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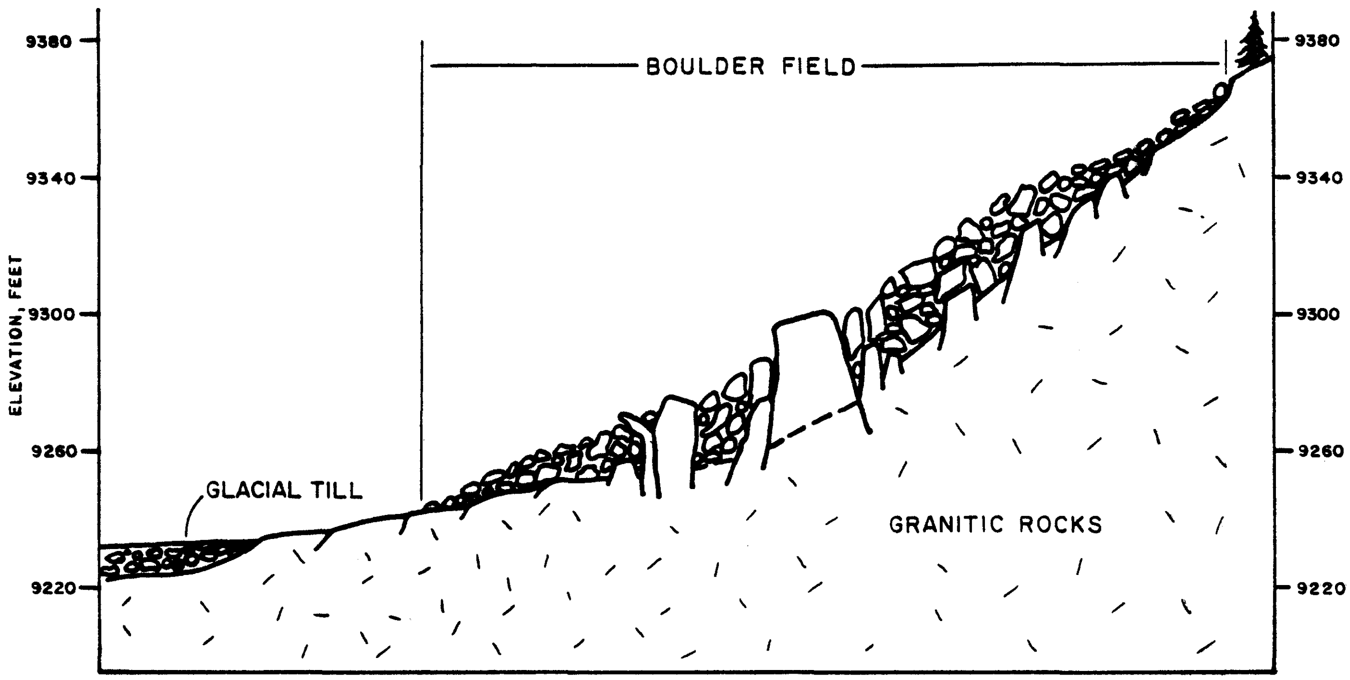
51.1248 (Lake Adelaide)

STATE OF WYOMING
WATER DEVELOPMENT COMMISSION



Contract No. 8-05846

SHELL VALLEY WATERSHED
Level III

PHASE I: Interim Report on Conceptual Design Lake Adelaide Enlargement

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

SHELL VALLEY WATERSHED
Level III

Phase 1: Interim Report on Conceptual Design
Lake Adelaide Enlargement

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I. INTRODUCTION

A. BACKGROUND

Shell Valley is a semi-arid valley located in north-central Wyoming, at the western base of the Big Horn Mountains. The Shell Valley Watershed contains over 370,000 acres with Shell Creek as the major drainage. The majority of water available to this drainage originates in the higher elevations of the Big Horn Mountains which lie to the east of the valley. The entire watershed, located within Big Horn County, is serviced by the town of Greybull (population of 2000) and the community of Shell (population of 50).

Several previous studies have identified the need for additional water for late season irrigation in certain areas of Shell Valley. A recent Level II study (HKM Associates, 1985)[1] performed for the Wyoming Water Development Commission (WWDC) indicated that it would be feasible to expand the water supply by enlarging Lake Adelaide Reservoir, which is owned and operated by the Shell Valley Watershed Improvement District (SVWID). The levels of study for water development projects, as designated by the WWDC, include;

Level I - Prefeasibility of project

Level II - Feasibility and Water Rights Study

Level III - Phase I Final Hydrologic Study, Geotechnical Investigation and Conceptual Design

Phase II Preparation of Final Design Plans and Specifications

Level IV - Construction

In early March, 1985, the WWDC was authorized by the Wyoming Legislature (House Bill 275) to issue a Request for Proposal No. 85-7 for a Level III study of the Shell Valley Watershed.

B. AUTHORIZATION

On June 5, 1985, ESA Geotechnical Consultants (ESA) entered into a contract with the Wyoming Water Development Commission to complete the required Level III study of the Shell Valley Watershed Project.

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1. All references cited herein are presented in alphabetical order in Chapter IX, References.

This contract is in two phases. The Phase I level of effort was specifically directed to conduct the following studies or provide the specified technical services:

1. A geotechnical and geological investigation necessary to support the final design of enlarging the existing Lake Adelaide Reservoir.

2. A hydrological study of the Adelaide Creek and Buckley Creek watershed areas to determine the potential yield of the new reservoir.

3. A hydrologic study to determine the flood potential of the Lake Adelaide drainage basin to provide discharge values for use in sizing the spillways.

4. A hydrologic study to develop a detailed plan for operating Lake Adelaide that will meet the project demand while satisfying regulations imposed by the State Engineer's Office.

