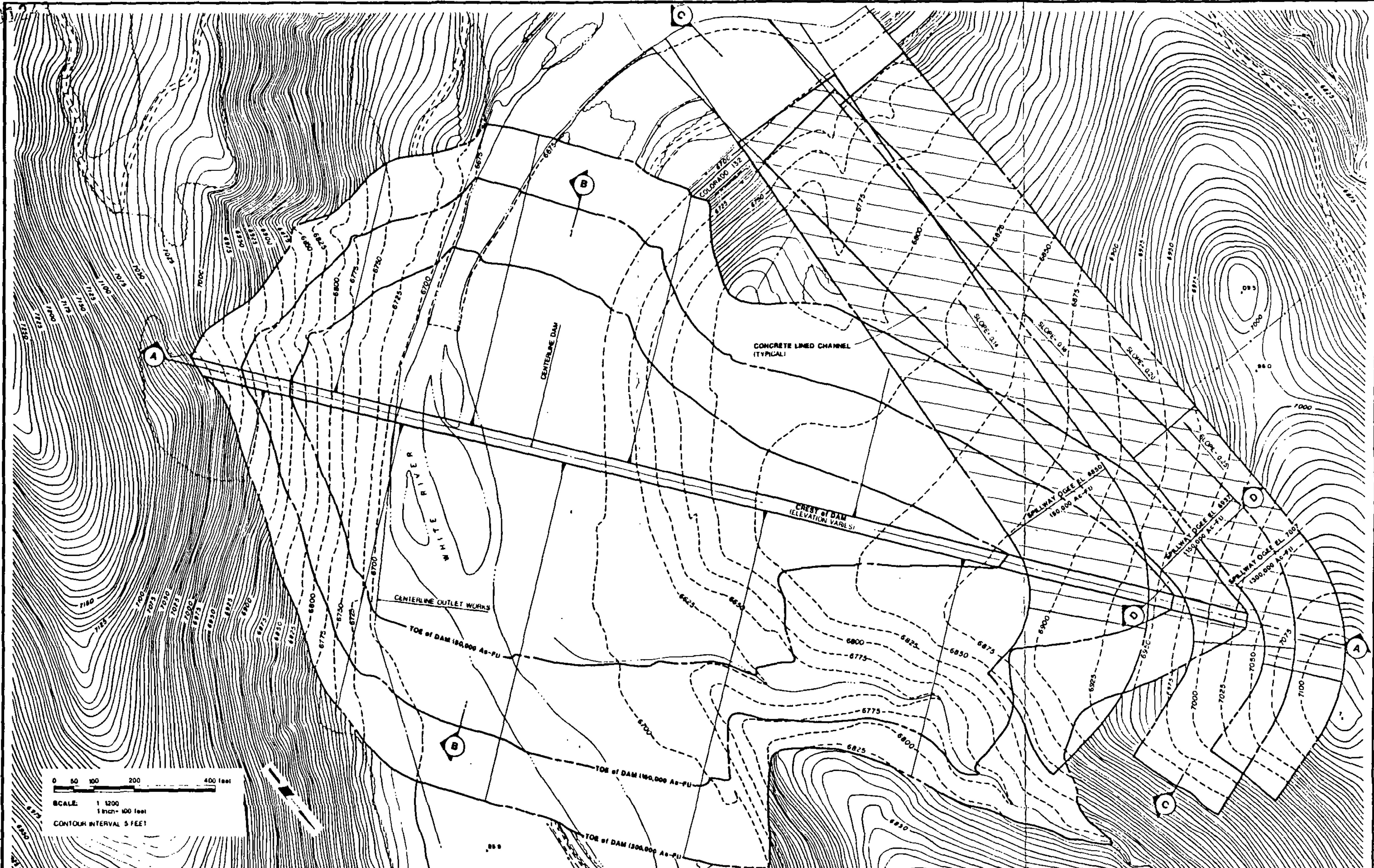
The dam designs are shown in plan on Plate 26 and in section in Plate 27. The three dam sizes would have crest elevations, heights above river bed, and normal maximum water surfaces as shown in Table 16.

TABLE 16

CHOKE CHERRY: Embankment Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)	NMWS Elevation (ft msl)
50,000	6879	189	6850
150,000	6966	276	6937
300,000	7036	346	7007

Foundation preparation would consist of complete removal of the colluvial debris and slope wash present on both abutments, and excavation of a cutoff trench at least 10 feet deep into the top of the weathered bedrock. A cutoff



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WHITE RIVER GEOTECHNICAL STUDY

EMBANKMENT DAM/PLAN

CHOKE CHERRY DAM

SHEET	OF	REV
Plate 26		

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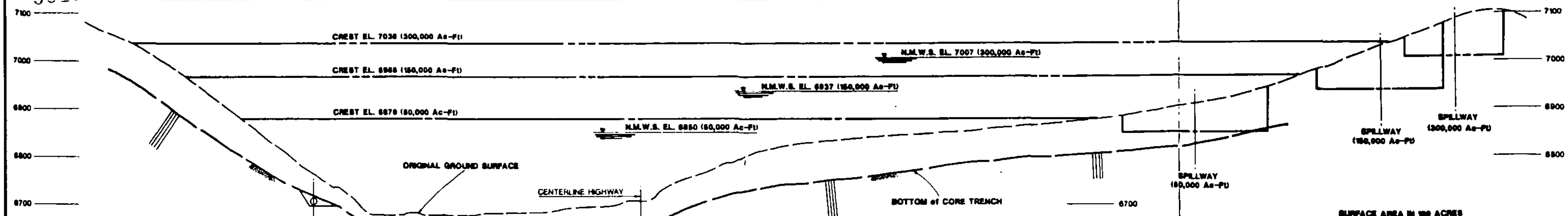
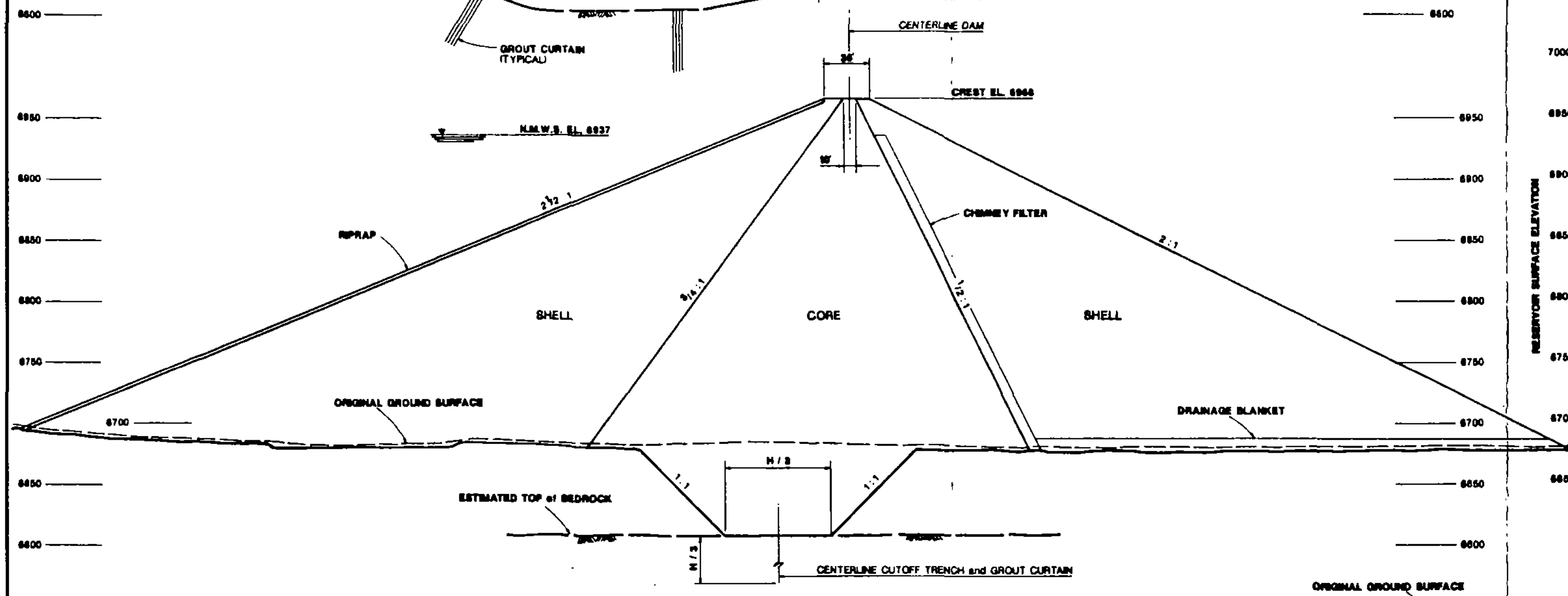
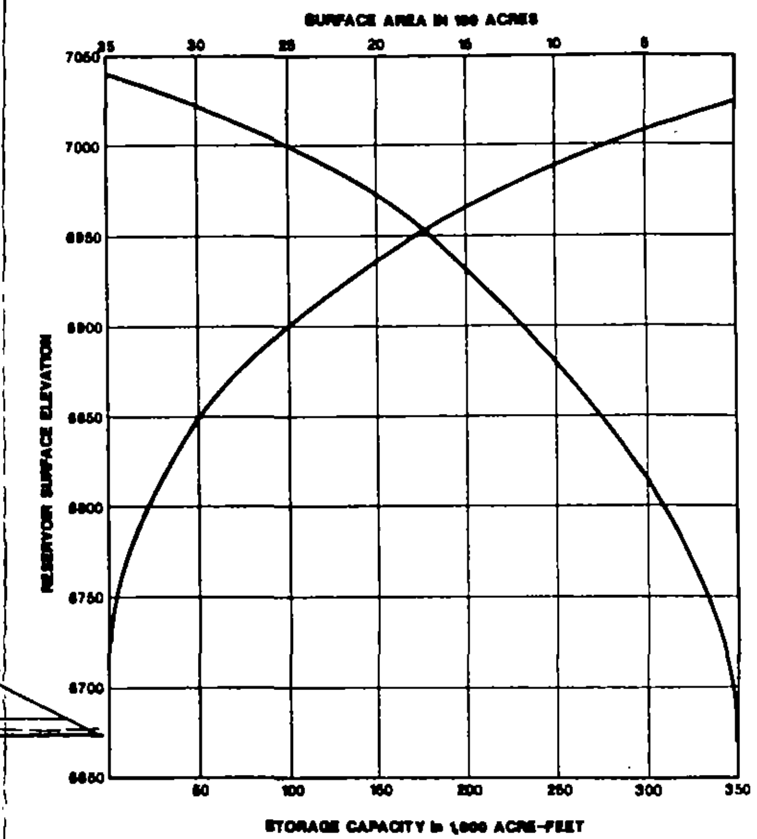


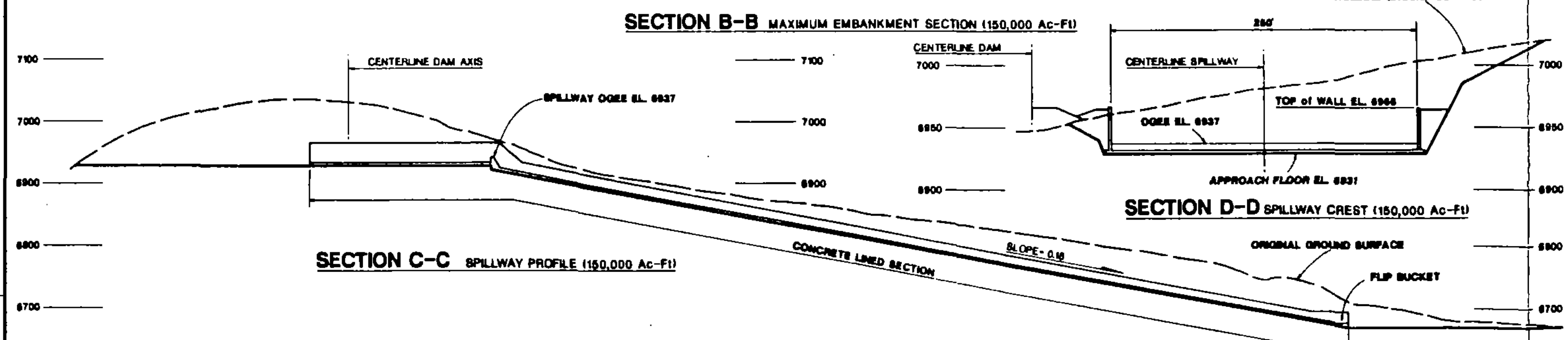
SECTION A-A PROFILE ALONG CENTERLINE of DAM



SECTION B-B MAXIMUM EMBANKMENT SECTION (150,000 Ac-Ft)



RESERVOIR AREA/CAPACITY



SECTION C-C SPILLWAY PROFILE (150,000 Ac-Ft)

SECTION D-D SPILLWAY CREST (150,000 Ac-Ft)

TABLE of ESTIMATED QUANTITIES

ITEM	RESERVOIR SIZE in ACRE FEET		
	50,000	150,000	300,000
COMMON EXCAVATION (Cu. Yd.)	892,473	1,484,776	1,767,584
ROCK EXCAVATION (Cu. Yd.)	714,186	806,378	862,851
CORE (Cu. Yd.)	1,113,512	2,573,061	4,406,661
SHELL (Cu. Yd.)	2,354,683	5,997,972	10,691,551
RIPRAP (Cu. Yd.)	58,686	112,908	150,711
GROUTING (Sh.)	6,465	8,433	9,467
STRUCTURAL CONCRETE (Cu. Yd.)	30,230	32,495	31,970
LAND ACQUISITION (Ac.)	1,010	1,931	3,407
HIGHWAY RELOCATION (Mi.)	6.3	6.8	7.2

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WHITE RIVER GEOTECHNICAL STUDY
EMBANKMENT DAM/SECTIONS
CHOKE CHERRY DAM

IECO NO
SHEET OF REV.
Plate 27

trench through the river alluvium to the top of bedrock would be required in the valley floor. The upstream and downstream shells would be founded on in-place alluvial gravel and weathered rock. A grout curtain is necessary along the entire length of the foundation due to the jointed nature of the bedrock.

The embankment would be a zoned earthfill structure conforming to the designs outlined in Section 8.1. The impervious core material would be derived completely from the borrow area described in Section 10.3.5. The shell zones would be constructed of colluvium, weathered rock, and gravel derived from required excavation of the dam foundation and spillway, and complemented by borrowed sand and gravel as necessary. The upstream slope would be protected with riprap produced from rock excavated for the spillway chute.

The outlet works and spillway design are described in Section 8.1. Details are shown in Plate 9. The side channel spillways were laid out through the right abutment for all dam heights. The concrete-lined chutes extend to the existing river channel where discharge energies would be dissipated by a flip bucket.

The detailed cost estimates of the Choke Cherry Project are included in Appendix I and are summarized in Table 17.

10.3.8 Findings and Conclusions

The only borehole drilled at the Choke Cherry site revealed the stream bed alluvial gravel to be at least as deep as at Warner Point. The borehole also revealed a thickness of only about 25 feet of sound sandstone of the Maroon Formation overlying poorer quality bedrock of the top of the Eagle Vally-Minturn sequence. Although no gypsum or anhydrite was recovered

TABLE 17

CHOKE CHERRY: Embankment Dam Costs

	STORAGE CAPACITY (Acre-Feet)		
	50,000	150,000,000	300,000,000
Estimated Cost	$34,369,000	$53,079,000	$70,906,000

as core, surficial mapping indicates these materials to be associated with this formation, making it a very unfavorable foundation for a large water storage facility.

No boreholes were drilled on the right abutment; but the seismic refraction study showed the weathered rock zone to extend over the entire length of the abutment to depths up to 100 feet. The amount of excavation required to provide a suitable foundation for an RCC gravity dam eliminated the feasibility of that type of design.

Adequate construction materials for an embankment dam are located close to the site. Alluvial sand and gravel for use in the shell zones is abundant in the river valley both upstream and downstream of the dam. Impervious core material is available from the same borrow source as for Warner Point, less than one mile north of the site.

The slopes of the reservoir that would be formed by the construction of Choke Cherry dam are potentially unstable in the area of the surficial landslides along the left side of the reservoir between Choke Cherry and Warner Point.

Any further studies of a dam at Choke Cherry would have to consider subsurface investigation and stability analysis of these slopes.

Problems relating to reservoir permeability would be similar to those at Warner Point. The new reservoir would flood areas underlain by evaporite rocks of the Eagle Valley-Minturn unit. Future studies would have to determine the potential for leakage to occur through this formation, as it crops out downstream of the dam. Mapping during the present study, however, indicates the evaporite tongues to be discontinous, and it is unlikely that a single layer would have continuity over a distance of several miles.

The cost estimates show that the Choke Cherry dam is the next to the least economical of the four sites studied in the upper canyon section. For all sizes considered, only Veatch Gulch proved to be more expensive. Choke Cherry is most competitive at the 300,000 acre-foot size, at which it would cost about 22 percent more than the embankment dams at Warner Point and Canyon.

10.4 VEATCH GULCH DAM

10.4.1 History

A dam located on Veatch Gulch was originally studied by IECO in 1982 as an off-channel storage facility of the Yellow Jacket Project. It did not compare favorably with the other alternatives considered at that time and was not recommended further.

The USBR identified a potentially good damsite on the mainstem White River just upstream of Veatch Gulch as a result of their geologic review of the upper canyon sites in 1984. This site was selected for investigation under the present study because the canyon configuration appeared suitable for an RCC structure.

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10.4.2 Reservoir Characteristics

Plates 28A and 28B show the location of the Veatch Gulch Dam and the area that would be inundated by reservoirs of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. The Veatch Gulch Dam would form a long narrow reservoir, flooding approximately 2,670 acres of privately owned agricultural land. About 50 percent of the area is under irrigation, planted in hay. The remainder is steeper forage terrain characterized by scrub oaks, small pine trees, and sage. The largest reservoir considered would extend two miles up Miller Creek and six miles up the mainstem.

10.4.3 Site Geology

The Veatch Gulch site is located in a broad, symmetrical, rectangular-shaped valley. The geology of the area is shown on Plate 6 and of the damsite on Plate 29. The higher abutments are formed by near-vertical cliffs of orange sandstone of the Entrada Formation. The sandstone is uniformly medium-grained, cross-bedded, and poorly cemented. The contact with the underlying Chinle Formation strikes northeast and dips about 15° downstream to the northwest. The contact is located at approximately one-half the dam height for the largest dam sized. Below the contact, the slope is more gentle than above and covered with a thin veneer of slope wash.

The Chinle Formation is composed of massive deep red siltstone with indistinct bedding. Colluvium consisting of angular blocks of hard sandstone in a brown sandy silt matrix has accumulated at the base of the abutments and partially covered the alluvium of the valley floor. A shallow raveled landslide was mapped on the left abutment just upstream of the dam axis. The alluvial gravels and sands are composed of well-rounded cobbles of granite, gneiss, and quartzite, together with darker basalt fragments and an occasional piece of red sandstone from the Maroon Formation.

No evidence of folding or faulting was observed in the surficial mapping at Veatch Gulch. Jointing corresponds to the regional trend with a subvertical

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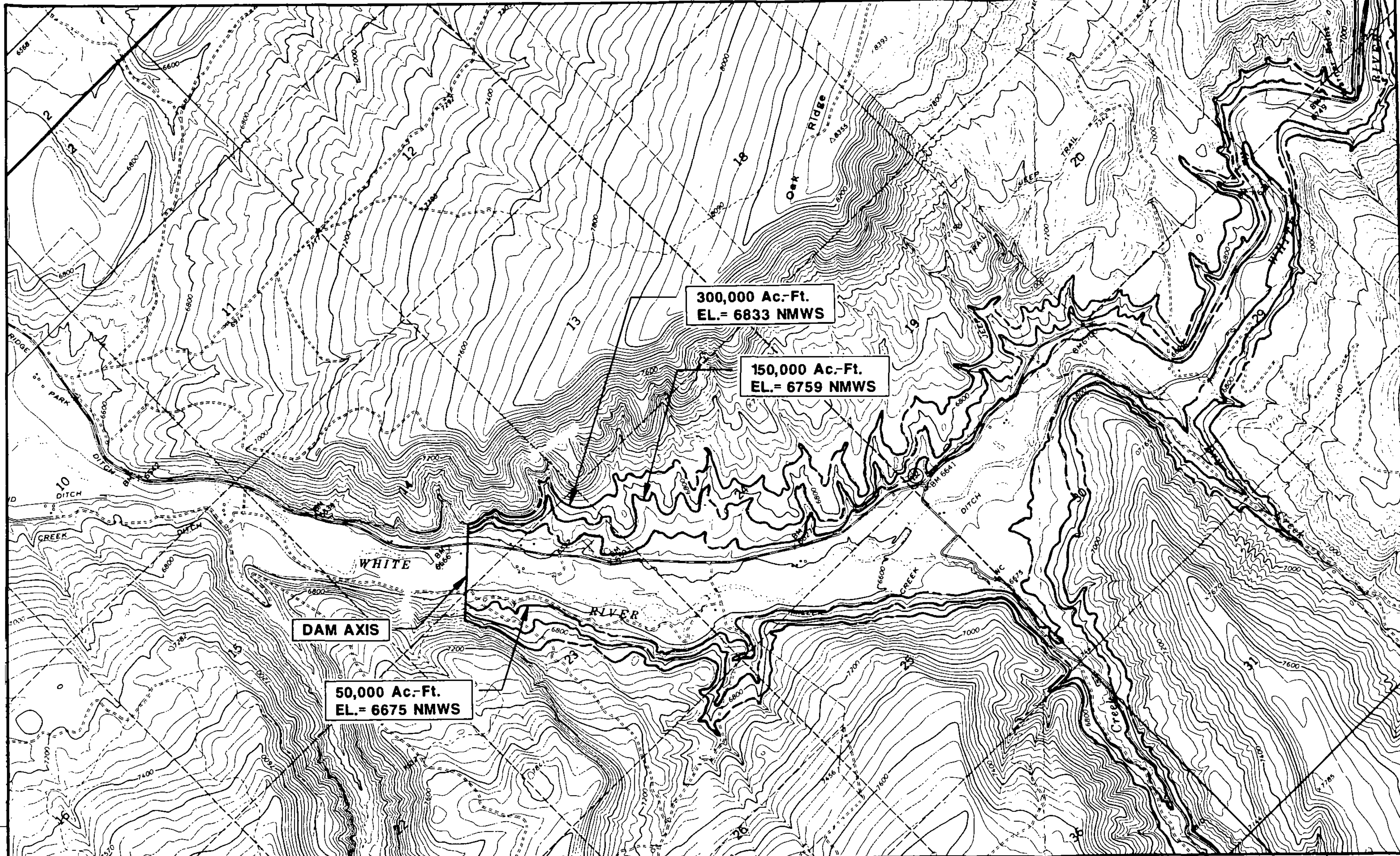

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WHITE RIVER GEOTECHNICAL STUDY RESERVOIR AREAS

VEATCH GULCH DAM

IECO NO

SHEET OF REV.

Plate 28A

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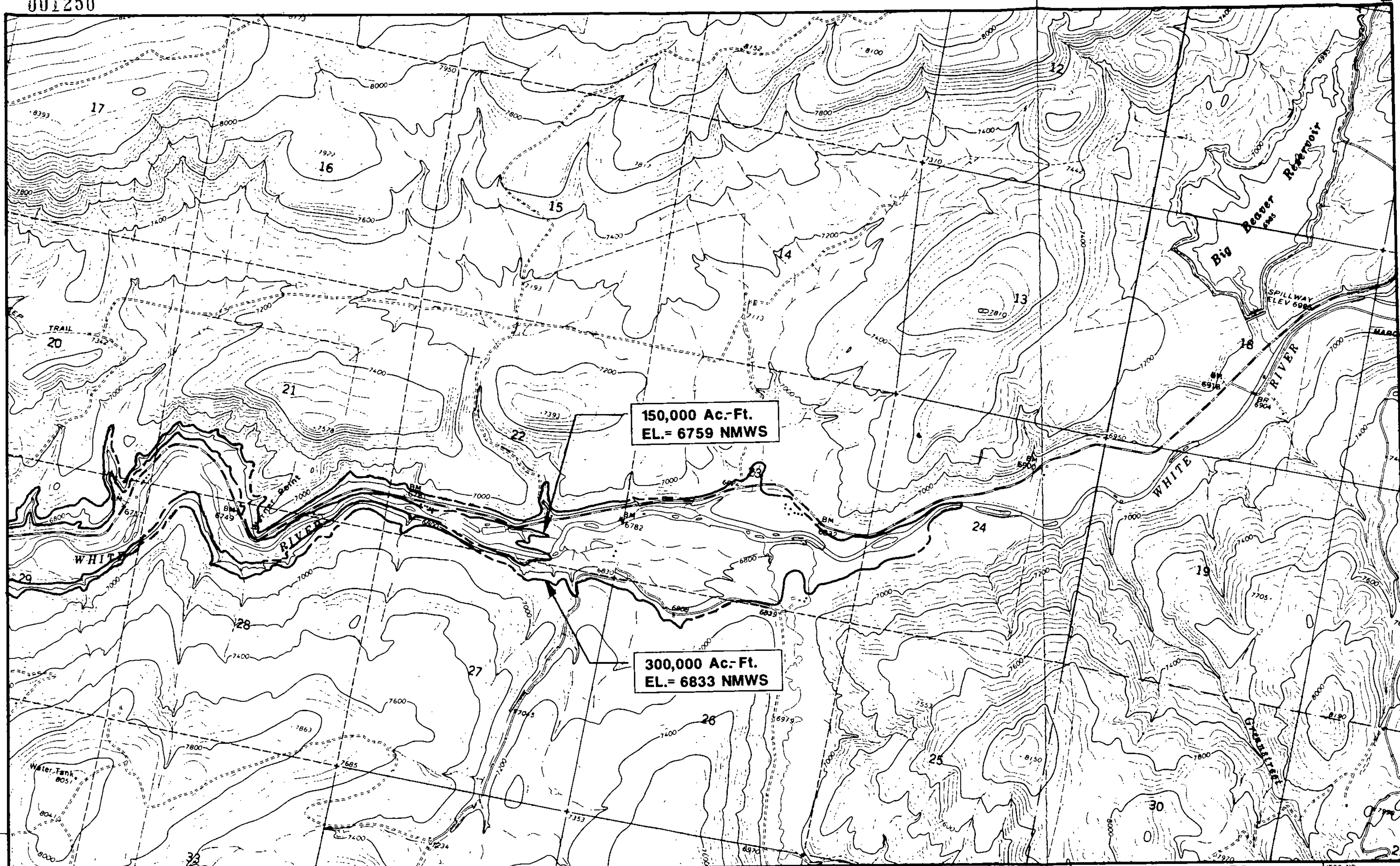

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VEATCH GULCH DAM

IECO NO
SHEET OF REV.
Plate 28B

001251

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WHITE RIVER GEOTECHNICAL STUDY
GEOLOGIC MAP/LOCATION PLAN
VEATCH GULCH DAM

SHEET	OF	REV
Plate 29		

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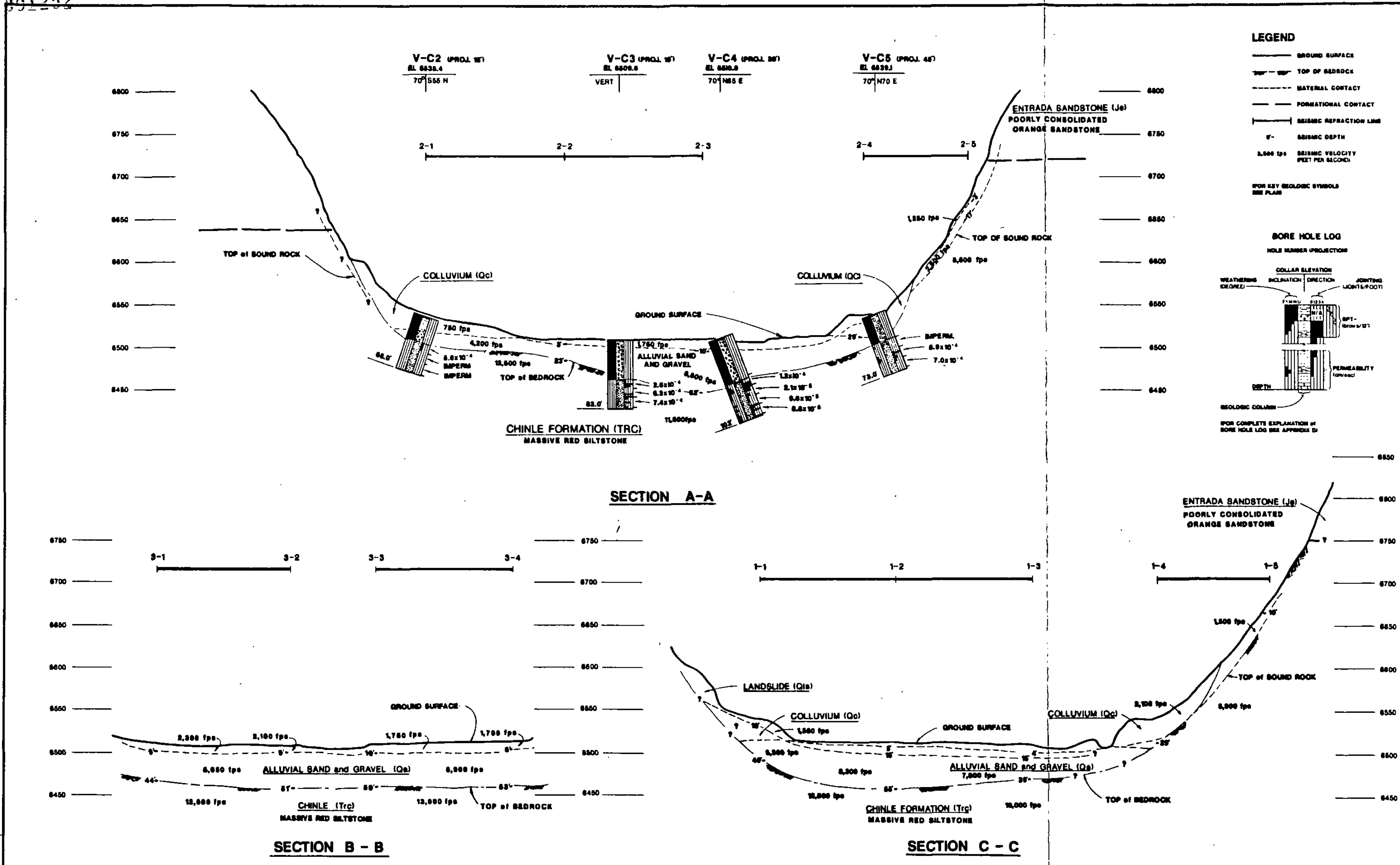

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GEOLOGIC SECTIONS
VEATCH GULCH DAM

IECO NO
SHEET OF REV
Plate 30

set striking northwest and another northeast. Jointing is well defined in the Entrada Sandstone cliffs but much less developed in the less brittle siltstone of the Chinle Formation. Stress relief joints parallel to the valley have formed in the Entrada Sandstone, resulting in the vertical cliffs, but little evidence of development of this joint set in the siltstone is visible. Bedding plane joints are not well developed in either of these units; the cross-bedding in the Entrada causes any partings that do occur to be discontinuous. The Chinle siltstone is compact, and bedding is not readily evident except at the contact and at weathered surfaces where stress relief has occurred parallel to bedding.

The Veatch Gulch Reservoir would inundate gently rolling topography formed mainly by the Chinle and State Bridge Formations. At the upper end of the reservoir, some bedrock of the Maroon Formation would underlie the reservoir.

10.4.4 Foundation Investigation

Originally eight boreholes were planned at the Veatch Gulch site, but when the decision was made to carry out subsurface work at the Canyon site as well, the number was reduced to five. Ultimately, only four holes were drilled. Hole V-C1 on the left abutment was not drilled because of possible damage to the Miller Creek Ditch; consequently, no subsurface exploration was done on the left abutment. Above the Miller Ditch, the abutment is a near-vertical cliff of outcropping Entrada Sandstone. Seismic refraction surveys were performed along the proposed dam centerline and parallel to the centerline upstream and downstream of the dam. The location of the boreholes and seismic lines are shown in plan on Plate 29 and in section on Plate 30.

Borehole V-C2 was located at the toe of the colluvial skirt at the base of the left abutment and was angled into the slope 70^{o}. The hole revealed a 15-foot thickness of colluvial tan sand derived from erosion of the sandstone cliffs above, covering 15 feet of alluvial sand and gravel. Bedrock is massive red siltstone of the Chinle Formation occasionally containing irregular gray limestone inclusions. Bedding was not clearly defined so the core could not be oriented. A few minor open joints together with some irregular calcite-filled joints were logged to a depth of 38 feet. Below that depth, the rock mass was unjointed. A permeability of 5.6×10^{-4} cm/sec was recorded in the upper 12 feet of bedrock. The test indicated a tendency toward plastic deformation of the joints or a washing out of weathered material. Pressure tests below 42 feet showed the rock to be impermeable.

Holes V-C3 and V-C4 were located in the valley floor. V-C3 was drilled vertically and V-C4 was inclined 70^{0} toward the right abutment. They revealed a depth of 46 and 51 feet of alluvial sand and gravel, respectively, overlying bedrock of the Chinle Formation. The rock was massive, sound, red-to-pink siltstone with occasional irregular limestone inclusions. Some core was lost in hole V-C4 possibly indicating weathered zones, but more likely because of excessive downpressure and circulation water. The core could not be oriented due to lack of distinct bedding. The rock was lightly jointed and the joints that were logged in V-C3 were generally dipping less than 20^{0}. V-C3 was not pressure tested; V-C4 showed a moderately low permeability of 1.2×10^{-4} cm/sec over the first 12 feet of bedrock. From 12 feet to the bottom of the hole at 102 feet, the rock mass permeability was low from 2.1 to 9.6×10^{-5} cm/sec.

V-C5 was located along the highway at the base of the near-vertical cliffs of the right abutment. The hole prenetrated 36 feet of colluvial debris derived from the cliffs of the Chinle Formation and Entrada sandstone above. The debris consisted of angular blocks of sound, pink sandstone surrounded by red silty sand. The bedrock was massive, sound red-brown siltstone of the Chinle Formation. As in the other holes, the core could not be oriented due to the lack of bedding. Jointing was light and permeability was moderate, 7.0×10^{-4} cm/sec, to impermeable.

The seismic refraction lines in the valley floor showed velocities for the alluvial gravel ranging from 4200 to 6800 fps. The variation is probably due to concentrations of boulders in certain strata. The bedrock contact below the alluvium was shown to be fairly uniform, ranging from 49 to 62 feet in depth over a large area. The bedrock velocity was from 10,000 to 13,000 fps. No deep channels were detected. A low bench was defined extending about 500 feet toward the river from the left abutment, where the depth to bedrock was only 20 to 28 feet. The colluvial accumulation at the base of the steep slope on the right abutment was found to approach 29 feet at the toe and gradually decrease up to the base of the outcrops at approximately elevation 6700. The velocity in the colluvium was 2100 to 2500 fps and in the bedrock 6900 to 8600 fps.

10.4.5 Construction Materials Investigation

As no adequate sites are available at Veatch Gulch Dam for a side channel spillway, the project was conceived as an RCC dam. Therefore the materials investigation was limited to backhoe pits in the alluvial gravel. The location of the test pits is shown on Plate 31. An adequate volume of sand and gravel is available adjacent to the dam in the valley floor to provide aggregate for RCC and conventional concrete. Samples were taken from the pits, and laboratory tests run. Test results are contained in Appendix E and summarized in Table 18.

TABLE 18

VEATCH GULCH: Construction Materials Test Results

DESCRIPTION

Test	Sand and Gravel (GP)
Specific Gravity	2.63
Sodium Sulfate Soundness	9.9% loss in 5 cycles
L.A. Abrasion	30.5% wear

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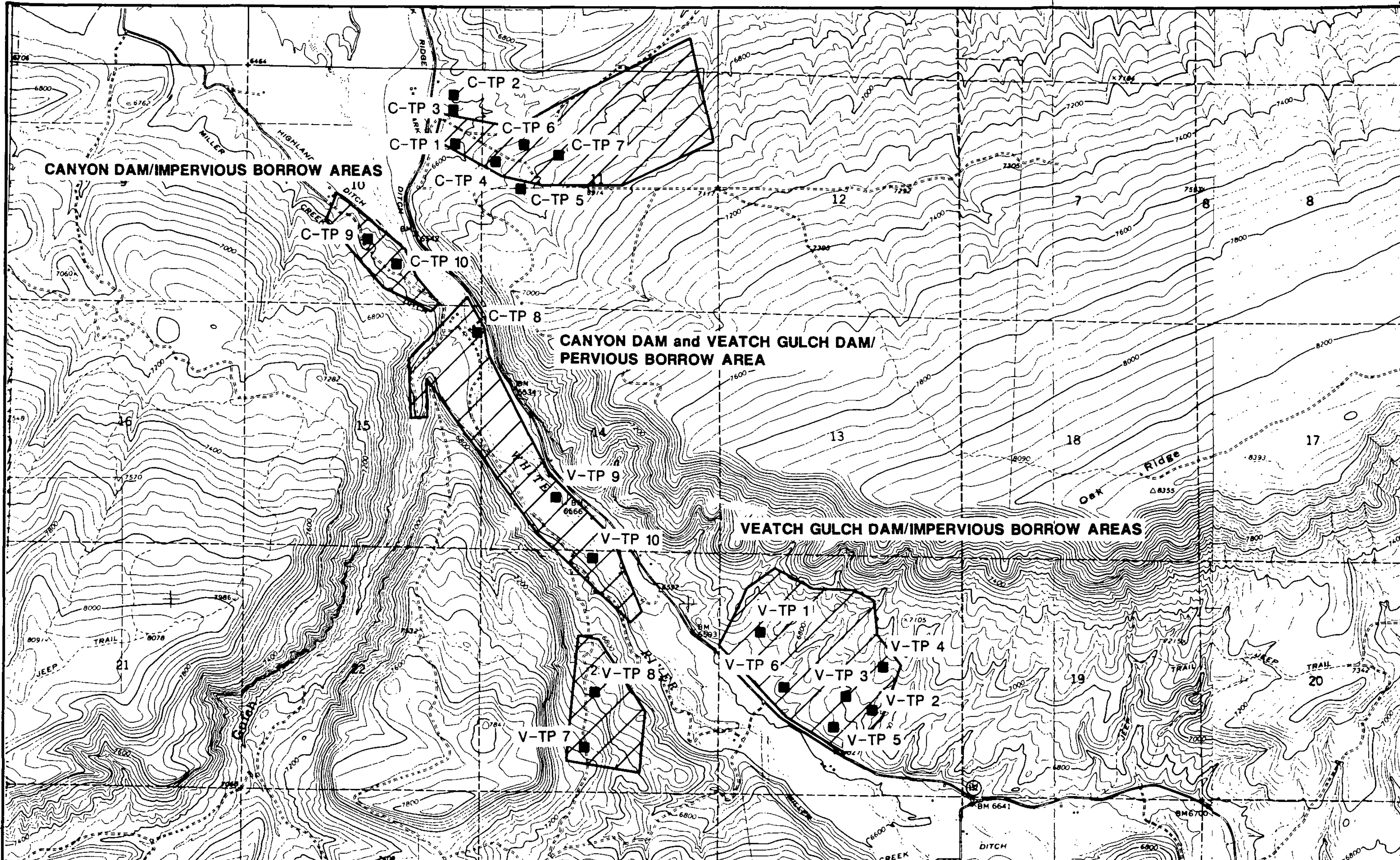

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CONST. MATERIALS/PLAN
VEATCH GULCH & CANYON DAM

IECO NO.
SHEET OF REV
Plate 31

001257

10.4.6 Engineering Geology

The subsurface exploration at the Veatch Gulch Dam Site showed the foundation to be of suitable quality for the proposed RCC dam section. The portion of the dam foundation below about elevation 6625, the Chinle Formation is massive, moderately hard, impervious siltstone for which minimal foundation treatment would be required. The overlying Entrada sandstone is probably less compact, but judging from the near-vertical outcrops, will also be of adequate quality. Due to the well-developed jointing parallel to the valley, however, the upper part of the foundation will probably have to be keyed back into the abutment and a grout curtain constructed up to the normal maximum water surface elevation. Stable slopes of 1H to 1V should be possible in the river bed alluvium during construction provided the infiltration of ground water is controlled.

Since no side channel spillways are anticipated with the RCC design, abutment slope stability would be limited to the high cliffs immediately adjacent to the dam crest. Some slabbing of large sandstone blocks could be expected, and could be controlled by stabilization with rock bolts and shotcrete.

Reservoir slope stability is not considered to be a problem with the Veatch Gulch Dam. With the exception of the subvertical cliffs on either side of the valley immediately upstream of the dam axis, reservoir slopes are gentle. The cliffs might be susceptible to toppling of individual blocks of sandstone, but no mechanism appears to exist to cause severe slumping of the abutments. The highest reservoir would flood the toe of the ancient landslides near Warner Point. The slopes at this point, however, are such that even rapid drawdown of the reservoir would not be expected to trigger slope failure. Any waves caused by a sudden slope failure would be attenuated by the bends in the reservoir over the nearly five mile distance between the slide area and the dam. Furthermore, the RCC gravity dam would be able to withstand safely overtopping by a sudden wave of water. The reservoir seepage questions associated with the Warner Point and Choke Cherry Reservoirs upstream do not exist at Veatch Gulch. The reservoir would be completely underlain by relatively impervious sandstone and siltstone units of the Entrada, Chinle, State Bridge, and Maroon Formations. The Eagle Valley-Minturn Formation would not be exposed to reservoir waters.

10.4.7 Design and Cost Estimate

Based on the foundation and materials information gathered from the geotechnical investigation, appraisal-level designs and cost estimates were prepared for dams at the Veatch Gulch site with storage capacities of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. A preliminary design evaluation showed that because of the high vertical cliffs adjacent to the axis, a side channel spillway with capacity to pass the PMF, as required by an embankment dam design, would not be economically feasible. Therefore, the designs and cost estimate for this study were made only for an RCC gravity dam alternative with the spillway built into the dam section.