5. Prepare conceptual designs illustrating the general project configuration; including, foundation preparation and embankment, diversion from Buckley Creek, service and emergency spillways, outlet works, and access roads.

6. Prepare cost estimates for the conceptual designs, estimate annual operation and maintenance costs, and recommend the suitable design for enlarging Lake Adelaide reservoir.

7. Prepare the necessary documents and permits associated with enlarging Lake Adelaide Reservoir.

8. Assist the WWDC in conducting public workshops, meetings, or hearings related to the project.

9. Prepare an interim report identifying the project configuration and operation that will be pursued during the development of construction plans and specifications.

This report presents the results of the Phase I, Level III studies associated with the enlargement of Lake Adelaide Reservoir.

C. EXISTING FACILITIES

Adelaide Reservoir is located in Section 36, Township 53 North, Range 88 West, on Adelaide Creek, a tributary to Shell Creek in Big Horn County, Wyoming. The town of Shell is located approximately 23 miles downstream (west) of Adelaide Reservoir. The location of the project site is shown in Figure I-1.

Lake Adelaide Dam was constructed in 1915 as a homogeneous earth fill dam with a structural height of 31 feet, a crest width of 15 feet, and a crest length of about 720 feet. The minimum crest elevation is 9259 feet above mean sea level. The upstream slope is inclined at 3:1 (horizontal to vertical) and the downstream slope is inclined at about 2.5:1.

Adelaide Reservoir has a capacity of 1700 acre-feet at the spillway crest elevation of 9254.8 feet. The existing earth cut spillway, which is located in the right abutment, has a bottom width of 30 feet, a length of 500 feet, and a discharge capacity of 620 $ft.^3/sec.$ (cfs).

The existing low level outlet works consists of a 30-inch diameter concrete pipe which has an inlet elevation of 9233, a length of about 169 feet and a discharge capacity estimated at 115 cfs at full reservoir. The outlet pipe is in very poor condition and is discussed in greater detail in Section II-D.

The Buckley Creek watershed is similar in size to, and lies to the east of, the Adelaide Creek watershed. In order to capture a portion of this flow, a diversion was constructed across Buckley Creek in about 1943 to divert water into Mud Lake which drains into Adelaide Reservoir. This diverted flow is critical to the overall scheme of increasing the size of Adelaide Reservoir.

D. PREVIOUS STUDIES

In 1966, the Soil Conservation Service (SCS) conducted an analysis of enlarging Adelaide Lake from the present capacity of 1700 acre-feet to a capacity of approximately 2700 acre-feet. The study also considered several rehabilitation needs including a new outlet, new headgate and a plunge pool basin. No action resulted from the SCS study.

A Phase I inspection was conducted in 1979 by the U.S. Army Corps of Engineers (COE) as part of the National Dam Safety Program. This inspection resulted in the identification of numerous problems including concrete deterioration, stability problems and an inadequate spillway.

In 1982, the newly formed Shell Valley Watershed Improvement District solicited support from the SCS in updating their previous investigation. Based on that investigation, the SVWID applied to the WWDC for funding assistance of the Shell Valley Watershed Project.

In 1984, the WWDC entered into a contract with HKM Associates of Billings, Montana, to conduct a Level II study on the Shell Valley Watershed. This study included geotechnical investigations, a hydrologic and water rights study, engineering study and a financial and economic feasibility assessment. This study, completed in January 1985, was helpful in developing the necessary field investigation, hydrologic investigation and other critical studies completed during the present investigation.