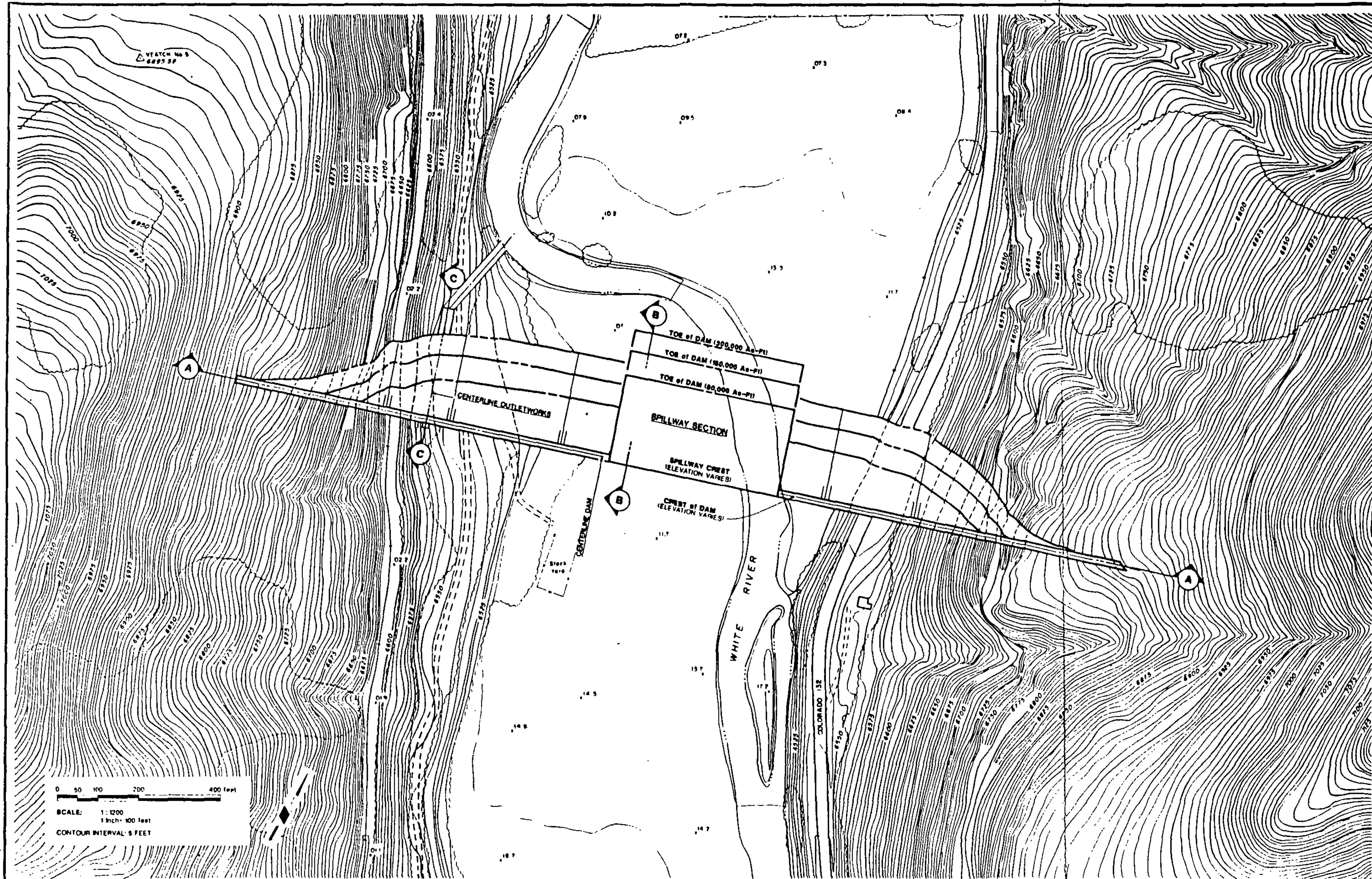
The designs for the RCC gravity dam are shown in plan on Plate 32 and in profile in Plate 33. The three sizes of dam would have normal maximum water surfaces, crest elevations, and height above stream bed as shown in Table 19.

TABLE 19

VEATCH GULCH: RCC Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)	NMWS Elevation (ft msl)
50,000	6698	188	6675
150,000	6781	271	6759
300,000	6854	344	6833

The foundation for the RCC gravity dam must be sound bedrock. Therefore foundation preparation would consist of removing all transported soil and debris as well as any weathered and highly fractured bedrock. It was assumed



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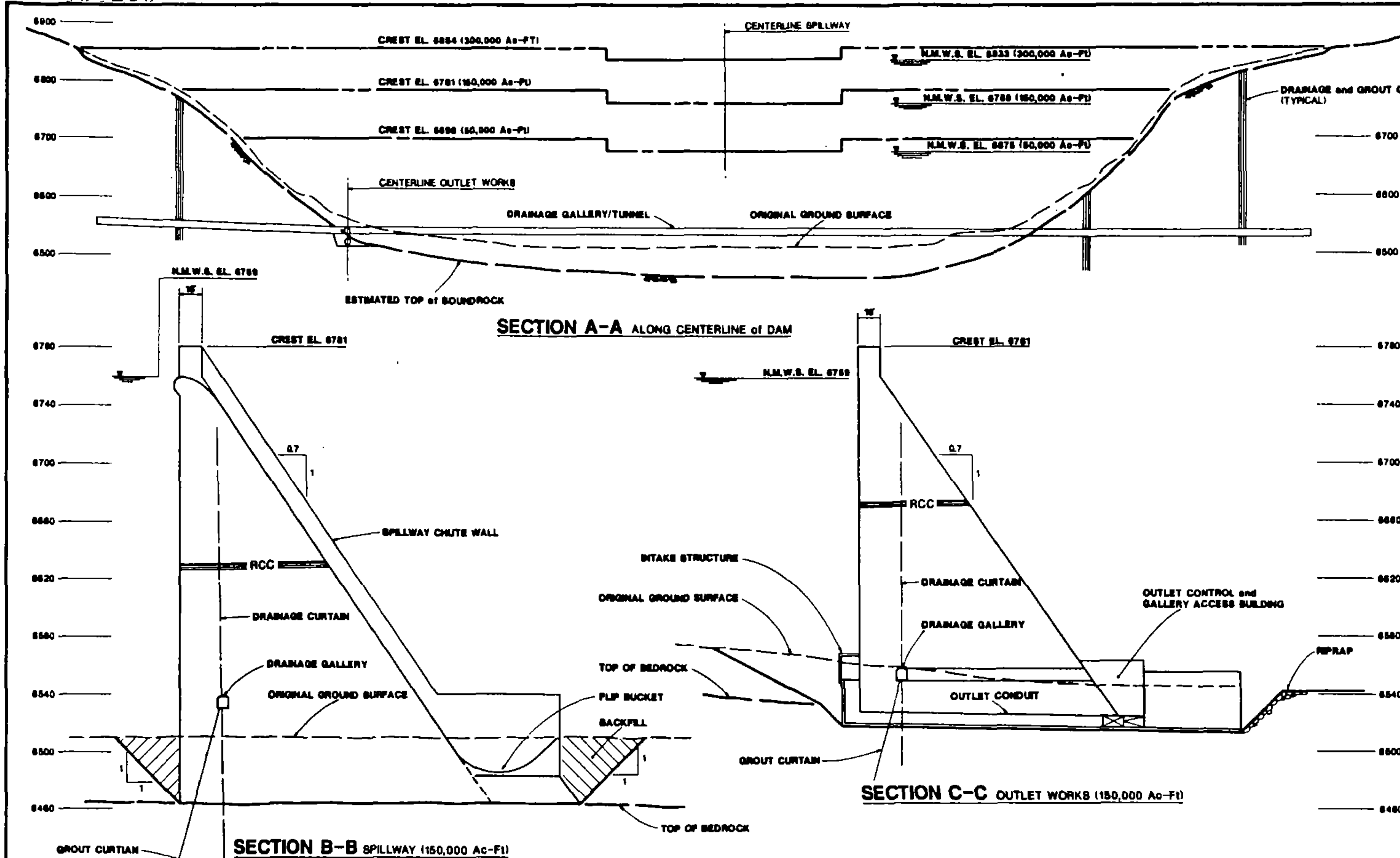
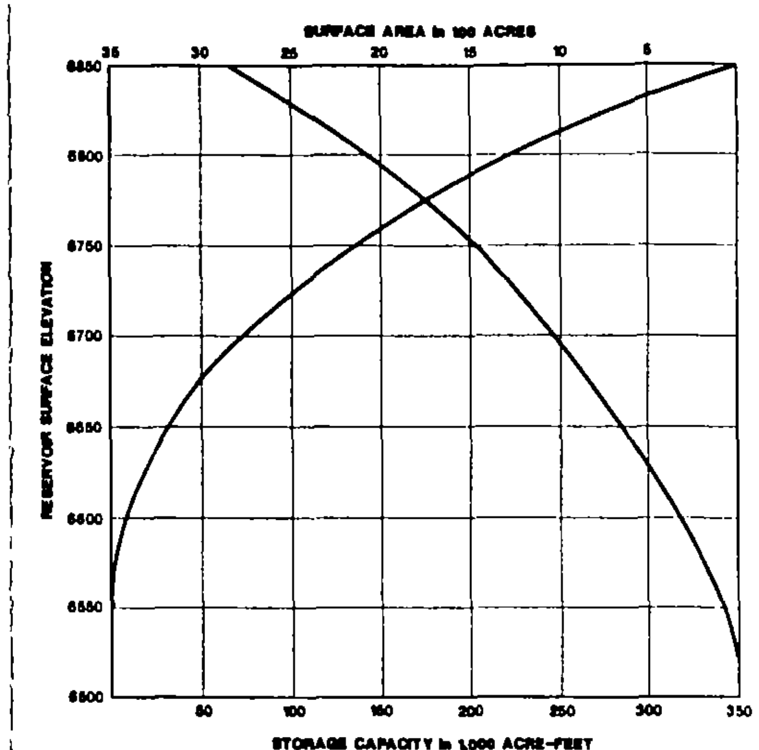
WHITE RIVER GEOTECHNICAL STUDY

RCC DAM/PLAN

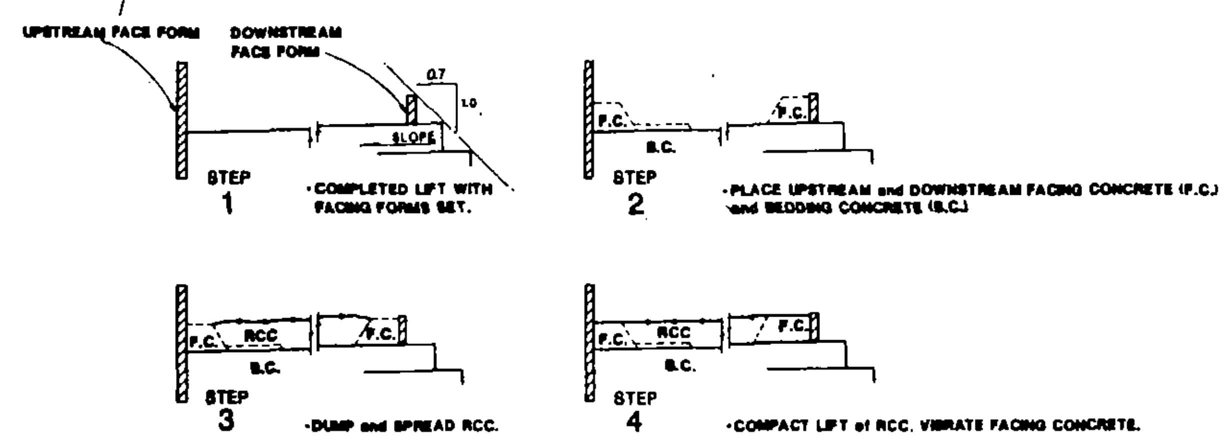
VEATCH GULCH DAM

SHEET	OF	REV
Plate 32		





RESERVOIR AREA-CAPACITY



PLACEMENT SEQUENCE FOR RCC and FACING CONCRETE

TABLE of ESTIMATED QUANTITIES

ITEM	RESERVOIR SIZE in ACRE FEET		
	50,000	150,000	300,000
COMMON EXCAVATION (Cu. Yd.)	369,780	610,160	887,670
ROCK EXCAVATION (Cu. Yd.)	1,600	7,670	23,790
RCC (Cu. Yd.)	806,850	1,618,090	2,611,050
GROUTING (Mi.)	186	732	1,580
STRUCTURAL CONCRETE (Cu. Yd.)	18,352	18,352	18,352
LAND ACQUISITION (Ac.)	1,037	1,821	2,893
HIGHWAY RELOCATION (Mi.)	8.3	7.7	8.3

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WHITE RIVER GEOTECHNICAL STUDY
RCC DAM/SECTIONS
VEATCH GULCH DAM

IECO NO
SHEET OF REV
Plate 33

for this estimate that excavation would extend 10 feet into the exposed bedrock on each abutment, and to the top of bedrock in the river.

A grout curtain would only be necessary along the length of the dam foundation where fractured sandstone of the Entrada Formation is exposed high on both abutments. The remainder of the foundation in the Chinle Formation is tight enough to not require a grout curtain.

The dam outlet works and spillway design would conform to the criteria outlined in Section 8.1.

The detailed cost estimates for the RCC dam at Veatch Gulch are included in Appendix H and are summarized in Table 20.

TABLE 20

VEATCH GULCH: RCC Dam Costs

	STORAGE CAPACITY (Acre-Feet)		
	50,000	150,000	300,000
Estimated Cost	$35,218,000	$57,888,000	$82,072,000

10.4.8 Findings and Conclusions

The alluvial gravel in the valley floor at Veatch Gulch is only about 40 feet thick as compared to 50 to 70 feet upstream at Choke Cherry and Warner Point. The reduced depth of excavation necessary for the gravity dam is favorable for the RCC dam design, but is offset by the width of the canyon.

The massive siltstone bedrock of the Chinle Formation which underlies the alluvium would be an excellent foundation for the RCC dam. No grouting or special foundation treatment would be necessary. The quality of the overlying sandstone of the Entrada Formation that forms the vertical cliffs on either abutment was not investigated at depth. Judging from the outcrops, it appears that the rock will provide an adequate foundation but will probably require grouting.

Adequate material for RCC aggregate is available in close proximity to the damsite. Alluvial sand and gravel from required excavation for the dam and spillway would be supplemented by additional gravel borrowed from the riverbed upstream or downstream of the dam.

Reservoir slope stability and reservoir permeability are not expected to be of concern for the Veatch Gulch Project.

The cost estimates show that the Veatch Gulch Dam ranks last among the four sites studied in the upper canyon stretch of the White River, for all reservoir capacities evaluated. It compares most favorably to the Warner Point and Canyon sites at a storage capacity of 150,000 and 300,000 acre-feet, but remains about 40 percent more expensive.

10.5 CANYON DAM

10.5.1 History

The Canyon Dam site was identified by Mr. Clifford Jex in an informal report to the White River Study Committee at the conclusion of IECO's White River Study in 1983. The site is located near the Highland Ditch Headgate at the extreme downstream end of the upper canyon section where the White River valley opens into Agency Park.

IECO considered the Canyon site (also referred to as the Oak Ridge site) in their informal report on sizing of the upper canyon sites. The Canyon Dam appeared to compare with Warner Point Dam from the cost versus storage capacity standpoint and was therefore considered as a candidate for additional investigation.

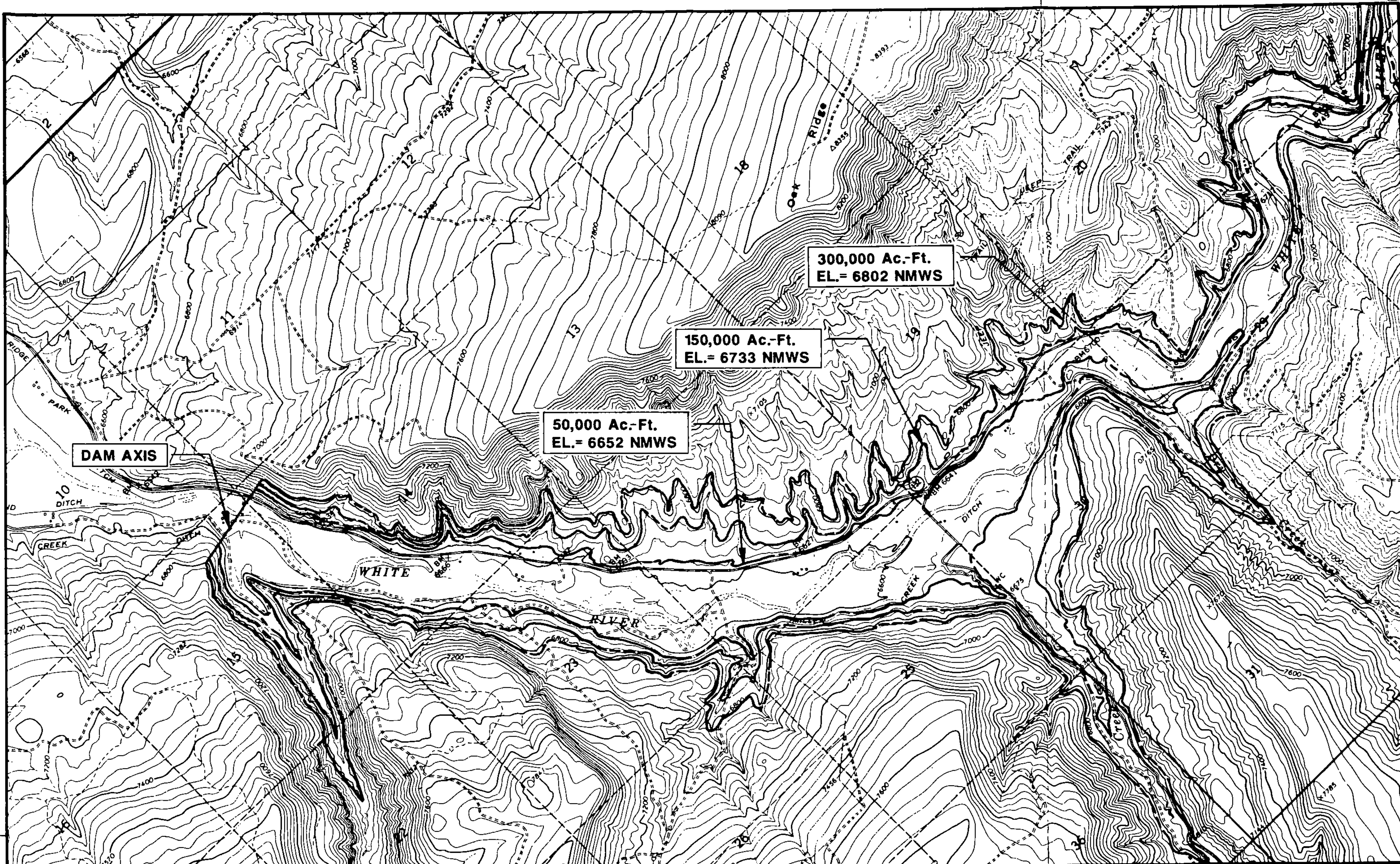
The USBR in their geological reconnaissance of the upper canyon sites in 1984 identified a landslide on the left abutment and pointed out the existence of two faults mapped by the USGS less than one mile southeast of the damsite.

10.5.2 Reservoir Characteristics

Plates 34A and 34B show the location of the Canyon Dam and the area that would be flooded by reservoirs of 50,000 acre-feet and 300,000 acre-feet. The characteristics of the Canyon Reservoir would be similar to those described for Veatch Gulch Reservoir in Section 10.4.2. All the inundated land is privately owned. About 50 percent is lowland hay fields along the mainstem White River and Miller Creek. The remainder is steeply sloping grazing land covered with sage, grass, and scrub oak.

10.5.3 Site Geology

Plate 6 is a geologic map of the Canyon area and Plates 35A and 35B show the surficial geology at the damsite. The Canyon Dam site is located at the narrows formed where the White River cuts through the resistant basal sandstone of the Morrison Formation. The damsite is underlain by the Entrada Sandstone composed of orange, poorly cemented, cross-bedded sandstone that crops out along the sides of the valley just upstream of the dam. The abutments are moderately steep slopes formed by the base of the Morrison Formation where sound outcrops were mapped on both sides of the river. The unit is a resistant, well-cemented, fine-grained, white sandstone, striking northeast and dipping about 20° to the northwest. The basal Morrison sandstone is overlain by a thicker, weak green clayey sandstone, and siltstone interbedded with thinner, hard purple, brittle layers of resistant sandstone. Outcrops of this unit are exposed along the river and in the road cut on the right side of the river downstream of the dam. Overlying the Morrison



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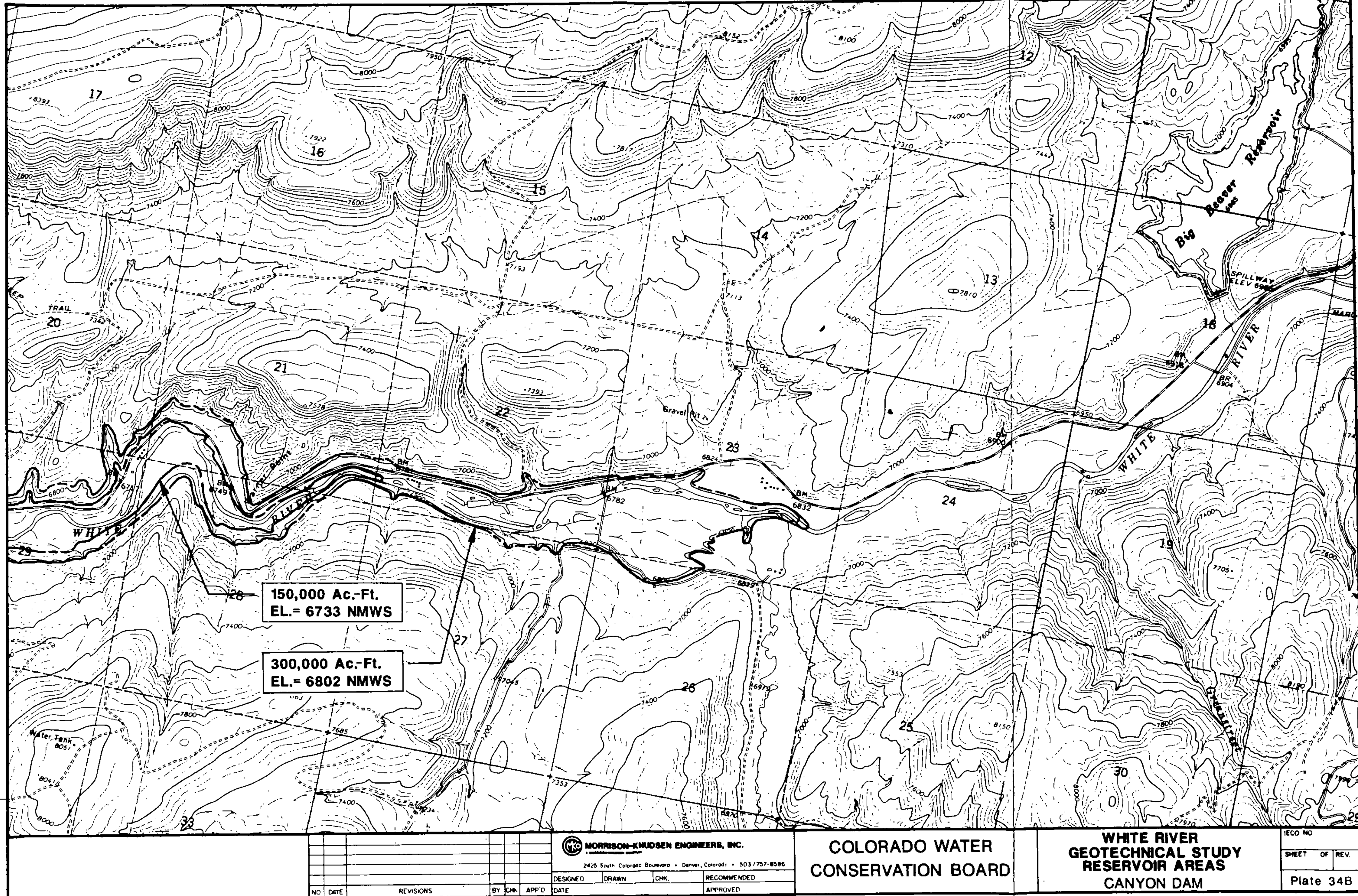
WHITE RIVER GEOTECHNICAL STUDY RESERVOIR AREAS
CANYON DAM

IECO NO | SHEET OF | REV.

Plate 34A

001265


150,000 Ac.-Ft.
EL.= 6733 NMWS
300,000 Ac.-Ft.
EL.= 6802 NMWS
WHITE RIVER
Big Beaver Reservoir
SPILLWAY ELEV 6960
Gravel Pit
Water Tank
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WHITE RIVER GEOTECHNICAL STUDY RESERVOIR AREAS
CANYON DAM
IECO NO
SHEET OF REV.
Plate 34B

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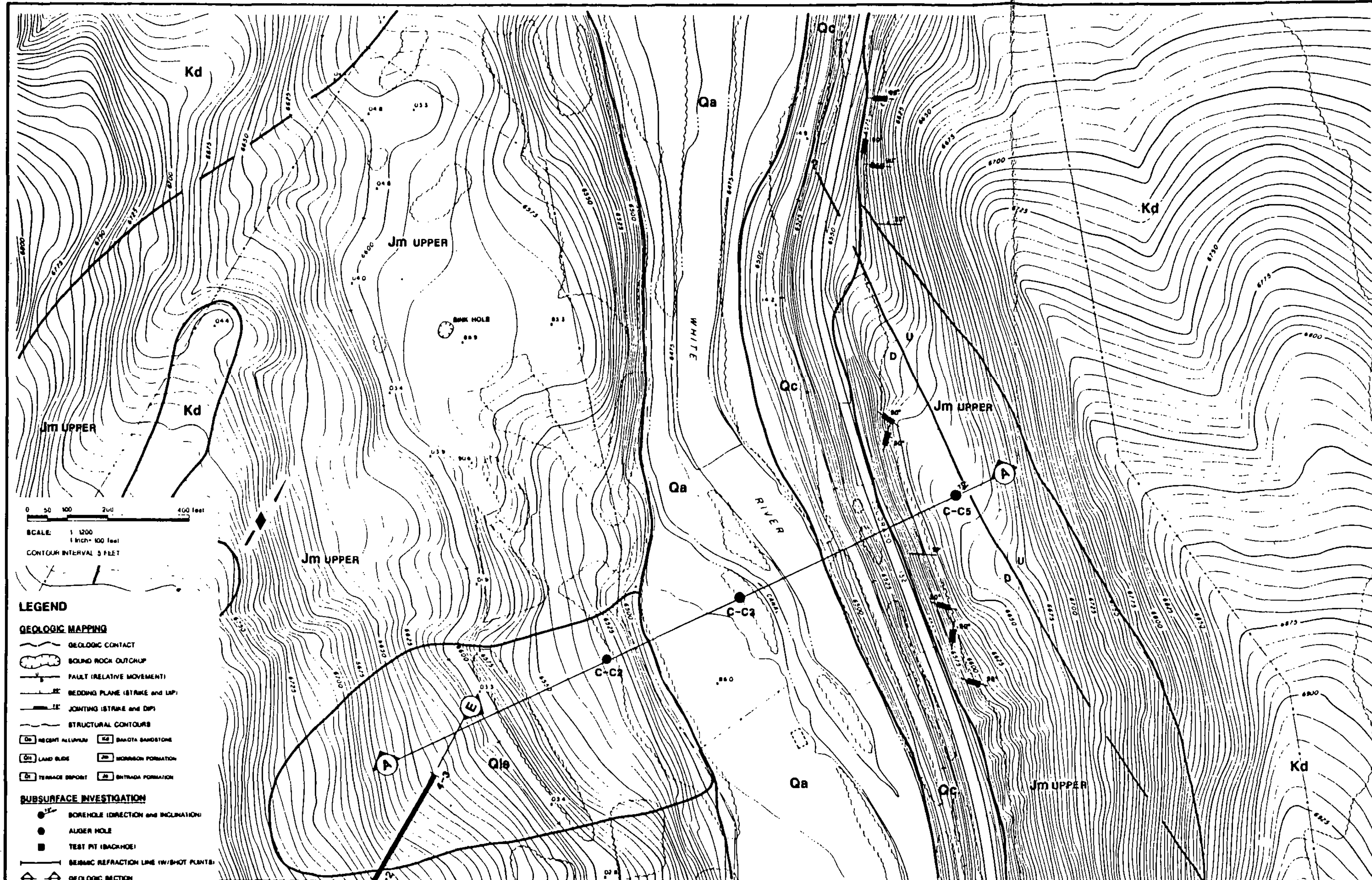
WHITE RIVER GEOTECHNICAL STUDY

GEOLOGIC MAP/LOCATION PLAN

CANYON DAM

SHEET	OF	REV.
Plate 35A		

331267



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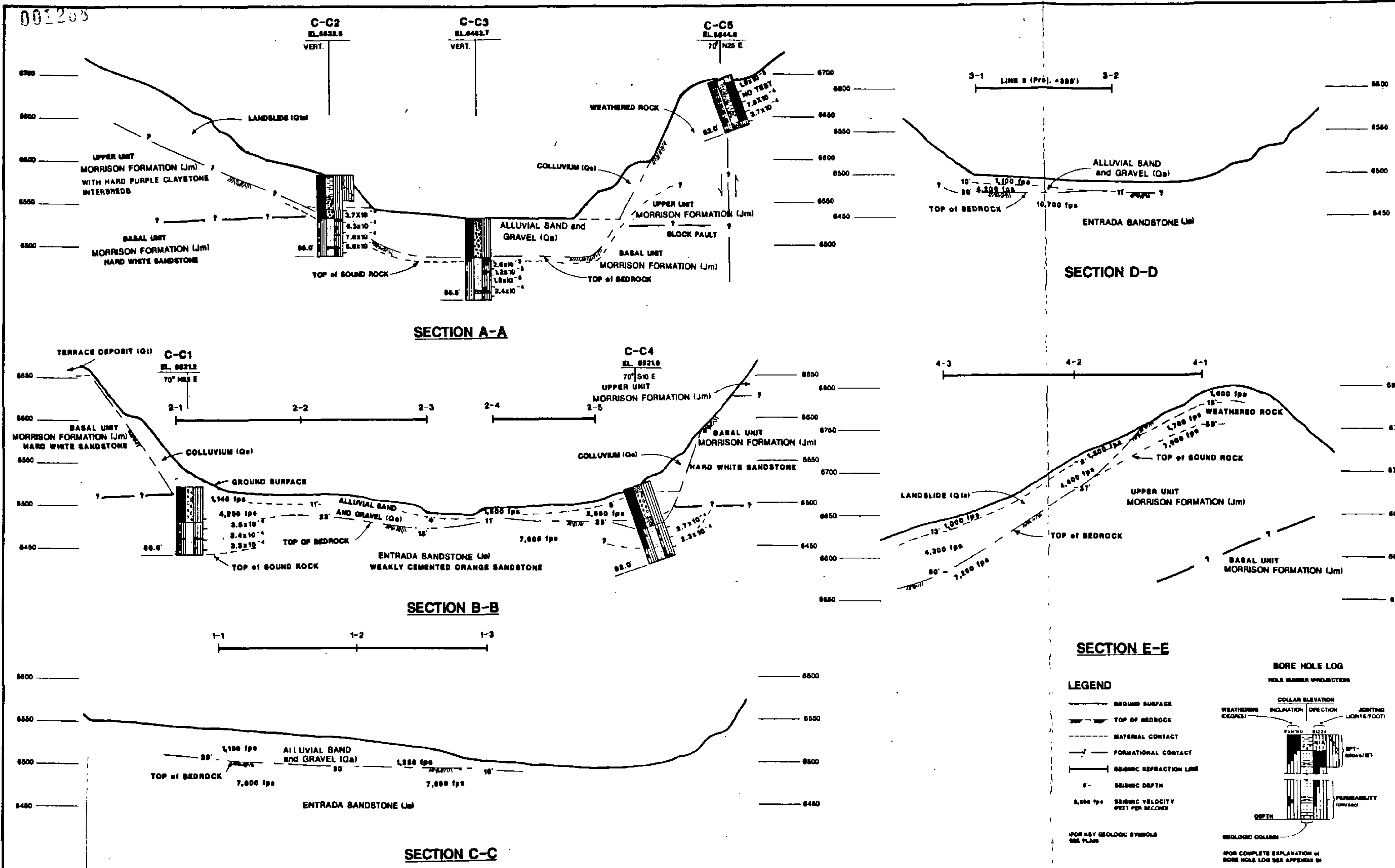
WHITE RIVER GEOTECHNICAL STUDY
GEOLOGIC MAP/LOCATION PLAN
CANYON DAM

SHEET	OF	REV.
Plate 35B		



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WHITE RIVER GEOTECHNICAL STUDY

GEOLOGIC SECTIONS
CANYON DAM

ECO NO
SHEET OF REV.
Plate 36

Formation and capping the high hogback ridges above the dam abutments is the resistant light-colored Dakota sandstone.

A variety of Quaternary deposits are present at the site. Alluvial sands and gravels blanket the valley floor. The gravel is made up of gray-to-pink granite, gneiss, and quartzite, black-and-purple basalt, and white, green and purple sandstone of the Morrison Formation. A terrace gravel deposit occurs about 100 feet above the valley floor on the left abutment. A landslide in the weak, clayey sandstone of the upper Morrison Formation occurs on the left abutment, partially covering the river alluvium. The steeper slopes in the more competent rock upstream of the site is masked by a veneer of slope wash with an accumulation of colluvium at the base of the slopes.

Two normal faults mapped previously by the USGS just south of the dam site were confirmed in the field. The faults strike about N45°W and are continuous for about one-half mile. The south side of the southernmost fault is upthrown Chinle Formation and Entrada Sandstone against Morrison Formation to the north. The block between the two faults is downdropped in relation to the Morrison Formation in the north forming a graben. The faults are evidenced by offsets in the hogback ridges on the south side of the river formed by the Dakota Sandstone. Another northwesterly-trending normal fault was mapped in the right abutment of the Canyon Dam site. The southwestern side is downdropped about 50 feet, as evidenced by the offset in the Dakota sandstone. The fault is characterized by the long, thin saddle that parallels the river midway up the abutment. No folding was observed at or near the Canyon site. Regional jointing patterns are visible in outcrops in the basal Morrison sandstone along the Miller ditch on the left abutment. The most frequent set is subvertical, striking N50°W. This set occurs roughly parallel to the valley at this point and most of the joints appear to be slightly open, probably having suffered some stress relief. Less frequent are the joints striking northeast. Bedding plane joints are discontinuous due to cross-bedding of the sandstone.

The reservoir geology consists of a sequence of progressively older rocks dipping downstream toward the damsite. The slopes of the reservoir would be generally steep, with the bedrock masked by slope wash and accumulations of colluvium at the base of the slopes near the waterline. The valley floor is covered with alluvial sand and gravel for the entire length of the reservoir.

10.5.4 Foundation Investigation

Following the preliminary site screening described in Section 9, no subsurface investigation was planned at the Canyon site. However, following the decision by MKE and the CWCB to obtain less detailed information on a wider variety of sites, Canyon Dam was added to the program. Five boreholes were drilled along two axes and seismic refraction surveys were carried out along the upstream axis, the center of the valley and along a potential spillway alignment on the left abutment. The location of the boreholes and seismic lines are shown in plan on Plates 35A and 35B, and the results are represented as sections on Plate 36.

Originally the centerline was located at the narrowest point in the canyon section near the Highland Ditch Headgate and through the landslide mapped on the left abutment. Five holes were planned along the centerline and numbered sequentially from left to right. The first borehole was C-C3 located in the center of the valley floor near the river. The hole was drilled vertically through 47 feet of alluvial gravel and boulders composed of pink-to-gray granite and gneiss, black-to-purple basalt and light gray, green, and red sandstone derived from the Maroon and Morrison Formations. Bedrock was composed of light gray sandstone of the basal unit of the Morrison Formation. The top four feet was slightly to moderately weathered and moderately to highly jointed. Below that point to the bottom of the hole at 95.5 feet, the rock mass was sound, slightly weathered, and nearly devoid of joints except in concentrated zones a few feet thick. Most of the joints were parallel to the purple siltstone interbeds, but a few regional joints striking N55^{o}W and dipping 70^{o}SW to 90^{o} were logged. Packer tests showed the rock to be relatively permeable in the upper 24 feet with values in the 1.2 to 2.6 x 10^{-3} cm/sec range. The lower part of the hole was tighter with permeabilities ranging from 2.4 x 10^{-4} cm/sec to 1.8 x 10^{-5} cm/sec.

Hole C-C2 was drilled vertically on the left abutment in the area of the apparent landslide. The boring penetrated 29 feet of mottled gray-brown clay of moderate plasticity. Standard Penetration Test blow counts averaged about 10 over the upper 13 feet, increased to about 30 blows between 13 and 20 feet, and ranged from 40 to 50 blows to the bottom of the clay zone. The increase in blow counts was due to a gradual increase in the number of rock fragments of sandstone, siltstone and shale incorporated into the clay matrix. Below 29 feet, the ground was inpenetrable due to the hardness of the angular conglomerate boulders encountered. At 41 feet, clean alluvial river gravel was contacted. This proved that the clayey fragment-strewn overburden was a landslide that had flowed down the left abutment partially covering the valley floor. The alluvial gravel, composed of sand, granite, and basalt cobbles, extends to a depth of about 55 feet where it overlies sound light green-to-gray sandstone of the basal Morrison Formation. The rock is locally sightly weathered or poorly cemented and lightly jointed. The joints logged were almost entirely open bedding plane joints, although a few high angle joints were encountered. The core was not oriented because of a lack of clear bedding. Packer tests showed the rock to be of moderately high permeability ranging from 3.7×10^{-4} to 7.0×10^{-4} cm/sec.

Borehole C-C5 was drilled in the saddle above the highway cut on the right abutment. The hole was intended to investigate the suitability of the green clayey siltstone and sandstone of the upper Morrison Formation as a foundation material, as well as to explore the condition of the northwest trending fault identified by the surficial geologic mapping. The hole was inclined toward the abutment 70^{0} into what appeared to be slightly weathered Dakota Sandstone. However, only a few fragments were recovered to a depth of nine feet. Below that depth, to the bottom of the hole at 62 feet, the core recovered was highly weathered and fractured, buff to dark-gray sandstone and siltstone presumably of the upper Morrison. The core did not resemble the weathered rock exposed in the road cut only a few feet distant. The only similarity was the thin interbeds of hard, brittle purple sandstone. The core was badly crumbled and destroyed except for a stretch of light-colored conglomerate between 55 and 60 feet in depth. Water pressure tests showed the zone to be of moderate to high permeability ranging from 1.9×10^{-3} cm/sec to 3.7×10^{-4} cm/sec, with some evidence of washing of material from within the open joints at high pressure.

Due to the unfavorable results of borings C-C2 and C-C5, it was decided to move boreholes C-Cl and C-C4 upstream to an alternative centerline. Although the new axis is not as narrow as the first, outcrops of sound sandstone of the basal unit of the Morrison Formation occur on both abutments. Each hole was inclined 70° into the base of the steep colluvial slopes fed from the sound white sandstone outcrops above. Hole C-Cl on the left abutment was located at the base of a cut in the colluvium about 10 feet high and penetrated 39 feet of that material to the top of bedrock. The colluvium was composed of well-rounded cobbles derived from a deposit of terrace gravel located up the slope and greenish-white angular sandstone blocks derived from the base of the Morrison. The coarse material was enveloped in a brown silty sand matrix. Hole C-C4 at the base of the opposite abutment was located in a 10-foot cut below the county road. It penetrated about 10 feet of colluvium identical to the left abutment except for the terrace gravels. The colluvium overlies about 27 feet of recent alluvial sands and gravels. The river gravels, in turn, cover about 11 more feet of ancient colluvium of the same nature as the more recent material above. Top of bedrock in this hole was about 48 feet. Bedrock in both holes was orange to pink, weakly-cemented, medium-grained Entrada Sandstone which underlies the Morrison Formation. The core appeared to be only slightly to moderately weathered, however it was difficult to determine because the core was often destroyed by drilling, probably due to the weak, uncemented nature of the rock. No clay seams were detected. The rock appeared to be very uniform and massive, but weak. Most of the measureable joints were regional, striking northwest and dipping 70°SW to 90°. No bedding plane joints were logged. Permeabilies in the rock mass were moderate, averaging about 3.4×10^{-4} cm/sec. All tests showed signs of washing or nonelastic deformation of the joints.

The seismic refraction survey at Canyon Dam focused on the valley floor along the upstream centerline between boreholes C-Cl and C-C4. Seismic velocities of 3600 to 4200 fps were recorded in the alluvial gravel and 7150 to 8300 fps in the underlying Entrada Sandstone. The depth to bedrock determined by seismic refraction correlated well with the boreholes near the abutments, ranging from 40 to 50 feet. The seismic line showed the thickness of gravel to be much less in the center of the valley, with as little as 11 feet overlying bedrock near the bridge over the river. The average depth across the valley is only about 25 feet, which is considerably less than at any of the other sites surveyed. Seismic lines upstream and downstream of the centerline showed depths ranging from 11 to 36 feet and detected no deep canyons incised into the valley floor.

One line of seismic refraction was run along a possible spillway alignment on the left abutment. The line crossed the area mapped as a landslide. A surface soil profile about 15 feet thick, with seismic velocities less than 1200 fps was detected over the entire line. The slide zone below that was characterized by velocities ranging from 1700 fps to 4400 fps, and extended from a depth of 38 feet at the top of the left abutment to 60 feet deep at the toe. Bedrock was typified by velocities of about 7100 fps.

10.5.5 Construction Materials Investigation

Canyon Dam was originally conceived as an embankment dam but with the relocation of the centerline upstream, an RCC dam might be feasible provided the foundation rock is of adequate strength.

Required excavation for a side channel spillway would probably not produce appreciable volumes of rock fill material, but might produce material that could be used in the impervious core and transition zones. No materials investigation or testing was done on material from required excavation. Riprap could be obtained by quarrying the Morrison or Dakota sandstones.

Abundant gravel for RCC and conventional concrete aggregate is available from required excavation and borrow in the river valley. This material could also be processed to provide filter and drain material. In addition, approximately five miles downstream, extensive terrace gravel deposits are being commercially explored at present along the White River in Agency Park.

The materials investigation focused on locating impervious borrow material. Surficial geologic mapping identified two potential sources downstream of the dam, on either side of the river, and one upstream of the Veatch Gulch site on the northeast side. Test pits were excavated by backhoe and samples were obtained for laboratory testing. The locations of the test pits are shown on Plate 31.

The upstream area was located in residual soil derived from weathering of sandstone, siltstone, and claystone of the Chinle Formation, and transported soils derived from erosion of this unit. Test pits were located along a line trending approximately east-west through the potential borrow area. The test pits showed the deposits to be very heterogeneous in terms of physical characteristics and depth. Samples ranged from red sandy clay to yellow sand. Due to the lack of uniformity, the area was determined to be infeasible as a source of impervious construction material.

A cursory investigation was also made of other potential sites on the south side of the river. Residual soil from the Chinle formation was determined to be too thin. Transported soil in the floodplain along Miller Creek would probably be of acceptable quality, but is located too far from the damsite to be economical.

Test pits were located on the high pediment on the left side of the river less than one mile downstream of the dam. The material is a dark brown clay of low plasticity, probably derived from in-place weathering of the upper Morrison Formation. The depth of the deposit is unknown but is at least 12 feet, the maximum depth of a backhoe pit. Enough material is available for construction of the impervious core of the dam.

Another potential borrow source for impervious core material is located in the northeasterly trending valley on the right side of the river one mile downstream of the dam. It appears to be composed of transported material derived from weathering of the Mowry Shale. Test pits were excavated over the area to depths ranging from six to 12 feet. The material is a dark gray clay of low plasticity. A borrow pit had been opened previously in this deposit, probably to supply material for construction of the county highway.

Samples were taken from all of the test pits for laboratory testing. The test results are contained in Appendix E and summarized in the Table 21.

TABLE 21

CANYON: Construction Materials Test Results

DESCRIPTION

Test	Clay (CL) and Silty Clay (CL-ML)	Gravel (GP)
Plasticity Index	16	NP
Natural Moisture	18%	N/A
Optimum Moisture	18%	N/A
Optimum Density	109 pcf	195 pcf
Shear Strength	C=11 psi, Ø=22^{o}	C=0, Ø=40^{o}
Permeability	2.1 x 10^{-7} cm/sec	1.0 x 10^{-2} cm/sec

10.5.6 Engineering Geology

The initial borehole at the Canyon Dam Site showed the bedrock foundation under the alluvial gravel to be sound and of excellent quality for either an RCC or an embankment dam. However, the holes drilled in either side of the river showed the foundation of both abutments to be totally inadequate. On the left abutment, over 50 feet of landslide debris overlies river alluvium; on the right side, the bedrock is highly fractured probably due to the proximity of the fault running through the abutment.