E. PERFORMANCE

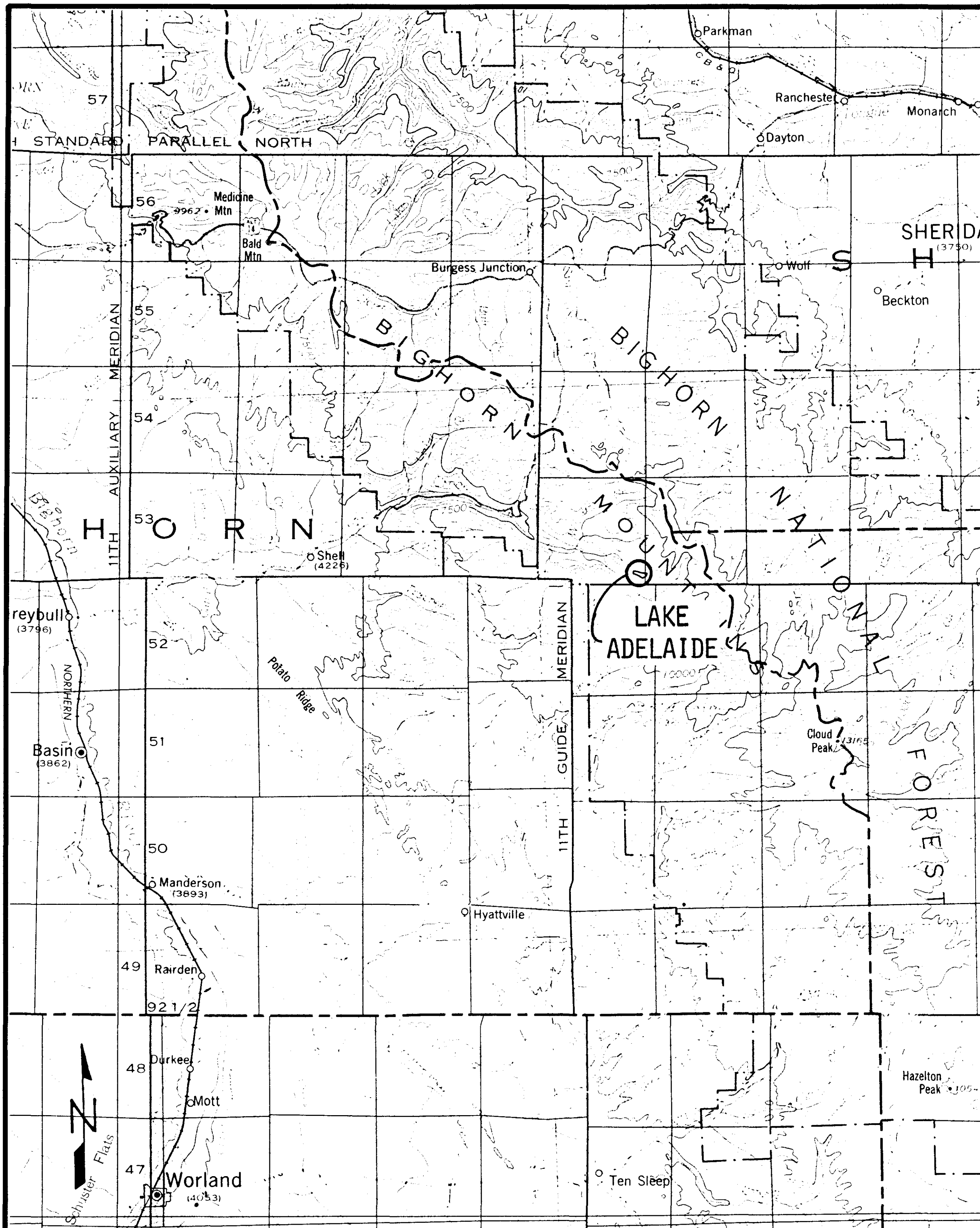
The engineering and geologic studies which form the basis of this report were completed by ESA Geotechnical Consultants of Fort Collins, Colorado. Subcontractor assistance in hydrology was provided by the Boyle Engineering Corporation (Boyle) of Denver, Colorado, and in hydraulic structures and general civil engineering by ARIX Engineers (ARIX) of Greeley, Colorado. The Principal-in-Charge for ESA was W. Roger Hail. Richard L. Volpe was the Project Manager and the registered civil engineer responsible for the overall study. He was assisted by Debora J. Hamberg who completed a majority of the detailed geotechnical engineering analyses and Sally W. Bilodeau who was responsible for the field investigation. For Boyle, Dr. Young Yoon served as the lead engineer and was assisted by Alan Mauzy and Robert Mahoney. For ARIX, Darryl Alleman served as the lead engineer and was assisted by William E. Kelly.

Evan J. Green served as Project Manager for the WWDC. ESA is grateful for his assistance and guidance throughout the project.

F. LIMITATIONS

This report has been prepared by ESA at the request of the WWDC for the purpose of documenting the conceptual design of expanding Adelaide Reservoir to some optimum and cost effective level for Shell Valley users. In order to provide the professional services called for in our contract, it was necessary to use engineering and geologic judgment that led to the conclusions presented herein. In the process of applying such judgment, and performing the required analyses, a standard of care was applied that was in accordance with generally accepted professional engineering practice. No other warranty is made on the professional engineering performed, either express or implied.

This report is considered to document a Level III, Phase I study as defined by the WWDC and, as such, is not adequate for construction of the facilities described herein. Rather, it is intended that the data collected and analyses performed in association with this study will form the basis for completing the final design plans and specifications required for construction.



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ESA Geotechnical Consultants

Fort Collins, Colorado

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

Checked by D.J. Hamilton	Date 10-22-85	Project No. 3235	Figure No. I-1
Approved by R.F. Volpe	Date 10/22/85		

II. PROJECT DESCRIPTION

A. BACKGROUND

The conceptual design alternatives for the Lake Adelaide enlargement are discussed in this chapter. The development of the various design alternatives required that the following concerns be addressed:

1) The construction season at the project site is very short, typically lasting from about the last week of June to the end of September. The shortness of construction season dictates that the planned facilities be constructed over two seasons.

2) The project site is remote. Current access to the site over the last four miles is only passable during the summer months by 4-wheel drive vehicles and requires about 1 hour of travel time. The existing access road must be enlarged to a width of about 15 feet and sufficiently improved to provide relatively easy access to the site for construction equipment and service vehicles.

3) The SVWID requires that the reservoir continue in operation throughout the construction of the new facilities. It will be necessary, therefore, to release reservoir water through the construction site. Also, it must be recognized that, at any time during construction, the existing reservoir could be at maximum capacity and subjected to flooding conditions.

4) The SVWID has a limited budget for the expansion project and would like to develop the maximum increase in reservoir volume commensurate with a safe structure and optimum reservoir yield.

B. RESERVOIR CAPACITY

The area-capacity relationship for the Lake Adelaide area is presented in Figure II-1 and summarized on Table II-1. As discussed in Section I.C, the current maximum reservoir surface is about elevation 9255. The topography of the reservoir area is such that it can be raised to a maximum elevation of 9280 before restraining dikes would be required to prevent water from spilling over the southern

TABLE II-1

AREA-CAPACITY RELATIONSHIP OF LAKE ADELAIDE

Reservoir Elevation (feet)	Reservoir Area (acres)	Incremental Storage Volume (acre-feet)	Total Storage (acre-feet)
9220	1.8		4
		20	
9224	8.6		24
		59	
9228	20.9		83
		110	
9232	33.9		193
		166	
9236	48.9		359
		219	
9240	60.8		578
		260	
9244	69.0		838
		292	
9248	76.8		1130
		321	
9252	83.9		1451
		349	
9256	91.0		1800
		381	
9260	99.3		2181
		413	
9264	107.2		2594
		443	
9268	114.5		3037
		473	
9272	121.7		3510
		503	
9276	129.7		4013
		535	
9280	137.9		4548
		578	
9284	151.3		5126
		430	
9287	161.4		5556

Notes:

1. The area-capacity relationships were computed assuming the upstream slope configuration of Alternative B.

2. The area and capacity values above Elevation 9280 required the assumption of a restraining dike to be placed in the southern portion of the borrow area.

reservoir rim, which is the ridge separating the Adelaide and Shell Creek watersheds. The total storage volume of the reservoir at elevation 9280 is 4548 acre-feet and the corresponding reservoir surface area is 137.9 acres.