Based on the results of these two boreholes, the dam centerline was relocated approximately 1000 feet upstream, between sound rock promontories on either side of the river. The two holes drilled near the abutments and the seismic refraction line along the axis showed the foundation across the river bottom to be shallow. The weakly cemented sandstone of the Entrada Formation would provide an adequate foundation for the cutoff trench of an embankment dam. However, the possibility of locating an RCC gravity dam at the site would have to be questioned due to the apparent low compressive strength of the core recovered. If further investigation of an RCC alternative were to be carried out at the Canyon site, strength testing of the sandstone would be essential to determine the feasibility of the design. The hard white sandstone at the base of the Morrison Formation that crops out on both abutments would provide an adequate foundation for either an embankment or RCC dam. Attention, however, would have to be given to any weak bedding planes that might occur, as they would dip about 15 degrees directly downstream. The contact between the competent basal Morrison Formation and the upper weaker unit occurs at approximately elevation 6650. The crest of an RCC dam might be limited to that elevation.

Regardless of which type of dam is selected, a grout curtain would have to be provided to cut off seepage through the fractured and permeable bedrock.

If an embankment dam were selected, the stability of the cut slopes of the spillway channel on the left abutment would be of concern. The excavation for the chute would cut across the landslide, and would penetrate into weathered bedrock of unknown condition. Because of the clayey nature of the slide material, relatively flat slopes would probably be required to ensure stability. Rock cuts for the channel could present a stability problem because they would be roughly parallel to the primary set of subvertical joints striking to the northwest. This set in conjunction with any local bedding plane joints and a second set of vertical joints could provoke movement of unstable wedges into the channel.

Reservoir stability and seepage characteristics at the Canyon site are identical to those at Veatch Gulch. With the exception of instability of an occasional sandstone block from the Entrada Sandstone cliffs, the reservoir has flat stable slopes. The reservoir would be mainly underlain by impervious units of the Chinle and State Bridge Formation composed of siltstone and claystone. Seepage through the more permeable sandstones of the Morrison and Entrada Formations directly under the dam could be controlled by a grout curtain.

10.5.7 Design and Cost Estimate

Based on the foundation and materials information gathered from the geotechnical investigation, appraisal-level designs and cost estimates were prepared for dams at the Canyon site with storage capacities of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. Preliminary comparisons between embankment and Roller Compacted Concrete gravity sections showed the two alternatives to be competitive, therefore designs and cost estimates were prepared for each capacity with both concepts.

10.5.7.1 Embankment Dam

The embankment dam designs are shown in plan on Plate 37 and in section on Plate 38. The three sizes of embankment would have crest elevations, heights above present river bed, and normal maximum reservoir surfaces as shown in Table 22.

Foundation preparation would consist of complete removal of all colluvial debris, terrace gravels, and slope wash. It would be necessary to excavate a cutoff trench through the pervious alluvium of the valley floor and 10 feet into weathered rock, where exposed on the left and right abutments. The upstream and downstream shells would be founded on in-place alluvial gravel and weathered rock. A grout curtain was considered to be necessary along the entire length of the dam due to the jointed nature of the bedrock.

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WHITE RIVER GEOTECHNICAL STUDY

EMBANKMENT DAM/PLAN
CANYON DAM

FCO NO
SHEET OF REV
Plate 37

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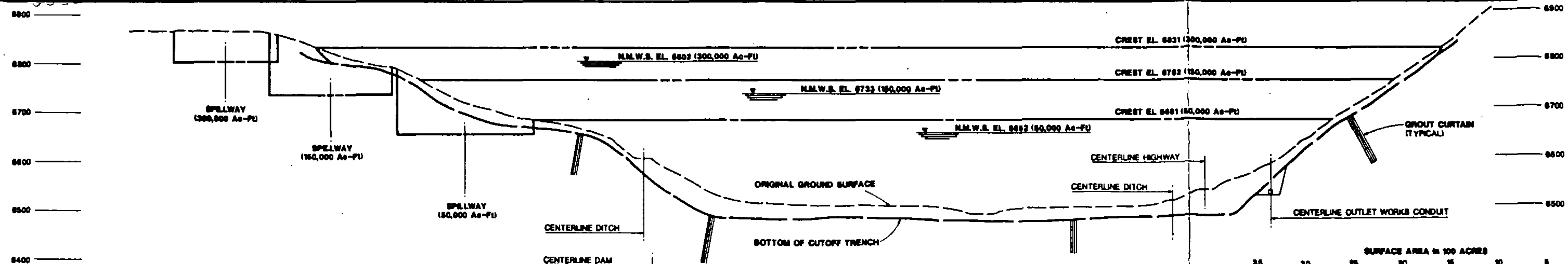
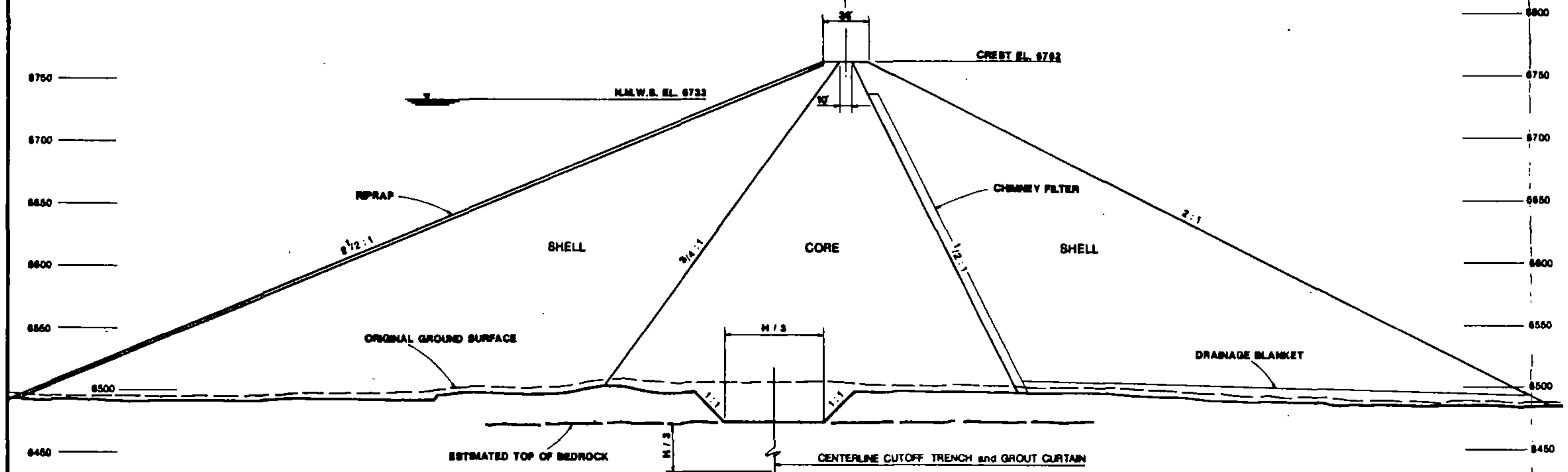
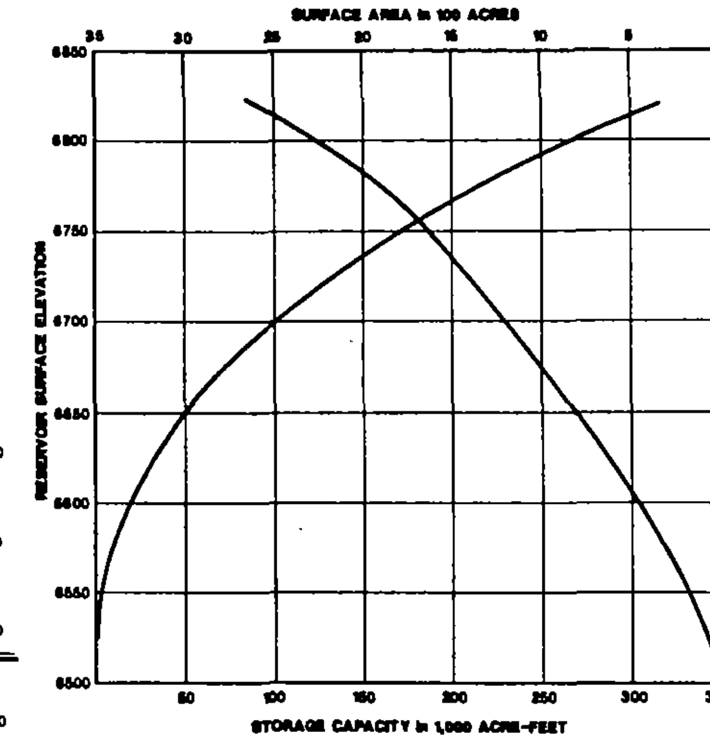


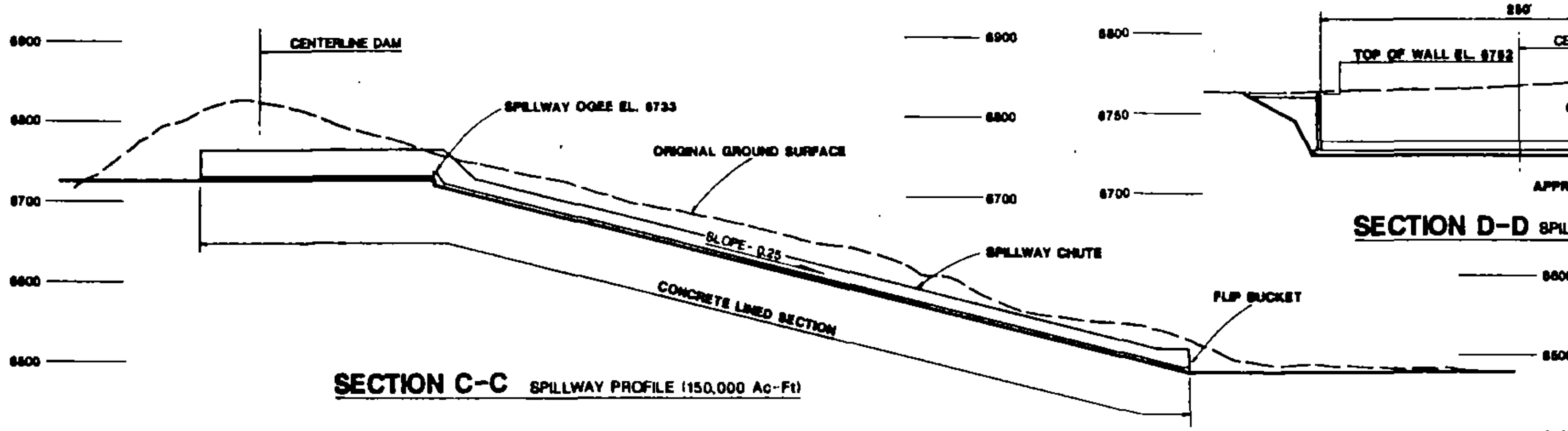
SECTION A-A PROFILE ALONG CENTERLINE of DAM



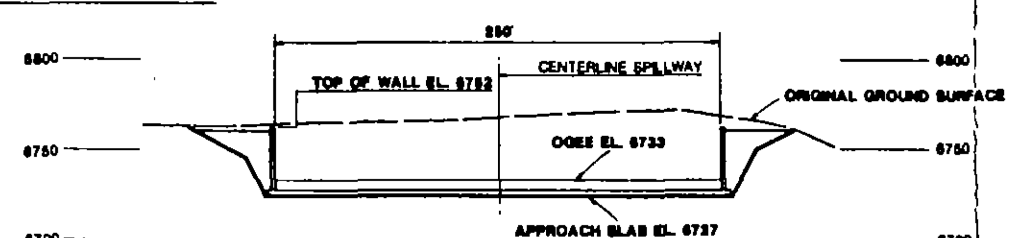
SECTION B-B MAXIMUM EMBANKMENT SECTION (150,000 Ac-Ft)



RESERVOIR AREA/CAPACITY



SECTION C-C SPILLWAY PROFILE (150,000 Ac-Ft)



SECTION D-D SPILLWAY CREST (150,000 Ac-Ft)

TABLE of ESTIMATED QUANTITIES

ITEM	RESERVOIR SIZE in ACRE FEET		
	50,000	150,000	300,000
COMMON EXCAVATION (Cu. Yd.)	637,368	806,064	834,375
ROCK EXCAVATION (Cu. Yd.)	364,171	135,481	126,741
CORE (Cu. Yd.)	1,096,808	2,328,913	3,989,264
SHELL (Cu. Yd.)	2,296,064	6,438,816	9,657,335
RIPRAP (Cu. Yd.)	66,847	111,701	167,352
GROUTING (Sk.)	4,806	7,348	10,223
STRUCTURAL CONCRETE (Cu. Yd.)	23,236	26,305	25,760
LAND ACQUISITION (Ac.)	1,053	1,756	2,830
HIGHWAY RELOCATION (Mi.)	6.3	7.7	8.3

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WHITE RIVER GEOTECHNICAL STUDY
EMBANKMENT DAM/SECTIONS
CANYON DAM

IECO NO
SHEET OF REV.
Plate 38

TABLE 22

CANYON: Embankment Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)	MMWS Elevation (ft msl)
50,000	6681	181	6652
150,000	6762	262	6733
300,000	6831	331	6802

The embankment would be a zoned earthfill conforming to the design description in Section 8.1. The impervious core material would be derived entirely from the downstream borrow areas described in Section 10.4.5. The shell zones would be constructed of rock and gravel produced from required excavation of the dam foundation and spillway, complemented by borrowed gravel as necessary. The upstream slope would be protected with riprap also derived from rock excavated for the spillway chute.

The outlet works and spillways for the embankment dam alternative are described in Section 8.1. Typical design details are shown in Plate 9. The spillway channels would be cut into the left abutment of the dam for all height alternatives. Discharge energy would be dissipated by a flip bucket at the downstream end of the chute.

The detailed cost estimates for the embankment alternative at Canyon Dam are included in Appendix H and are summarized in Table 23.

10.5.7.2 RCC Dam

The designs for the RCC gravity dam alternative are shown in plan on Plate 39 and in sections in Plate 40. The three sizes of dam would have normal maximum water surfaces equal to the embankment dam but the crest elevations and heights above stream bed would be as shown in Table 24.

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WHITE RIVER GEOTECHNICAL STUDY

RCC DAM/PLAN

CANYON DAM

SHEET OF | REV.

Plate 39

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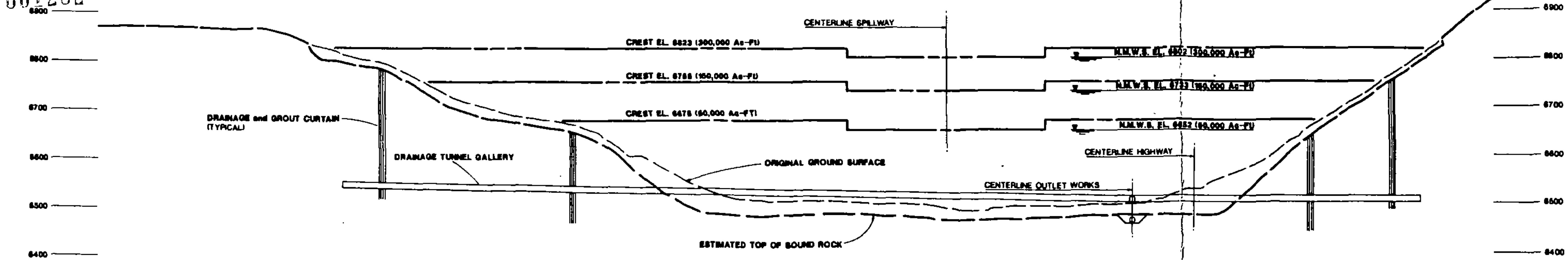
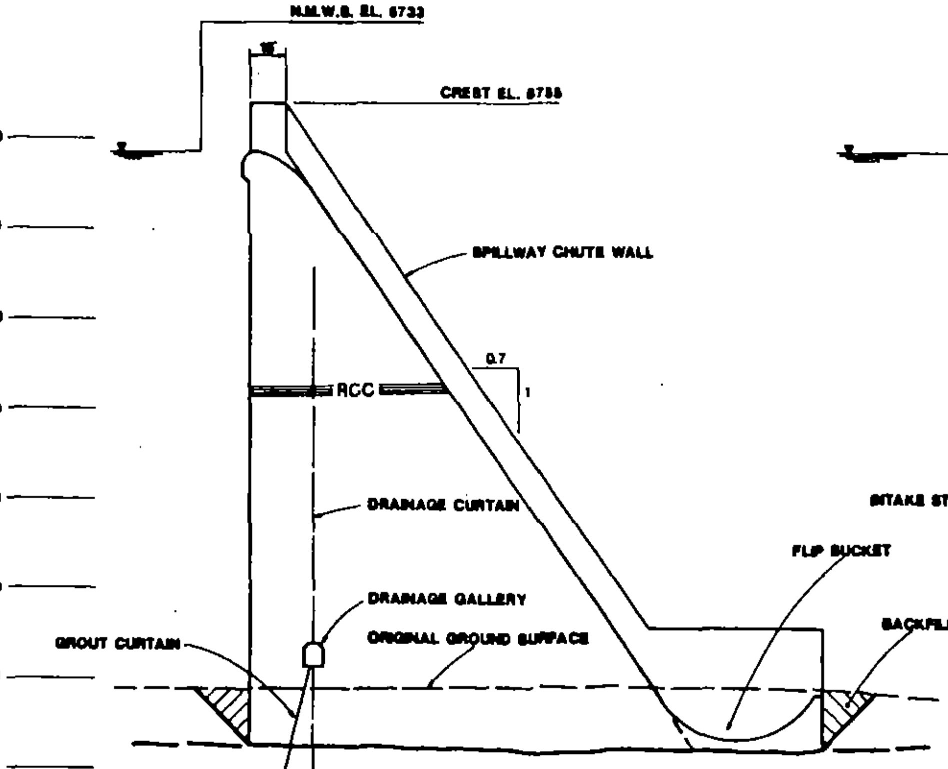
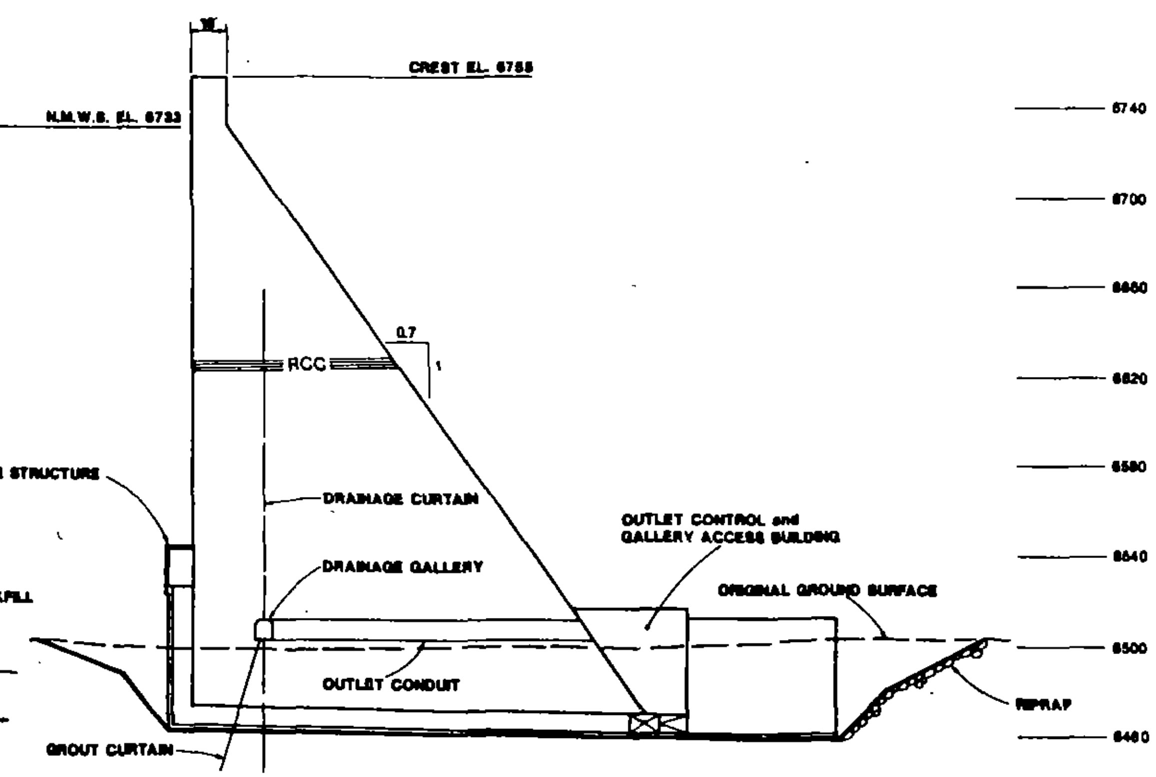


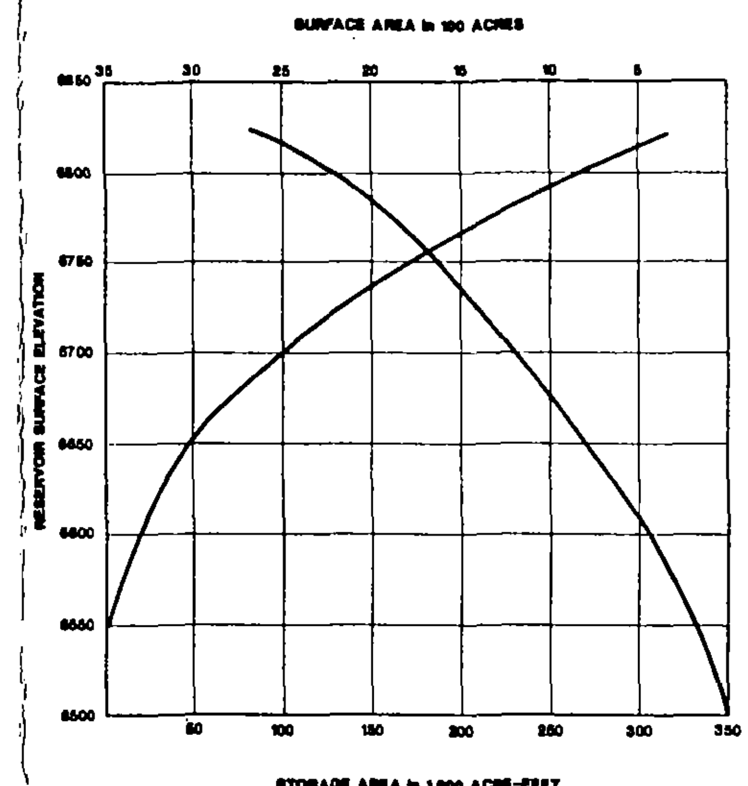
SECTION A-A PROFILE ALONG CENTERLINE of DAM