The maximum practicable increase in reservoir volume ~~possible~~ at Lake Adelaide, therefore, is from about 1700 acre-feet (current capacity) to 4548 acre-feet. This 168 percent increase in volume could be accomplished by increasing the maximum controlled water surface from elevation 9255 to elevation 9280.

The current minimum controlled water surface is defined by the inlet elevation of the outlet pipe, which is elevation 9233. This water surface elevation corresponds to a reservoir volume of about 219 acre-feet which is dead storage. The conceptual designs for raising Lake Adelaide Dam that are discussed in the following section will result in a slightly larger dead storage volume of 234 acre-feet.

The Level II study proposed to raise the reservoir to elevation 9273 in order to capture a storage volume of approximately 3635 acre-feet. The hydrologic studies performed as a part of this study, which are discussed in Chapter IV, indicate that an optimum reservoir yield of approximately 4100 acre-feet can be reliably developed in 8 out of 10 years, if the reservoir capacity is enlarged to about 4500 acre-feet. This yield value accounts for minimum flow requirements in Adelaide and Buckley Creeks and a dead storage volume of 219 acre-feet. It is estimated that 100 percent of the Shell Canal diversion requirements can be met approximately 70 percent of the time for a reservoir of that size.

After discussing the increased reservoir potential with both the WWDC and the SVWID, it was decided to develop conceptual designs and construction cost estimates for new dams to cover the range of reservoir storage between 3500-4500 acre-feet. The possible dam configurations that would accomplish this increased storage are discussed below. A project configuration to develop the optimum yield is recommended herein, which requires a dam with a crest elevation of 9287 feet and a reservoir storage capacity totaling 4548 acre-feet.

C. DAM ENLARGEMENT

In order to increase the reservoir volume above the current storage capacity, it will be necessary to expand the dam in a downstream direction. Although an upstream expansion is technically feasible, it would require that the reservoir be taken out of operation for a period of at least two years which is not desirable.

The planned dam raise proposed in the Level II studies was to extend the existing upstream slope at an inclination of 3:1 to a new crest elevation of 9280. After evaluating this proposal, it was decided that the new dam could not be founded on the existing dam as planned for the following reasons.

1. The existing dam and the new dam will have totally different stress-strain characteristics. As a result, the likelihood that the new dam would experience cracking due to differential settlement within the existing dam was quite high. The resulting cracking would be totally unacceptable to the continued safe performance of the raised dam.

2. The proposed morning glory spillway was to have been located near the downstream toe of the existing dam. In order to develop the foundation for this structure, it would have been necessary to excavate a very steep cut in the downstream slope of the existing dam. Since the dam has a history of seepage problems, and the excavation would have to be completed under reservoir operating conditions, it was determined that the overall safety of the existing dam could be severely jeopardized by such a cut, especially under full reservoir conditions.

3. As a result of the low density of the existing dam, as indicated by a blow count (N Value) of 7, and since the existing dam would be innundated if it were left in place, there is a possibility that it would either liquefy or deform significantly more than the new dam in the event that a moderate size earthquake occurred near the site. Such distortion of the existing dam would create a major loading condition within the upstream shell of the raised dam that could result in severe cracking or failure.

For the reasons stated above, it was decided to abandon the idea of simply raising the existing dam and to only consider developing the new dam in a manner that did not rely on the support of the existing dam. Three alternative designs for a new dam were evaluated and are briefly described as follows:

1. Alternative A

   The first design evaluated considered a dam with a crest elevation of 9280 and a maximum water surface elevation of 9273, thus creating a reservoir capacity equal to 3635 acre-feet. This configuration is essentially a modification of the structure proposed in the Level II study.

2. Alternative B

   This is the recommended design which would be a dam with a crest elevation of 9287 and a maximum water surface elevation of 9280. It would form a reservoir with a total storage capacity of 4548 acre-feet.

   The design for Alternative B includes a relatively large downstream rockfill zone. The size of this zone is intended to allow the incorporation of oversize material (greater than 6-inches) that will be encountered in the borrow area.

3. Alternative C

   The third conceptual design has the same crest elevation and storage capacity as Alternative B. Rather than a large rockfill zone, however, this design alternative includes only a small rockfill toe, constructed solely from boulders which must be removed from the right abutment. Oversize materials encountered in the borrow area would be removed from the construction materials and wasted. This alternative was evaluated to make cost comparisons associated with handling the oversize materials.

Common to the three conceptual designs is the required treatment of the large boulder field which forms the right abutment of the site. It is intended that the excavated right abutment rock will be incorporated as a downstream rock toe. A discussion of the geologic conditions in the right abutment, and other geotechnical conditions existing at the site, is presented in Chapter III, Geotechnical Conditions. Other significant aspects of the various design alternatives are presented in Chapter V, Embankment Design.

D. SPILLWAY CONCEPTS

Spillway concepts evaluated included (1) a side channel spillway cut through the left abutment ridge in combination with an emergency spillway in the borrow area; and (2) a 6-foot diameter drop inlet (morning glory) spillway, coupled with an auxiliary spillway in the borrow area. The side channel spillway is designed to pass more than one-half the PMF outflow on its own. The remaining portion of the PMF outflow would be handled by the side channel operating in tandem with a relatively crude emergency spillway cut into the southwestern rim of the reservoir inside the borrow area. The drop inlet spillway was designed to handle up to a 100-year storm outflow before an auxiliary structure would go into operation. The auxiliary spillway associated with the drop inlet spillway would have to be a relatively well designed structure compared with the emergency spillway described above, since it would be required to pass the majority of a PMF event on its own and would go into operation more frequently. Either of these concepts are compatible with Alternative B and C dams. However, the drop inlet-auxiliary spillway concept is probably impractical for the Alternative A dam because of the deeper excavation required (partially in bedrock) for the auxiliary spillway.