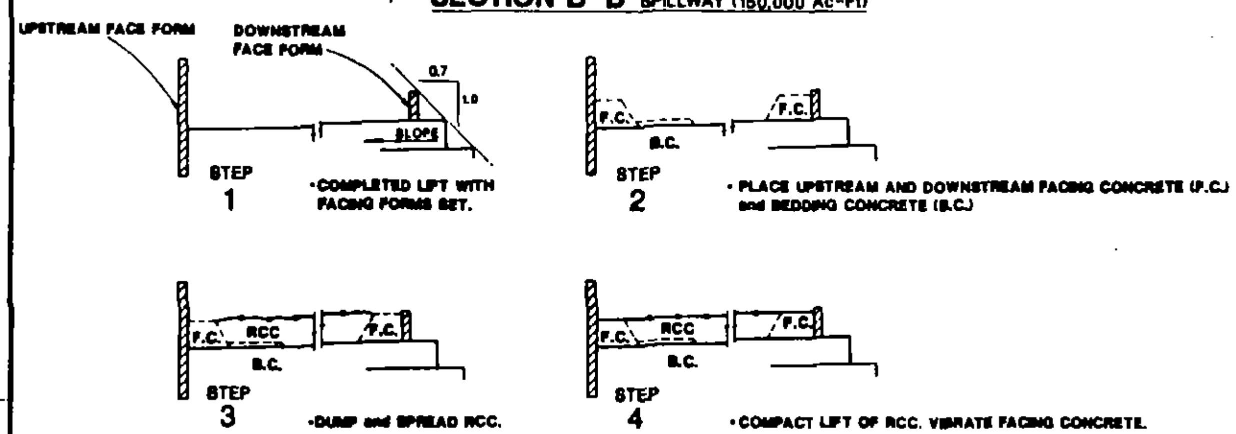
SECTION B-B SPILLWAY (150,000 Ac-Ft)



SECTION C-C OUTLET WORKS (150,000 Ac-Ft)



RESERVOIR AREA/CAPACITY



PLACEMENT SEQUENCE FOR RCC and FACING CONCRETE

TABLE of ESTIMATED QUANTITIES

ITEM	RESERVOIR SIZE in ACRE FEET		
	60,000	150,000	300,000
COMMON EXCAVATION (Cu. Yd.)	278,770	388,900	485,720
ROCK EXCAVATION (Cu. Yd.)	8,640	36,085	70,870
RCC (Cu. Yd.)	637,630	1,395,920	2,367,710
GROUTING (St.)	4,208	7,348	10,223
STRUCTURAL CONCRETE (Cu. Yd.)	18,352	18,352	18,352
LAND ACQUISITION (Ac.)	1,008	1,723	2,900
HIGHWAY RELOCATION (Mi.)	6.3	7.7	8.3

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WHITE RIVER GEOTECHNICAL STUDY
RCC DAM/SECTIONS
CANYON DAM

IECO NO.
SHEET OF REV.
Plate 40

TABLE 23

CANYON: Embankment Dam Costs

	STORAGE CAPACITY (Acre-Feet)		
	50,000	150,000	300,000
Estimated Cost	$27,651,000	$40,223,000	$58,699,000

TABLE 24

CANYON: RCC Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)
50,000	6675	175
150,000	6755	255
300,000	6823	323

Foundation preparation would consist of removal of all transported soil and debris as well as weathered and highly fractured bedrock in order to expose a suitable clean foundation of sound rock for the gravity dam. It was assumed for this estimate that excavation would extend 10 feet into the exposed bedrock on each abutment and to the top of bedrock in the river floodplain. A grout curtain will be necessary along the entire length of the dam. The dam, outlet works and spillway designs are described in Section 8.2.

The detailed cost estimates for the RCC alternative at Canyon Dam are included in Appendix H and are summarized in Table 25.

TABLE 25

CANYON: RCC Dam Costs

	STORAGE CAPACITY (Acre-Feet)		
	50,000	150,000	300,000
Estimated Cost	$31,104,000	$53,356,000	$80,353,000

10.5.8 Findings and Conclusions

The surface investigation at the Canyon Dam site showed that an acceptable foundation for an embankment dam exists at the upper of the two centerlines drilled. The foundation for an RCC gravity dam is questionable due to the apparent low compressive strength of the sandstone of the Entrada Formation. Further investigation and testing would be necessary to determine the feasibility of an RCC design. In addition, the height of the RCC alternative might be limited to the elevation of the contact between the hard sandstone of the lower Morrison Formation and the highly weathered green siltstone and sandstone of the upper part of that unit. The downstream centerline is infeasible due to a deep landslide on the left abutment and a fault and intensely fractured and weathered zone on the right abutment.

The seismic refraction lines across the valley floor showed the alluvium at the site to be thinner than at any of the other sites studied in the upper canyon. Although the boreholes at the base of the abutments revealed the bedrock to be about 40 feet deep, in the center of the valley it is apparently only 10 to 20 feet deep. The lesser depth to bedrock should have a positive effect on the economic feasibility of the site, especially the competitiveness of an RCC alternative.

Geologic mapping at the site identified three northwest trending normal faults within one mile of the dam. As discussed earlier in Section 10.4.4, the faults are not thought to be potentially active. Any future studies of the Canyon site would have to address these faults in more detail than provided by the scope of this study.

Excavation of the side channel spillway cuts might present problems of stability because of the landslide on the left abutment. Future site investigations would have to characterize this slide in more detail.

Adequate construction materials are located within a short distance of the damsite. Alluvial sand and gravel suitable for RCC aggregate, conventional concrete, filter and drain material, or shell material for an embankment dam, is available in the floodplain immediately upstream and downstream of the dam. In addition, terrace gravels are being explored in commercial pits a few miles downstream of the site. Impervious core material is available from two downstream sites within one mile of the dam axis on either side of the river.

The reservoir formed by Canyon Dam is not expected to present any problems in regard to permeability or slope stability.

The embankment dam alternative at the Canyon site is slightly less expensive than Warner Point for the 50,000 acre-feet reservoir, slightly more at 150,000 acre-feet and equal at 300,000 acre-feet. The estimates are so close that within the accuracy of this level of design and cost estimate, the Canyon and Warner Point embankment dams can be considered equal at all three heights studied. Unlike Warner Point, however, the centerline at the Canyon site cannot be adjusted to coincide with the height of dam selected. The RCC dam alternatives at Canyon Dam are all about 30 percent higher than at Warner Point.

10.6 POWELL PARK DAM

10.6.1 History

A dam, located in the relatively narrow canyon between Powell Park and the confluence of Piceance Creek with the White River, was first studied by the USBR in their reconnaissance of the Upper Colorado River Unit in the late 1960s and early 1970's. No subsurface investigation or project layouts were done at that time.

In 1972, the consortium of Woodward-Clevenger and Jex Engineers prepared a report entitled "Engineering and Geologic Investigations for Powell Park Dam" for Cameron Engineers. The study was conducted for Occidental Petroleum Corporation to evaluate water supply for their future oil shale project located in the Piceance Basin. A surficial geologic map was prepared with that report, but no subsurface exploration was undertaken. The site recommended in that report was located approximately 2.5 miles downstream of Powell Park.

In IECO's Yellow Jacket Project report of 1982, the existing geotechnical data was reviewed and a cursory field reconnaissance was made of the area. No new mapping or subsurface investigation was carried out; however, new layout designs and cost estimates were prepared.

In the 1983 White River Study, another limited geological field investigation was made of all the sites between Powell Park and Piceance Creek. At that time, all of the three sites discussed previously in Section 9 were identified. The site that appeared to be the most economical at that time, especially for a moderately high dam, was located at the head of the canyon, at the lower end of Powell Park. The advantage of this axis was the near-200-foot high rock pediment on the left abutment. Due to limited funding and access, only one borehole was drilled about 1.7 miles downstream of the recommended site and about 0.8 miles upstream from the old Occidental site. The hole showed the alluvial gravel to be about 25 feet thick. Revised layout and cost estimates were prepared for the new site.

10.6.2 Reservoir Characteristics

The reservoir formed by Powell Park Dam would, depending on location and height, inundate Powell Park to some degree. The lower elevations of the Park adjacent to the White River and its main tributary, Strawberry Creek, are irrigated land dedicated to hay production and livestock grazing. The gentle slopes down to the floodplain and the rolling hills beyond are generally arid undeveloped land covered with sage brush. The highest dam investigated for this study at the upstream site would back water up as far as the state highway bridge just downstream of Meeker and would inundate the State Historic Monument at the site of the Meeker massacre. Plate 41 shows the location of the Powell Park Dam selected for subsurface exploration and the area that would be flooded by reservoirs of 50,000 acre-feet, 150,000 acre-feet, and 300,000 acre-feet.

The new reservoir would act as a regulatory and storage facility. The reservoir would fill in the spring and remain full most of the year. Water would be released in late summer to meet downstream agricultural requirements and emergency water for oil shale development in the Piceance Basin. The reservoir should serve as a year-round recreational facility. There is some potential for installing a run-of-the-river power facility.

10.6.3 Site Geology

Plates 7 and 8 are geologic maps of Powell Park and the Canyon downstream. Plate 42 shows the surficial geology of the damsite. The site is located astride the contact of the Wasatch Formation and the overlying Lower Member of the Green River Formation. Both geologic units are of Tertiary age. The contact is conformable, with bedding striking northwest and dipping 10^{o} to 20^{o} to the southwest. The Wasatch Formation is characterized in Powell Park by gently rolling topography. It is composed primarily of claystones and clayey sandstones with occasional thick layers of more competent sandstone. Surface outcrops of the weaker rocks of the Wasatch are characterized by a wide variation in color from purple, to red, to yellow. Vertical outcrops of

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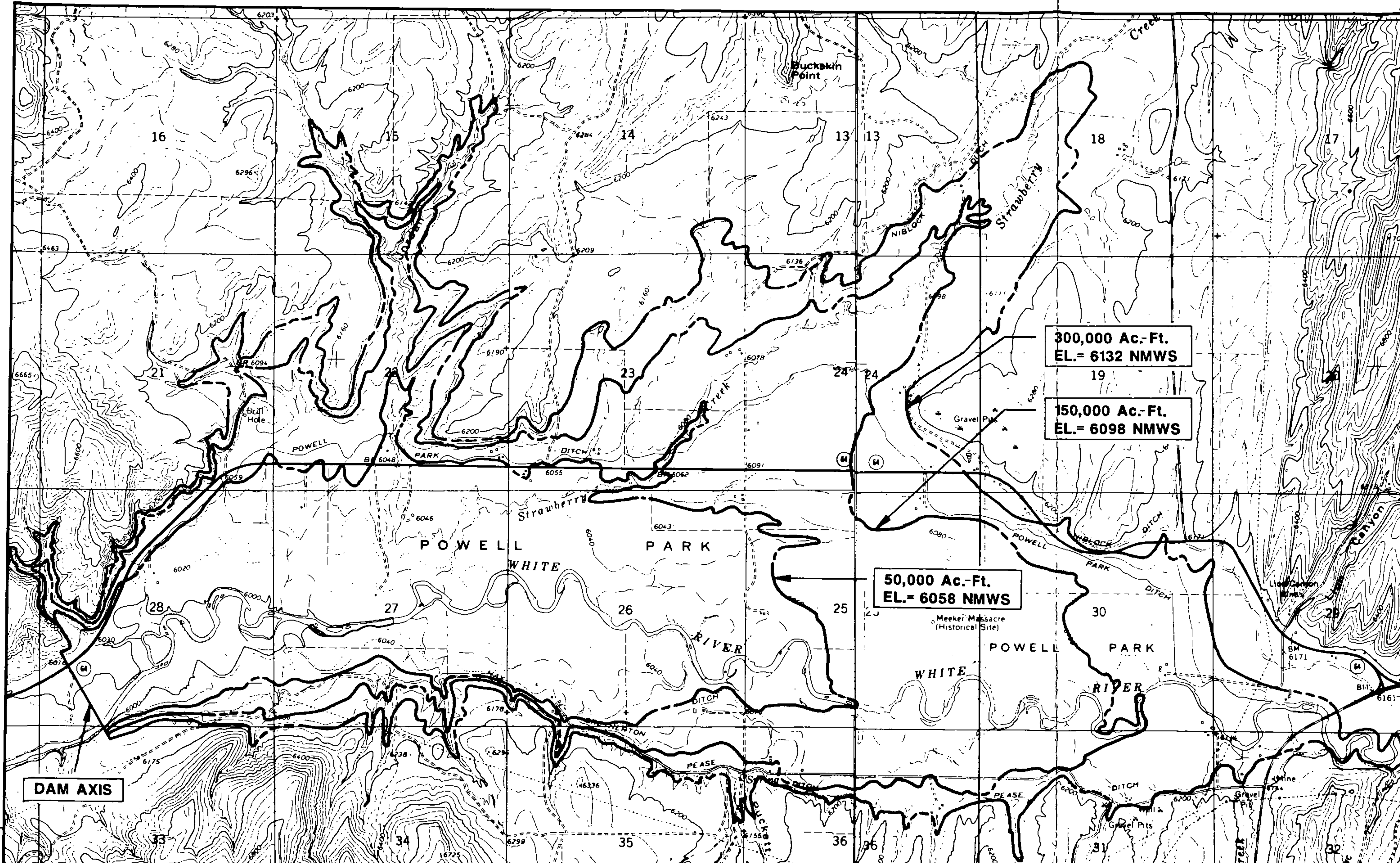

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WHITE RIVER GEOTECHNICAL STUDY RESERVOIR AREAS
POWELL PARK DAM

MKE NO.
SHEET OF REV
Plate 41

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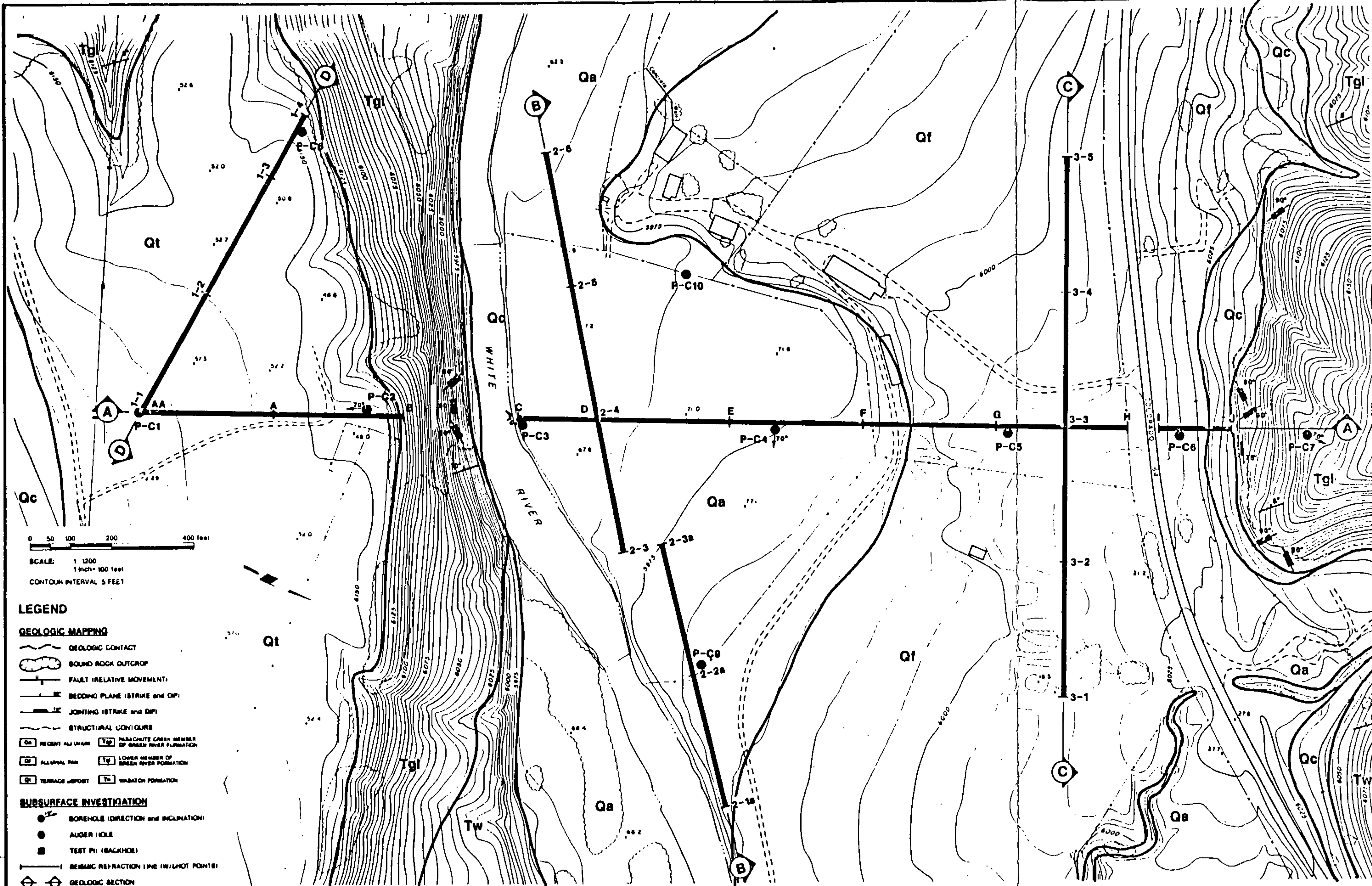

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WHITE RIVER GEOTECHNICAL STUDY
GEOLOGIC MAP/LOCATION PLAN
POWELL PARK DAM

SHEET OF REV.
Plate 42

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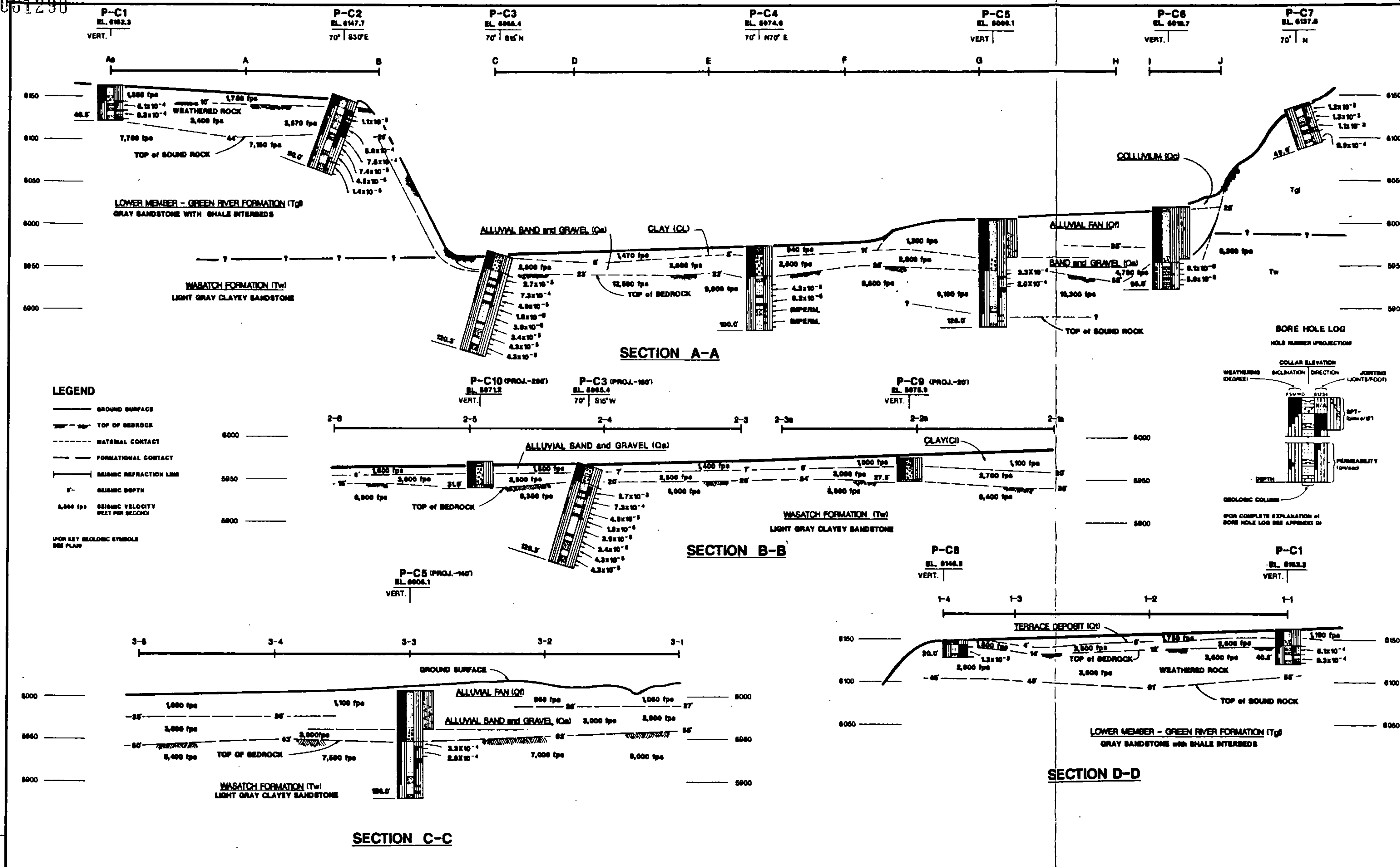

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WHITE RIVER GEOTECHNICAL STUDY
GEOLOGIC SECTIONS
POWELL PARK DAM

IECO NO.
SHEET OF REV
Plate 43

the formation occur near the damsite where protected by the overlying sandstone of the Green River Formation. The contact disappears below Quaternary deposits just upstream of the dam centerline.