An incremental damage analysis assuming a dam breach was performed to determine if a side channel spillway alone, that would pass one half the PMF, would be adequate. The results of the incremental damage analysis were considered marginal and the ESA team was directed to design for the full PMF. This was an additional reason for recommending the higher dam. However, Alternative A is judged to still be viable with some relatively minor design changes to reduce incremental damages to an acceptable level and is retained herein as an alternative. The side channel spillway is the preferred option because it is more reliable, involves less construction problems, and passes all but the extremely infrequent flood flows (once in 1000 years or more) back into Adelaide Creek. For Alternative B and C dams, an emergency spillway will be added in the borrow area to supplement the capacity of the side channel spillway to handle the full PMF. This can be accomplished by simply removing enough materials from the borrow area in the natural topographic low area to create a broad overflow channel that would only spill during extremely rare flood events (probably not at all during the life of the project). This emergency spillway can be constructed at little or no additional cost. Further, optimization of the side channel spillway configuration during final design should result in additional flow capacity without increasing costs.

E. OUTLET FACILITY

The outlet works will be constructed within the left side of the valley. Depending on the final spillway configuration, the outlet pipe will consist either of a 36-inch diameter reinforced concrete pipe extended though the entire embankment or a 36-in diameter pipe connecting to the lower portion of a 54-inch diameter morning glory drop inlet spillway within the dam proper. Flow control through the 36-inch pipe will be achieved using a sluice gate mounted at the inlet. The outlet pipes for either arrangement will be extended about 40 feet beyond the downstream toe of the dam and will terminate in a concrete headwall structure with riprap protection. The 36-inch outlet pipe was sized to provide a flow of 120 cfs at a reservoir elevation of 9,245 feet.

F. BUCKLEY CREEK DIVERSION

A gated reinforced concrete structure is proposed to divert water from Buckley Creek into a diversion ditch which will carry water into Mud Lake upstream from Adelaide Reservoir. The diversion works will replace the existing diversion and will be capable of diverting 50 cfs while bypassing 550 cfs during a 100-year flood event on Buckley Creek. The diversion is designed so that flow in Buckley Creek must reach a flow rate of 1.3 cfs before any water is diverted to Mud Lake. Also, if diversion is occurring, a flow of between 1.3 and 2.0 cfs is returned to Buckley Creek.

G. BORROW AREA

The principal borrow area that will be the source of materials to construct the new dam is located approximately 2000 feet due south of dam in an area bordering the western side of Lake Adelaide. This was the same borrow area used for the original dam construction. The limits of the borrow area, which covers a surface area of approximately 25 acres, has been configured such that virtually all required excavation will be confined to the new reservoir area if the largest reservoir is developed. A discussion of the engineering properties of the borrow area materials expected to be developed when the materials are compacted into the dam is presented in Section V.B of the report.

The conceptual designs propose the incorporation of all material from the required excavations into the embankment. However, oversize rock from the borrow area will only be used if it is determined economical to do so. This approach requires a more flexible design, but will result in the lowest possible construction cost. The need to waste materials or incorporate special handling techniques for the embankment zones can be eliminated totally or limited to those required for overall safety of the dam.

H. ACCESS ROAD

The access road improvement plan proposes a route similar to the existing access road. This proposal was made largely to avoid conflict with the nearby wilderness area. Limited blasting and road realignment will take place between Shell Reservoir and Lake Adelaide. Construction for road improvements is planned for July-August, 1986, or the season prior to the start of dam construction.

I. ENVIRONMENTAL CONDITIONS

It is our opinion that the proposed Lake Adelaide enlargement will have relatively little impact on present environmental conditions. However, some impacts will be unavoidable and may be difficult to mitigate completely. The primary impacts will be related to construction activities such as the unusual number of workers in the area and heavy equipment noise and traffic, which would disturb wildlife and limit recreational activities during the construction season. Dam construction will require two summer seasons in 1987 and 1988. Each season will entend from about June 20 to the end of September or as weather permits.

Construction activities at the dam site will involve stripping of soils with heavy equipment and will include some blasting to remove large boulders and to excavate the side channel spillway. Blasting of rock will not be a continuous operation. It will be limited to the first construction season and may be limited to two primary series of shots during the middle of the season (approximately mid to late July). These shots will be required to excavate rock for the side channel spillway area and to reduce boulders in the right abutment area to usable sizes. The contractor will avoid noisy air blasts in order to minimize his explosives costs. The contractor will probably elect to do his primary shooting during a short period of one to two weeks and perhaps all in one day. Some secondary, light blasting will be required to break occasional boulders. This would probably be limited to 10 to 20 low order shots in the dam foundation area. Additionally there would be one rock outcrop to be shot on the access road near the left abutment of Shell Reservoir. Road construction is tenetively scheduled for the summer of 1986.

Construction in the stream channels at the dam site and at the Buckley Creek Diversion will cause some sedimentation impacts. A settling pond will be incorporated in the final design downstream from the dam. At Buckley Creek the stream would be bypassed during construction. The existing natural bypass channel can be used simply by blocking flows to the diversion site. Some turbidity in the streams below both construction sites is unavoidable, but the low clay content of the soils in the area will minimize suspended solids.

Construction of the new dam (Alternative B) will result in covering an area approximately 300 by 300 feet, or about 2 acres, which contains riparian vegetation. Aerial photos show that about 1/2 of this area has a cover of conifers which would limit the amount of actual riparian vegetation to something less than 2 acres. Alternative A covers about 10 percent less area, however because of limited working room, riparian vegetation impacted will be about the same in the short term.

The maximum operational pool will increase from about 90 acres to 138 acres (4548 acre-foot reservoir) or an increase of 48 acres. This area within the maximum operational pool will encompass virtually all of the borrow area except for minor side slopes and an emergency spillway area. The borrow area (25 acres) contains several acres of disturbed land (old borrow areas) resulting from 1915 dam construction. Most of the rest of the reservoir rim area is relatively steep.

The northern rim that will be inundated is a mixture of open grassland and conifer forest with very little wetland and riparian vegetation, including the channel of Adelaide Creek. The southern reservoir rim is also relatively steep except in the borrow area. The southern portion of the new reservoir area is mostly open grassland except for forested areas between the borrow area and the dam.

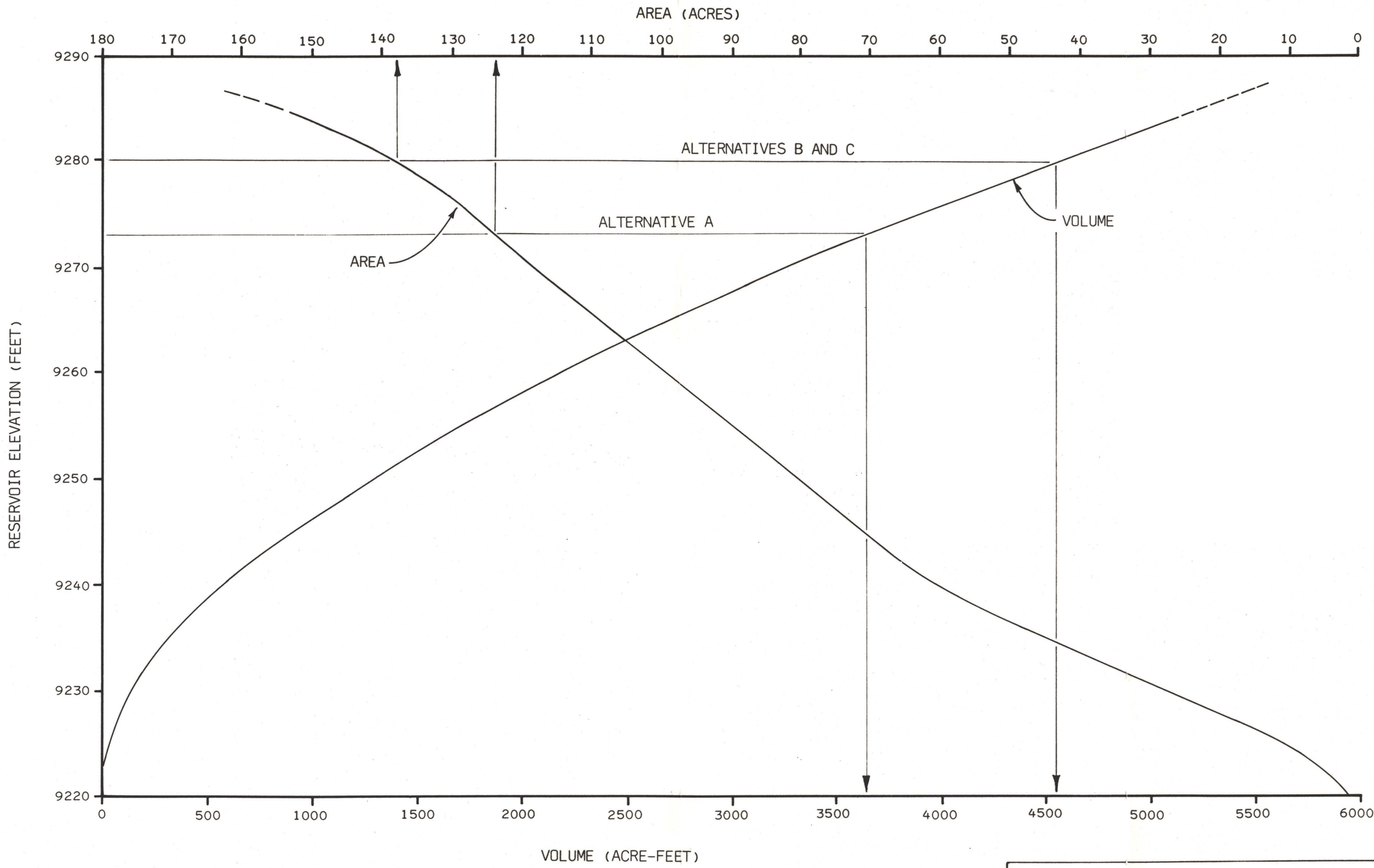
All disturbed areas will be reseeded with suitable grasses, including portions of the borrow area and emergency spillway above the high water line and along the access road. Mitigation of the loss of riparian vegetation may be possible by planting suitable shrubs in the vicinity of the emergency spillway and around the shallow bay formed by the borrow area. High water tables will prevail in this area from reservoir bank storage. There should, therefore, be enough shallow ground water to support willows or other riparian vegetation if it can be established and if it is more desirable than grasses.