The Lower Member of the Green River Formation forms steep hills with nearly vertical canyon walls on both sides of the river above the contact. Surface outcrops of the units are characterized by a series of thick (10 to 40 feet) layers of brown to yellow sandstone separated by thinner beds of gray shale and siltstone. The rock is highly jointed with most joints occurring along bedding planes. A principal set of vertical joints trends northwesterly and an additional set of vertical stress relief cracks has formed parallel to the valley.

The narrowness of the White River Valley at the damsite is caused by the difference in erosional resistance of the Wasatch and Green River Formations. In the more resistant Green River Formation the river has incised a fairly deep channel through the sandstone until reaching the underlying weaker Wasatch Formation which has eroded more evenly over a large area.

The Lower Member of the Green River Formation is overlain by the Parachute Creek Member which is composed in part of Kerogen rich marlstone. This "oil shale" unit crops out to the southwest of the damsite, forming near-vertical, light-colored cliffs that mark the limit of the Piceance Basin. The cliffs are capped by weaker sandstones and marlstones of the Uinta Formation that form the core of the basin.

Two parallel east-west trending faults were preliminarily mapped by the USGS less than one mile north of the right abutment of the dam. Neither of these faults was confirmed in the field by the surficial mapping, but neither were they disproved. Any fault activity in the vicinity is probably associated with the latest period of uplift in the area that formed the Piceance Basin, about 28 million years ago. Therefore, it is not likely that the faults are potentially active and would not be a factor in dam feasibility.

The bedrock units have been covered during Quaternary time by transported deposits of several origins. The oldest of these deposits are the terrace gravels that mask the left abutment. They are in turn partially covered by colluvial deposits originating from the adjacent hills formed by Green River sandstone and shales. The present river channel is composed primarily of alluvial gravels originating upstream from basalt, sandstone, and crystalline rocks. These gravels are overlain by a thin layer of fine grained flood-plain soils. The most recent Quaternary deposits are the alluvial fans emanating from the drainages on both sides of the river. Above the damsite, these deposits are composed of fine material derived mainly from the Wasatch Formation. Further downstream the fans are more heterogeneous containing large blocks of sandstone from the Lower Green River Member and flakes of oil shale from the Parachute Creek Member of that same formation.

The left abutment of the proposed dam is formed by a near vertical cliff of Green River sandstone and shales. The top of the cliff is a pediment overlain by terrace gravel and colluvium that slopes gently towards the river and is covered with sage. Adjacent to the river at the base of the cliff, talus debris has accumulated. A wide (1500 feet) flat floodplain occurs on the floor of the valley and overlies gently dipping bedrock of the Wasatch Formation. Two alluvial fans, originating from side tributaries to the north, join near the dam to form a long bench sloping gently toward the floodplain. Up to State Highway 64 the bench has been planted in hay, and the area between the highway and the steeper right abutment slope is occupied by two irrigation ditches. The right abutment slopes more gently than the left and is formed by alternating outcrops of Green River sandstone and shale that have resulted in a series of vertical sandstone cliffs, and shale benches. The base of the slope is covered with colluvial debris.

10.6.4 Foundation Investigation

Seven cased rotary boreholes were drilled along the dam centerline, one each at the upstream and downstream toe of the highest earthfill configuration, and one along a potential spillway alignment on the left abutment. Lines of seismic refraction were run along the centerline, perpendicular to the centerline in the flood plain and an alluvial fan, and parallel to the spillway centerline. The location of the borings and seismic lines are shown in plan on Plate 42 and in section on Plate 43.

The four holes drilled in the present river floodplain, P-C3, P-C4, P-C9, and P-C10, showed a surface cover of one to 14 feet of thick brown sandy clay overlying 15 to 20 feet of fresh alluvial sand and gravel. The bedrock encountered was a light gray, clayey, silty, fine-to medium--rained sandstone of the Wasatch Formation. Except for the top few feet, the rock was fresh to slightly weathered, with almost no joints other than partings along bedding planes. Holes P-C3 and P-C4 were inclined 70^o to intercept the maximum number of subvertical joints, but few were detected. Water pressure tests in these holes showed the rock to be moderately permeable in the top 20 feet with permeabilities ranging from 2.7×10^{-3}cm/sec to 7.3×10^{-4}cm/sec. Below that depth, the rock is tight with values from 5.2×10^{-5}cm/sec to impermeable, within the time duration of the test.

Boreholes P-C5 and P-C6 were drilled vertically through the alluvial fan. The material recovered was a uniform light gray fine sand about 50 feet thick overlying 10 to 15 feet of alluvial gravel. Undisturbed samples of sand were obtained for laboratory testing to determine its suitability as foundation material. The test results are presented in Appendix E. Blow counts average less than 15 in the sand except where hard rock fragments were encountered. Bedrock was a slightly more weathered version of the light gray, clayey sandstone of the Wasatch Formation revealed in the other four holes. Permeabilities were lower, ranging from 3.3×10^{-4} to 5.6×10^{-6}cm/sec.

Holes P-C1, P-C2, and P-C8 were drilled into the left abutment terrace. They revealed zero to 10 feet of transported silty sand overlying five to 10 feet of weathered terrace gravel. The bedrock was moderate to highly weathered sandstone, siltstone, and shale of the Lower Member of the Green River Formation. The rock was highly jointed and oxidized. Most joints were parallel to bedding planes but the inclined hole revealed high angle stress relief joints as well. Rock quality improved below a depth of 50 feet from the surface. Packer tests showed the upper rock zone, down to about 25 feet, to be moderately permeable with values from 1.1 x 10^{-3}cm/sec to 5.1 x 10^{-4}cm/sec. Below that the rock was tighter with permeabilities of 7.4 x 10^{-5}cm/sec to 4.5 x 10^{-6}cm/sec.

Hole P-C7 was located on the right abutment and drilled into a sandstone outcrop at an angle of 70^{0}. The borehole penetrated a alternating sequence of slightly weathered, tan, fine-to medium-grained sandstone and weaker but only slightly weathered gray siltstone and shale. The rock was moderately jointed, with about half the joints being open or calcite-filled in the plane of bedding. The other principal set of joints was subvertical, open near the surface and tighter with depth, trending N50^{0}W. This corresponds to the regional jointing pattern but also, at this site, is parallel to the valley walls and may be more open due to unloading towards the valley. Packer tests showed the rock to be permeable ranging from 1.3 x 10^{-3}cm/sec to 9.9 x 10^{-4}cm/sec over the entire length (48.5 feet) of the hole.

The seismic refraction surveys correlated well with the boreholes except in the area of the alluvial fan. Line Aa to B along the centerline and 1-1 to 1-4 along the possible spillway alignment were both run on the left abutment terrace deposits. They showed a layer of silty sand with a velocity of 1800 fps or less, about four feet thick, overlying five to 10 feet of terrace gravels with an average velocity of 2500 fps. Top of bedrock occurs at a depth of 10 to 15 feet but appears to be fractured and weathered, registering velocities in the 2400 to 3600 fps range. The top of sound bedrock occurs at a depth of 25 to 55 feet and is characterized by velocities of from 5000 to 7700 fps.

Seismic lines C-F and 2-1a to 2-6 show the top soil covering the river flood plain to have velocities less than 2000 fps and extending to a depth of five to eight feet. The alluvial gravel ranges in total depth from 16 to 26 feet, and shows velocities in the 2500 tp 3000 fps range. Sound bedrock underlies the gravel and is characterized by velocities of from 8300 to 12,500 fps.

Lines F-J along the centerline and 3-1 to 3-5 perpendicular to it cover the alluvial fan at the base of the right abutment. The seismic data in this area do not correlate as well with the borehole data as in the other areas. They consistently showed the fine sand with velocities of less than 1200 fps, to be 15 to 20 feet thinner than did the boreholes, probably because of an increase in rock fragments in the lower part of the fan. The lower fan and alluvial gravel plotted together with velocities ranging from 2800 to 4700 fps. The top of bedrock detected by the seismic refraction corresponded to the drill holes, showing a depth of 55 to 65 feet. The seismic line along the centerline did indicate a dip in the top of bedrock to 85 feet adjacent to the right abutment, under the highway.

10.6.5 Construction Materials Investigation

Plate 44 shows the locations of the auger holes and test pits that were carried out in support of the exploration for construction materials.

It appears that the fine sand in the alluvial fan on the right side of the river is suitable for a transition zone or as filter material with little or no processing. Required excavation of the floodplain overburden will produce a plastic clay probably suitable for use in the impervious core of the dam. Depending on the requirements for borrowed gravel in the final design, a significant quantity of this clay might become available as overburden excavation. Additional sources of impervious material were investigated upstream and downstream of the damsite. Within a mile upstream of the dam, an adequate quantity of white-to-buff, dry, slightly plastic clayey sand occurs in sufficient quantity to meet project requirements. However, laboratory testing of this material showed it to be of questionable plasticy characteristics. Small deposits of high plasticity material were located within this area, but quantity is questionable and further investigation would be neccessary to determine if adequate volume exists.

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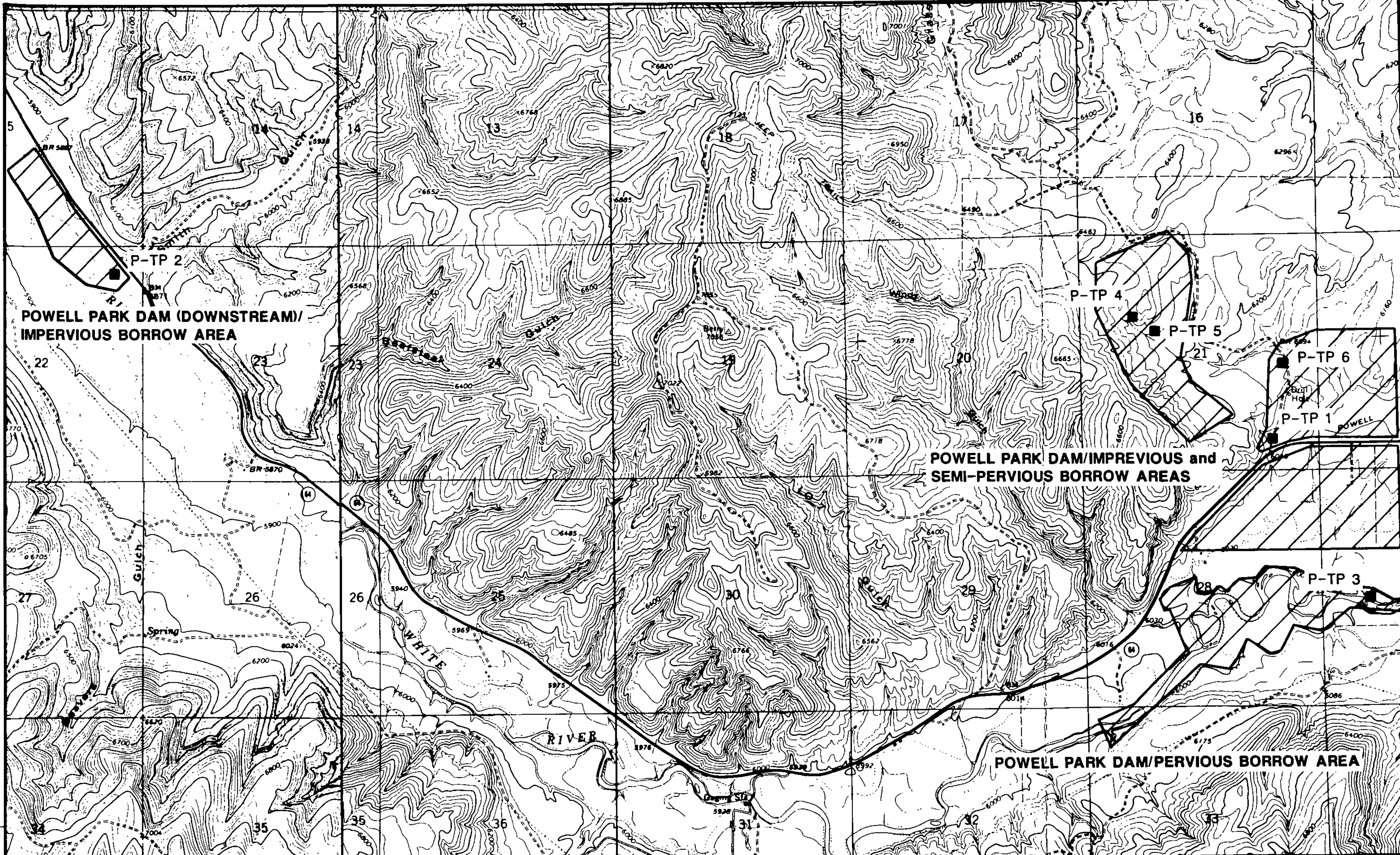

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WHITE RIVER GEOTECHNICAL STUDY CONST. MATERIALS/PLAN
POWELL PARK DAM

IECO NO
SHEET OF REV.
Plate 44

An additional quantity of impervious fill is available from the alluvial fans about one mile downstream of the damsite. Although these deposits show generally more plasticity than the upstream borrow areas, they are more heterogeneous with blocks of rock and accumulations of unconsolidated oil shale flakes.

Slope protection and riprap sources were not investigated specifically, however, it is thought that required rock excavation from the spillway will produce enough riprap for the dam. Samples were obtained from the auger holes and test pits for laboratory testing. The test results are included in Appendix E and summarized in Table 26.

TABLE 26

POWELL PARK: Construction Materials Test Results

DESCRIPTION

Testing	Upstream Impervious (CL/ML/SP)	Downstream Impervious (CL/SC)	Gravel (GP)
Plasticity Index	8	13	NP
"In Situ" moisture	6%	8%	NA
Optimum moisture	12%	15%	NA
Optimum density	120 pcf	112 pcf	195 pcf
Shear Strength	C=14 psi, $\emptyset=41^{0}$	C=14 psi, $\emptyset=41^{0}$	C=0, $\emptyset=40^{0}$
Permeability	6.3×10^{-6} cm/sec	6.3×10^{-6} cm/sec	1.0×10^{-2} cm/sec

10.6.6 Engineering Geology

Surficial geologic mapping and subsurface investigation by drilling and seismic refraction indicate the Powell Park Dam site has an adequate foundation for an embankment dam. A positive cutoff trench with temporary slopes of 1H to 1V would be excavated through the alluvium in the valley floor to the clayey siltstone bedrock of the Wasatch Formation. The contact of the impervious core with the bedrock can probably be prepared with heavy equipment due to the soft impervious nature of the rock. A grout curtain is not expected to be necessary under the valley floor. To provide an adequate foundation for the upstream and downstream shells of the dam in the current river flood plain, the top five to 10 feet of clayey soil would have to be removed. The exposed sand and gravel should be of comparable strength to the shell zones. The alluvial fan material on the right side of the valley does not appear to be of sufficient strength to support the dam. It is likely that the entire 50 feet thickness of this zone overlying river alluvium would have to be removed as far upstream and downstream as necessary to catch the toes of the embankment.

The outcropping bedrock of the left abutment, which is characterized by a near vertical cliff, will have to be laid back to a uniform slope to prevent possible differential settlement. In addition, the near surface portion of the outcrops containing open weathered joints should be removed, especially in the core trench. The gravel portion of the terrace deposit overlying the weathered rock of the left abutment will probably provide an adequate foundation for the shells of the largest embankment dam. A core trench should be excavated through the gravel at least 10 feet into the top of the weathered rock. Extensive grouting is expected to be necessary to cut off seepage through the left abutment, as packer tests in the boreholes show the upper 40 feet of bedrock to be permeable. Foundation cleanup of the relatively hard rock in the bottom of the core trench would have to be by hand and with air and water jets.

The rock outcrops on the right abutment have a naturally flatter slope and are not as poor in quality as on the left. The present ground surface could provide an adequate foundation for the shell zones following minor trimming of prominent ledges. A cutoff trench about 10 feet into rock should be sufficient to provide a base for construction of a grout curtain through the Green River Formation to the top of the Wasatch. The bottom of the trench would be hand cleaned and some slush grouting and dental concrete should be expected in open joints and weathered zones.

Excavation of the spillway cuts through the left abutment, for the two highest dam alternatives, may result in some slope stability problems due to the highly fractured nature of the upper 50 feet of the bedrock. Another factor affecting stability is that both the northwesterly-and northeasterly-trending principal vertical joints in the abutment would strike at a skew of about 45 degrees to the channel alignments. These joints, together with bedding plane joints, could provoke instability of wedges in the walls of the rock cuts. This problem could be decreased to some extent by providing sufficiently wide benchs in the vertical cut to provide stability and catch any loose rock that might fall. In addition, some rock bolting and shotcreteing may be necessary. The channel through the right abutment, as proposed for the lowest dam, would share some of the same stability problems as the left abutment channels except that the rock is less weathered and jointed.

Slope stability and permeability of the Powell Park Reservoir are of negligible concern. The entire reservoir would be within the tight clayey sandstone and claystone of the Wasatch Formation. This formation is characterized within the reservoir area by gently rolling hills that in no way could be expected to produce landslides during reservoir operation that might affect stability of the structure.

10.6.7 Designs and Cost Estimates

Based on the foundation and materials information gathered from the geotechnical investigation, appraisal-level designs and cost estimates were

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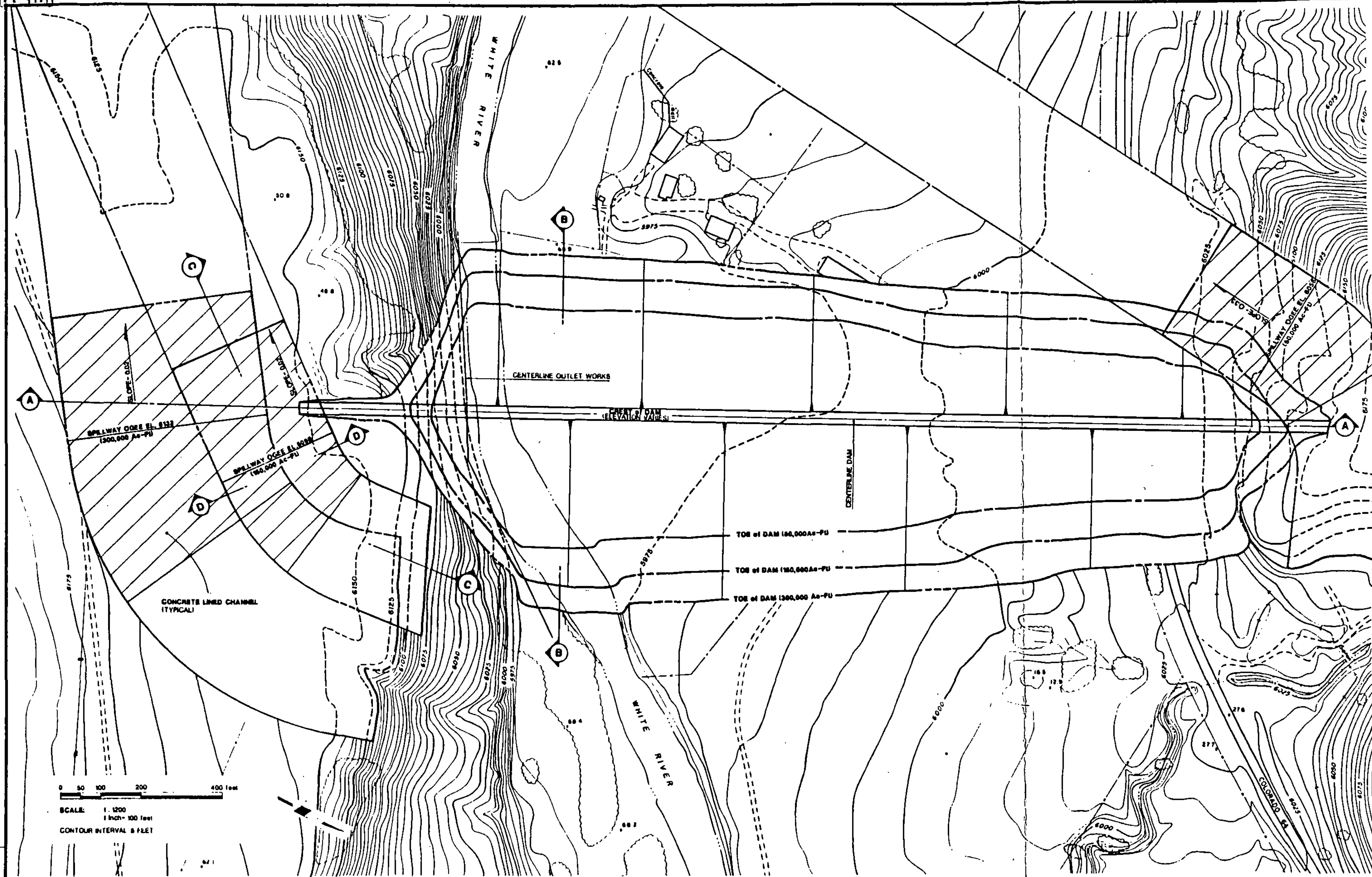

NO	DATE	REVISIONS	BY	CHK	APP'D

MORRISON-KNUDSEN ENGINEERS, INC.