The recommended alternative (B) will include a side channel spillway across the left abutment and an emergency spillway near the borrow area. The side channel spillway will be mostly cut in rock except for the lower chute which would be in till with 25 to 50 percent boulders. It is our judgement that the lower chute and plunge pool can be left unlined without excessive erosion and sedimentation. However, this is difficult to predict since the depth to bedrock is unknown and till composition is based on projections of data from the left abutment of the dam. The bouldery nature of the till should rapidly stabilize erosion and develop a natural riprapping effect. This can be enhanced by placement of the boulders in the chute during excavation of the spillway. Also, it may be feasible to follow more closely the bedrock surface if it is within practical reach in terms of depth. Finally, if erosion and sedimentation is perceived to be a problem, the plunge pool can be enlarged to act as a sedimentation basin with boulders placed downstream to stabilize the overflow channel. Sedimentation controls will be needed during construction and a larger plunge pool will be considered in conjunction with construction requirements during final design.

The emergency spillway will be configured to discharge the rare spills to Shell Creek below Shell Reservoir. The frequency of significant emergency spills would be extremely small, probably about once in 1,000 years or more. The emergency spillway would not spill at all until flood outflows reach about 1500 cfs. A PMF would cause considerable erosion, however, shallow bouldery till and bedrock would limit downcutting of a new channel. This damage would probably be small in comparision to overall watershed damage during a flood event of this size, regardless of the presence of the reservoir and spillway configuration.

Seepage beneath the proposed new dam will not be cut off. This will result in increased seepage return flows to Adelaide Creek downstream from the dam which should be beneficial to fish habitat, especially during the fall, winter and spring seasons. Seepage analyses indicate these releases will amount to about 100 to 200 acre-feet per year. These releases will supplement minimum flow releases from the reservoir.

The Buckley Creek Diversion is designed to operate in a manner similar to historic operations. When flows in Buckley Creek are 1.3 cfs or less, all water will be bypassed. Conversely, with the diversion gate open, no water will be diverted until flows exceed 1.3 cfs. The structure will divert a maximum of 50 cfs and bypass 2 cfs. Bypassed flows will thus vary from 1.3 cfs or less during minimum flows, to 2 cfs or more (when flows exceed 52 cfs). In addition to the designed bypass, there is natural underflow through the steep alluvial cone that will be left undisturbed. Much of this underflow apparently surfaces as seeps at the toe of the slope at the head of a small wet meadow about 400 feet below the diversion site.



NOTE:

CONFIGURATION	DAM CREST ELEVATION (feet)	SPILLWAY CREST ELEVATION (feet)
EXISTING DAM	9259	9254.8
ALTERNATIVE A	9280	9273
ALTERNATIVES B AND C	9287	9280

1 acre-foot = 43,560 ft.^{3} = 325,800 gallons

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY

AREA - CAPACITY CURVES

Checked by D. J. Hamby	Date 10-22-85	Project No.	Figure No.
Approved by R. R. Volpe	Date 10/22/85	3235	II-1

III. GEOTECHNICAL CONDITIONS

A. PHYSICAL SETTING

1. Landforms

Lake Adelaide is located on the upper western slopes of the Big Horn Mountains within the upper part of the Shell Creek Watershed, on Adelaide Creek. The watershed behind Lake Adelaide, including the Buckley Creek drainage, extends to the crest of the range which reaches 11,000 feet at Elk Peak.

Present landforms are largely a result of extensive glaciation of the large mass of uplifted rock that forms the mountains. Except for the spine of the range, the side slopes are relatively subdued, cut by shallow U-shaped glacial valleys. Many lakes and relict lakes, now swamps and meadows, occupy much of the landscape. Westward flowing streams draining the watershed are collected by Shell Creek which is more deeply incised and plunges off the Western flank of the mountains through the rugged Shell Canyon.

2. Geology

The Big Horn uplift was formed by Larimide thrust faulting and subsidiary normal faults resulting in the uplift of granitic basement rocks to form the core of the mountains. The uplift is asymetrical with major thrust faults near the eastern base of the mountains and with the western flank formed by a faulted homoclinal structure. The Mesozoic and Paleozoic sedimentary rocks that once covered the granitic basement rocks have been stripped by erosion to expose the pre-Cambrian granitic core.

At present, the entire Adelaide and Buckley Creek drainages are underlain by granitic rocks and till, which is detritus left by glaciation. The pre-Cambrian core of the mountains is reported to be largely quartz monzonite and quartz diorite, but granite and gneiss have been identified (megascopically) in the project area. In detail, there are probably other crystalline rock types present. However, differentiation of these rock types has no engineering significance at the project site. In the vicinity of Lake Adelaide, the granitic rocks are generally covered by glacial till. Peaks, knolls and small topographic knobs are often granitic outcrops. Whereas, the more uniform slopes, rounded ridges and valleys are usually underlain by till of variable thickness, up to about 50 feet at the dam site.

The quality of the granitic rocks is controlled by geologic structure and weathering. Weathering is generally rather superficial, but there are decomposed granite zones up to 10 to 20 feet thick. Decomposition is typically erratic, but it is often more prevalent beneath the more heavily timbered areas and is common beneath till deposits, although not always present.

The granitic rocks are generally jointed throughout the area with widely spaced fractures (4 to 20 feet). These fractures, along with stress relief and freeze-thaw cycles, account for boulder fields that are fairly common to the area, including the right abutment of the dam site. There are however, fracture zones where jointing is moderate. These zones produce some poorly distinct linear features that can be seen on aerial photos. In most cases, these fracture zones are covered by glacial till adjacent to the reservoir.

No faults have been reported or identified during exploration at the dam and reservoir site, nor at the Buckley Creek diversion site. A swarm of small faults are reported in a zone east and southeast of Lake Adelaide. The closest of these faults are 2 to 3 miles to the east in a zone along the crest of the mountains. These faults are probably normal faults that generally trend north, northwest and northeast.

A moderate sized normal fault is reported about 4-5 miles southwest of Lake Adelaide. It may extend northward along Crooked Creek to within 2 to 3 miles of the lake.