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DESIGNED	DRAWN	CHK	RECOMMENDED
DATE			APPROVED

COLORADO WATER CONSERVATION BOARD

WHITE RIVER GEOTECHNICAL STUDY

EMBANKMENT DAM/PLAN

POWELL PARK DAM

SHEET	OF	REV.
Plate 45		

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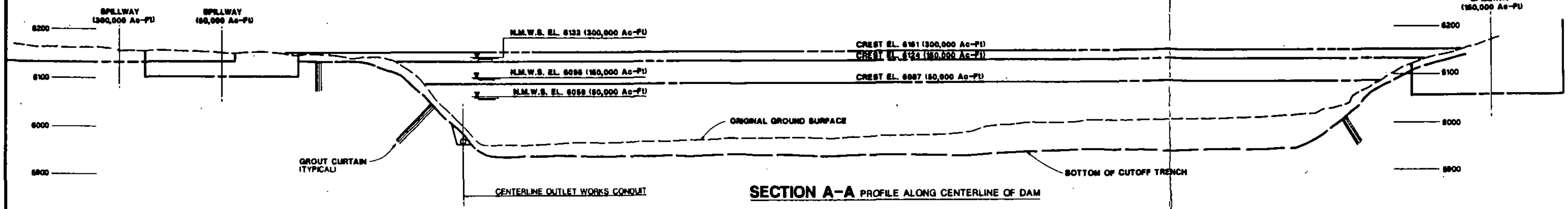
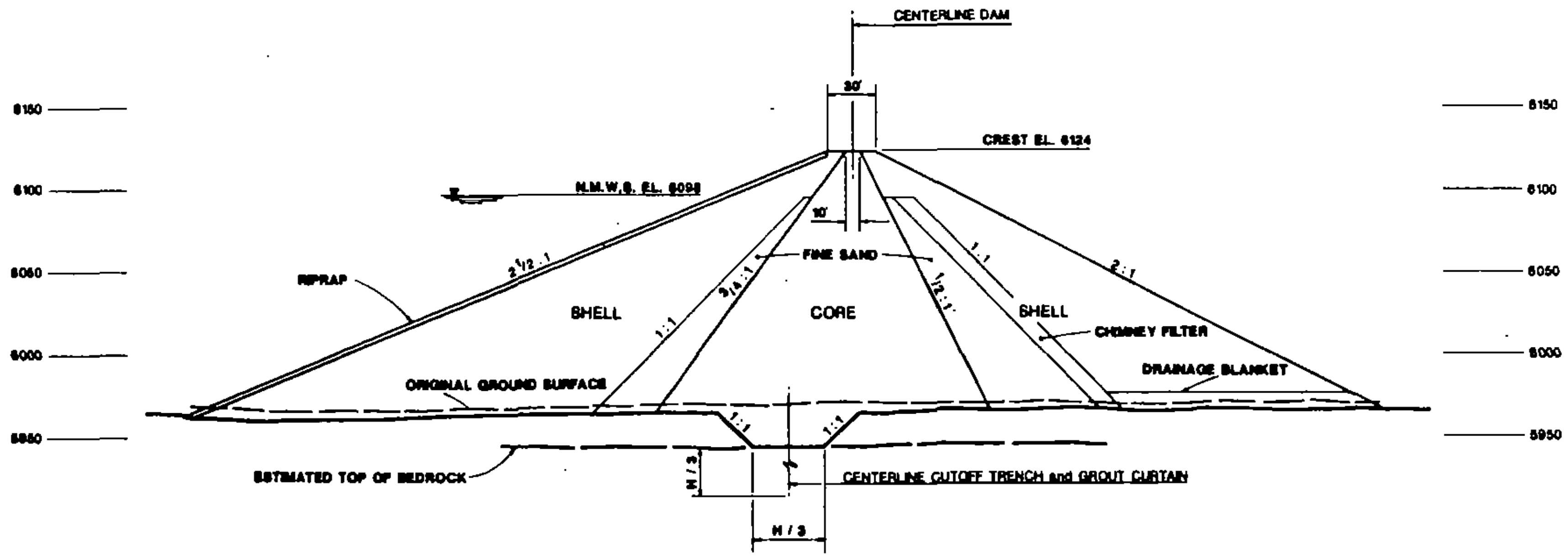
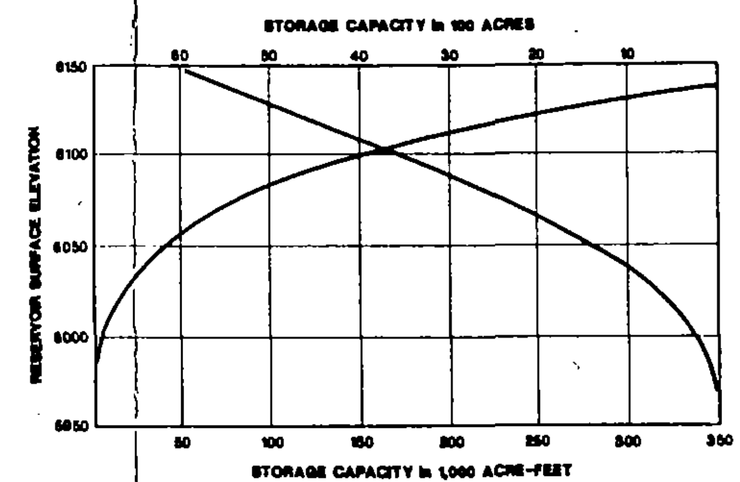


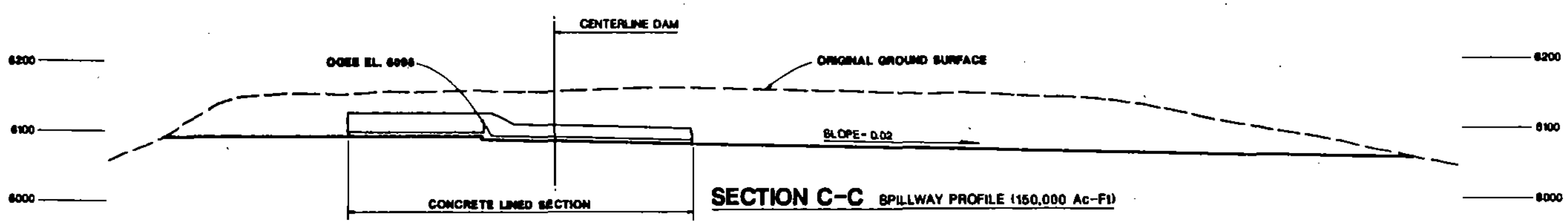
SECTION A-A PROFILE ALONG CENTERLINE OF DAM



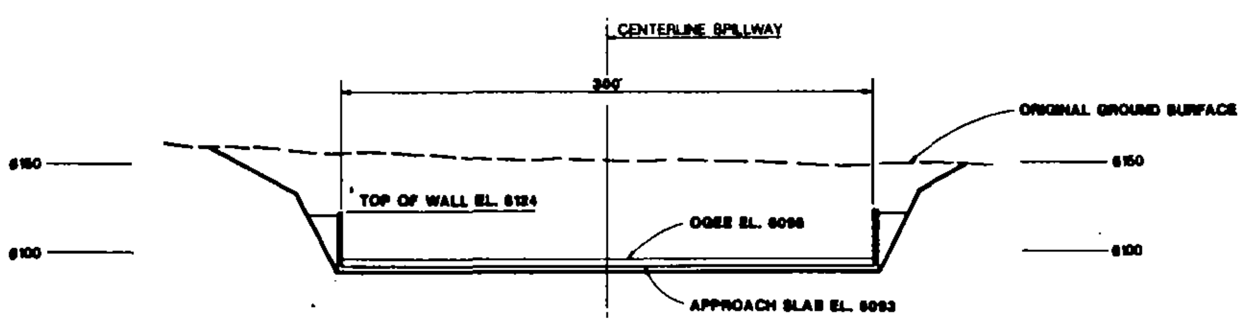
SECTION B-B MAXIMUM EMBANKMENT SECTION (150,000 Ac-Ft)



RESERVOIR AREA/CAPACITY



SECTION C-C SPILLWAY PROFILE (150,000 Ac-Ft)



SECTION D-D SPILLWAY CREST (150,000 Ac-Ft)

TABLE of ESTIMATED QUANTITIES

ITEM	RESERVOIR SIZE in ACRE FEET		
	50,000	150,000	300,000
COMMON EXCAVATION (Cu. Yd.)	1,646,909	1,558,406	1,946,673
ROCK EXCAVATION (Cu. Yd.)	851,020	1,086,316	703,593
CORE (Cu. Yd.)	786,265	1,244,667	1,732,343
SHELL (Cu. Yd.)	1,938,437	2,185,121	4,350,221
RIPRAP (Cu. Yd.)	56,109	78,970	99,376
GROUTING (L.F.)	484	841	1,397
STRUCTURAL CONCRETE (Cu. Yd.)	13,770	13,670	21,565
LAND ACQUISITION (Ac.)	2,940	4,884	6,174
HIGHWAY RELOCATION (Mi.)	2.6	6.9	6.9

NO	DATE	REVISIONS	BY	CHK	APP'D

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COLORADO WATER CONSERVATION BOARD

WHITE RIVER GEOTECHNICAL STUDY
EMBANKMENT DAM/SECTIONS
POWELL PARK DAM

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SHEET OF REV.
Plate 46

prepared for dams at the Powell Park site with storage capacities of 50,000 acre-feet, 150,000 acre-feet and 300,000 acre-feet. Preliminary comparisons between embankment and roller compacted concrete gravity sections showed the embankment design to be much more economically feasible. In addition, the bedrock of the Wasatch Formation is of questionable bearing strength to serve as a foundation for a gravity dam.

The embankment dam designs are shown in plan on Plate 45 and in profile on Plate 46. The three sizes of embankment would have crest elevations, heights above present river bed, and normal maximum reservoir surfaces as shown in Table 27.

TABLE 27

POWELL PARK: Embankment Dam Characteristics

Storage Capacity (ac-ft)	Crest Elevation (ft msl)	Height (ft)	MMWS (ft msl)
50,000	6087	127	6058
150,000	6124	164	6048
300,000	6151	191	6132

Foundation preparation would consist of complete removal of all terrace deposits, and colluvial debris, excavation of a cutoff trench through the pervious alluvium of the valley floor and 10 feet into weathered rock where exposed on the left and right abutments. The upstream and downstream shells would be founded on in-place alluvial gravel and weathered rock. A grout curtain was not considered to be necessary where the foundation of the dam is composed of the fine silty sandstone of the Wasatch Formation that underlies the valley floor. However, where the dam foundation on the abutments is made up of fractured sandstone and shale of the Green River Formation, a grout curtain was provided.

The embankment would be a zoned earthfill design as described in Section 8.1. The impervious core material would be derived entirely from the borrow areas described in Section 10.2.5. A transition zone constructed of material from the required excavation of the alluvial fan on the right side of the valley floor would be included between the core and shells. The upstream and downstream shell zones would be constructed of rock and gravel derived from required excavation of the dam foundation and spillway, complemented by borrowed gravel as necessary. The upstream slope would be protected with riprap also derived from rock excavated for the spillway chute.

The outlet works and spillway designs conform to the descriptions outlined in Section 8.1. Details of these designs are shown in Plate 9. The spillway channels are located through the right abutment for the lower dam and left abutment for the two higher dams. On the left abutment spillways, beyond the chute lining, flow would discharged into a natural channel in the bedrock and the energy would be dissipated by the White River channel downstream. On the right abutment spillway, a flip bucket is located at the downstream end of the chute.

The detailed cost estimates for the Powell Park Dam are included in Appendix H and are summarized in Table 28.

10.6.8 Findings and Conclusions

The single most favorable natural topographic feature at the Powell Park Dam site is the nearly 200-foot-high vertical cliff on the left abutment that gives way to a nearly flat bench over 800 feet in width. The bench is capped with 10 to 25 feet of terrace deposits but provides an excellent location for an inexpensive side channel spillway in conjunction with an embankment dam. The cliff is at its highest at the location investigated in this study and, at that point, is most favorable for the largest reservoir size. For the smaller

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TABLE 28

POWELL PARK: Embankment Dam Costs

	STORAGE CAPACITY (Acre-Feet)		
	50,000	150,000	300,000
Estimated Cost	$32,604,000	$44,884,000	$44,806,000

reservoir capacities, a centerline further downstream might be more economical, where the left abutment bench is lower and excavation of the spillway would be less.

Acceptable construction materials for an embankment dam were found in close proximity to the dam. Sand and gravel for the shells and for concrete aggregate is available in the river floodplain and from several terraces occurring at different elevations upstream of the dam, within the reservoir. The required excavation of gravel and rock could be used in its entirety as shell material and riprap. Suitable impervious core material is located just upstream and downstream of the damsite on the right side of the river.

No slope stability or reservoir permeability problems are expected with the Powell Park Reservoir as it would be located wholly within gently rolling terrain formed by claystone and siltstone of the Wasatch Formation.

The cost estimates show Powell Park to be the third least expensive site studied for the 50,000 acre-foot reservoir capacity ranking behind the Warner Point and Canyon sites by about 40 percent and 15 percent respectively. At the 150,000 acre-foot capacity Powell Park was 10 percent to 15 percent more than the least expensive alternatives at Warner Point and Canyon. This is due to the amount of excavation required for the side channel spillways. If the centerline were moved downstream about 2.5 miles where the left abutment bench occurs at a lower elevation relative to the valley floor, spillway costs would be greatly reduced and Powell Park would probably be comparable in cost to Warner Point and Canyon. For the larger 300,000 acre-foot reservoir Powell Park is 15 percent less expensive than any of the alternatives at Warner Point or Canyon.

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CONCLUSIONS

11. CONCLUSIONS

In this section of the report, comparisons and conclusions are presented regarding the results of the geotechnical investigation. It is important to keep in mind that it was not the objective of this study to formulate, size and select a water development project in the White River Basin. While the investigation has presented additional information relating to cost of reservoir storage, each site has unique characteristics in relation to the amount of storage required to provide the necessary yield to meet future water needs. Each potential reservoir location, size, and dam type would also present certain site specific environmental impacts if the project were to be implemented.

It is initially concluded that the basic level of geotechnical information and data on potential damsites in the White River Basin has been significantly advanced by this investigation and that the primary study objectives have been reached.

The following represent the more significant conclusions of the study:

(A) Of the six damsites and associated storage reservoirs evaluated, none can be eliminated from future consideration on the basis of geologic fatal flaws.

(B) While landslides were detected on the left abutments at the Warner Point and Canyon Dam locations, the surface geologic mapping and subsurface exploration revealed that in each case the detrimental effect of these slides can be avoided by either moving the axis of the dam or removing the undesirable material during foundation excavation.

(C) No potentially active faults were identified within the study area. The three faults mapped in the vicinity of the Canyon Dam site were the most prominent instance of faulting near a damsite. The occurance of these faults does not provide a basis to reject the site or to cause major cost increases to the dam. Future investigation of the Canyon Dam site should, however, include a more detailed evaluation of these faults.

(D) The occurence of evaporites of the Eagle Valley-Minturn Formation within the reservoir area of Warner Point and Choke Cherry reservoir areas could potentially result in above normal reservoir leakage. The unconsolidated sands and pervious limestone layers in the foundation area of the Choke Cherry site will require additional exploration beyond the scope of this study to assure that a serious problem does not exist.

(E) Once overburden and weathered materials are excavated at the sites investigated, the foundations at each damsite would have adequate strength to support the dam types and sizes presented in this study.

(F) There is an abundance of construction materials within a relatively a short distance of each of the damsites. Materials derived from required excavation and selected borrow areas can satisfy project requirements for a wide variety of designs and sizes.

(G) With no identifiable fatal geologic flaws to restrict selection of a project with a dam at any of the sites, the selection of the most desireable site will be based on environmental, operational, institutional, and economic considerations. Only general economic considerations are within the scope of this study. The updated designs and cost estimates were based on the new geotechnical and topographic data acquired during the investigation. This information allows the dams to be ranked and compared according to estimated cost for the range of storage capacities. Tables 29, 30 and 31 summarize and compare the estimated costs for each project at reservoir capacities of 50,000, 150,000 and 300,000 acre-feet.

TABLE 29

COST COMPARISON: 50,000 ACRE-FEET RESERVOIR

Dam Location	Dam Type	Total Cost (Million $)	Percent Increase Above Least Cost Alternative	Cost/Acre-Foot ($)
WARNER POINT	(RCC)	$23.5	0	$469
CANYON	(Emb)	$27.7	18	$553
CANYON	(RCC)	$31.1	33	$622
WARNER POINT	(Emb)	$31.4	34	$628
POWELL PARK	(Emb)	$32.1	37	$641
CHOKE CHERRY	(Emb)	$34.4	46	$687
VEATCH GULCH	(RCC)	$35.2	50	$704

At the smallest reservoir size evaluated, the RCC alternative at Warner Point is clearly the least expensive. With all other factors being equal, it should be the only one considered by further studies for mainstem reservoirs of this general size. This is especially true when it is remembered that the sites chosen for this study were primarily based on the best alignment for the largest reservoir size of 300,000 acre-feet. At the Warner Point site the cost of the embankment alternative and probably the RCC alternative for this reservoir size could be reduced by moving the dam axis downstream about 2000 feet. No such flexibility exists at the Canyon site; although, a change in axis is an option at Powell Park. By moving the Powell Park axis approximately 2.5 miles downstream to the original Occidental site, it appears that cost reductions would likely be achieved due to the decrease in spillway costs. This might make the Powell Park Dam somewhat more competitive with Warner Point at the 50,000 acre-foot size. The Choke Cherry and Veatch Gulch sites are significantly more expensive than the others, and should be dropped from future consideration in this size range.

TABLE 30

COST COMPARISON: 150,000 ACRE-FOOT RESERVOIR

Dam Location	Dam Type	Total Cost (Million $)	Percent Increase Above Least Cost Alternative	Cost/Acre-Foot ($)
WARNER POINT	(Emb)	$38.9	0	$259
CANYON	(Emb)	$40.2	3	$268
WARNER POINT	(RCC)	$41.3	6	$276
POWELL PARK	(Emb)	$44.9	15	$299
CHOKE CHERRY	(Emb)	$53.1	37	$354
CANYON	(RCC)	$53.4	37	$356
VEATCH GULCH	(RCC)	$57.9	49	$386

At the intermediate 150,000 acre-foot size reservoir, both embankment and RCC alternatives at Warner Point along with the embankment dam at Canyon are very competitive. Powell Park Dam, for the same reasons mentioned above with reference to reducing costs by moving the axis downstream, should be kept under consideration at this size. All other sites and alternatives are significantly higher and should no longer be considered.

TABLE 31

COST COMPARISON: 300,000 ACRE-FOOT RESERVOIR

Dam Location	Dam Type	Total Cost (Million $)	Percent Increase Above Least Cost Alternative	Cost/Acre-Foot ($)
POWELL PARK	(Emb)	$49.8		$166
WARNER POINT	(Emb)	$58.4	17	$195
CANYON	(Emb)	$58.7	18	$196
WARNER POINT	(RCC)	$60.0	20	$200
CHOKE CHERRY	(Emb)	$70.9	42	$236
CANYON	(RCC)	$80.4	61	$268
VEATCH GULCH	(RCC)	$82.1	65	$274

The Powell Park site becomes the most economical of all the mainstem sites for the largest reservoir considered. Both RCC and embankment dams at Warner Point and the embankment dam at Canyon are reasonably close on total cost and should all remain under consideration if a large upstream storage project were required. All other sites and alternatives should be dropped.

It can be seen from these comparisons that the cost per acre-foot of storage at the mainstem sites decreases dramatically with increased storage capacity. This does not mean that the cost per acre-foot of yield would be proportionally lower than for smaller projects. Once developed storage in the White River Basin exceeds a total of 300,000 to 400,000 acre-feet, the rate at which new yield is produced with additional storage decreases.

(H) Lake Avery Dam, the only off-channel project studied, was evaluated for different storage capacities than the mainstem sites. The results are summarized on Table 32.

TABLE 32

COST COMPARISON: LAKE AVERY

Dam Location	Storage Capacity (Acre-Feet)	Total Cost (Million $)	Cost/Acre-Foot
LAKE AVERY	60,000	45.5	$758
LAKE AVERY	40,000	30.7	$768
LAKE AVERY	20,000	15.7	$787

Although there are potential advantages to off-channel storage that have not been reflected in the cost estimates, it would appear that none of the Lake Avery enlargements are competitive with the smaller 50,000 acre-foot mainstem reservoirs from a strictly economic standpoint. While the unit cost of storage decreases with greater storage, the 60,000 acre-foot size is nearing a practical limit for both the dam and diversion system. This storage can only regulate partial flows of the North Fork of the White River and can never achieve the degree of regulation provided by mainstem reservoirs.

(I) The topographic characteristics of the valley are unique at each of the mainstem damsites. The change in the valley cross section for the various dam heights, strongly influences the relative cost of each reservoir size considered. The valley shape is also a prominent factor in the location and resulting cost of the spillways for each of the dam heights considered.

These topographic changes, over the range of dam heights needed to form the 50,000, 150,000 and 300,000 acre-foot reservoirs, have resulted in the Warner Point Dam being the least expensive at the lower sizes while the Powell Park Dam becomes the most economical at the larger sizes. The relative order of the dam location and types by cost are not the same for any of the three reservoir sizes.

The Warner Point, Canyon and Powell Park dams are within 20 percent of each other for any given size. It is concluded, therefore, that dams at the Warner Point, Canyon, and Powell Park sites should remain in consideration for future water development projects in the White River Basin.

(J) Lake Avery enlargement does provide an alternative for obtaining storage capacity without constructing a dam on the mainstem of the White River. Compared to equal-sized mainstem reservoirs, the construction cost for the larger Lake Avery Dam is substantially greater.

K) For major water resource development to be provided on the White River Basin within the State of Colorado, a moderate to large mainstem reservoir will eventually be required and there appears to be no general geotechnical restriction that would prohibit such a project at one of the three most cost-effective sites identified in this study.

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PUBLIC MEETINGS

12. PUBLIC MEETINGS

Following review and acceptance of this report by the CWCB, residents of the White River Basin and other interested parties will be invited to a public meeting to be held in Meeker. The report will be made available for review to any interested citizens. A presentation will be made at the meeting by CWCB and MKE personnel on the scope, conduct, results, and conclusions of the study. A question and answer session will be scheduled following the presentation for public participation.

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001316

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FINAL REPORT

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