The closest major fault reported is the thrust fault along the eastern base of the Big Horn Mountains. This fault is about 24 miles east-northeast of the reservoir and dips toward the lake. Essentially, the project site is within the hanging wall block of this major structure. None of the faults discussed above are reported to be active. However, a moderate sized earthquake can occur in the project vicinity as discussed later in this section under Seismicity.

Investigations have not revealed any geologic conditions that will constitute a "fatal flaw" for the enlargement of Lake Adelaide. Large boulders in the till and residual boulders in the right abutment area along with remoteness of the site and the short construction season, will result in difficult and relatively expensive dam construction.

B. FOUNDATION CONDITIONS

1. General

Lake Adelaide occupies a broad, rolling glacial bench, perched on the ridge north of Shell Creek and Shell Reservoir. This bench contained the original Lake Adelaide which was a small glacial lake drained by Adelaide Creek. The natural lake was created by a dam of glacial till or perhaps a small moraine. In 1915, the present dam was constructed by borrowing till and placing it on top of the natural dam. In this manner the natural lake was raised about 25 feet. Level II drill holes indicate that the fill was poorly compacted and organic materials were left in the foundation. Further, the right side of the embankment was bent upstream to avoid a boulder field that covers the right abutment ridge. Because of the low density condition of the existing embankment, it was decided for this study that an entirely new dam should be constructed immediately downstream from the existing dam. Further, field exploration indicates that the boulder field is underlain by competent granite bedrock which makes it attractive to eliminate the upstream bend in the embankment by tying it directly into the right abutment ridge. This configuration decreases the length and volume of the new dam considerably. Most of the old dam will be used for borrow materials to top out the new embankment and the remainder will be left as a blanket to help control seepage. Figure III-1 shows the footprint of the proposed new dam (Alternative B) along with surface geology and locations of exploration work.

Several downstream configurations were examined initially. These were explored at a preliminary level with several refraction seismic profiles as shown on Figure III-1. Seismic data were in turn used to position new drill holes and trenches to supplement the six holes drilled in 1984. The primary purpose of the new drill holes was to define conditions in the channel section and right abutment areas of the foundation. All test holes were completed with casing to act as observation wells to measure ground-water levels. One well, DH-7, was completed through the till to bedrock with 4-inch diameter casing and used for multiple-well pumping tests to determine hydraulic conductivities within the till foundation. Casing installed in the other three test holes was 2 inches in diameter. The four new holes ranged in depth from 29 to 48.8 feet with granitic bedrock encountered at depths of 8.5 to 40 feet. Test holes DH-8, DH-9, and DH-10 were cored well into bedrock and packer tests were performed to determine the fracture dominated hydraulic conductivities of the granitic bedrock.

In addition to these test holes, six backhoe test pits were excavated in the channel section, left abutment and the potential service spillway area above the left abutment. These test pits were excavated to depths of 6.0 to 9.0 feet. They were used to observe the composition of the till in place and to obtain bulk samples for laboratory testing. Figure III-1 shows the location of test holes and test pits in reference to the plan layout of dam Alternative B. Detailed logs along with results of the seismic refraction survey are included in Appendix A and C, respectively. Further, the exploration data were used to develop sections A-A' and B-B' shown on Figure III-2.

In general, the proposed dam foundation occupies a narrow, but relatively shallow valley. The valley is partially filled with up to 50 feet of glacial till and reworked till over granitic bedrock. A prominant boulder field dominates the site, forming most of the right abutment ridge. The boulders are derived from the underlying bedrock and are a result of weathering in-place and stress relief along fractures. Glacial till covers the rest of the foundation area except for one small outcrop at the base of the right abutment. In section, the till deposits are asymetric with a maximum depth of 48 feet near the left side of the channel section and thinning to 0 feet at the aforementioned outcrop on the right side. The till covers the left abutment area, but gradually thins to a few feet near the dam crest elevation. Outcrops of granitic rock occur at or above 9300 feet in elevation in the alternate service spillway area above the left abutment.

2. Right Abutment

The slope forming the right abutment area consists of a boulder field underlain by granitic bedrock. The boulder field consists of residual granitic boulders up to 15 feet or more in diameter with little or no soil in the matrix except near the toe of the slope. Consequently, there is little to no vegetative cover. The boulders are loose, but are essentially weathered mechanically in place with little transport down slope. Apparently the boulder field is the result of stress relief along the joints, perhaps by unloading with the melt of the Pleiotocene ice cap(s), combined with yearly freeze-thaw cycles. Along the axis of the dam there is a small outcrop at the base of the slope with boulders reaching a thickness of 20-25 feet thick near mid-slope (near DH-10), and thinning to outcrops near the top of the slope near elevation 9470. The boulder field and underlying bedrock is shown schematically on Figure III-3.

The boulders are angular and joint patterns can still be seen in portions of the deposit, indicating little gravity transport. Large voids occur between boulders indicating the permeability is extremely high. Foundation preparation will require their removal to sound bedrock beneath the right abutment section of the dam. The boulders are durable and will make excellent rock fill when reduced to usable sizes.

Underlying the boulders is very competent, granitic bedrock. This rock is essentially unweathered with only a little iron staining; no decomposed zones were detected in the field. Joints are spaced at about 5 to 15 or more feet which accounts for the large size of the overlying residual boulders. These joints are generally tight resulting in relatively low hydraulic conductivities of 1 to 40 feet per year in DH-10. At the base of the right abutment slope and beneath the right side of the valley section, there is a zone of more intensely fractured granitic rocks. This zone was encountered in DH-9 and probably extends upstream to DH-5. This zone is rather narrow, probably not over 50 feet wide, but the hydraulic conductivity while still relatively low, is highest for the bedrock at the dam site, ranging from 100 to 300 feet per year. The fracture zone classifies as moderately fractured, with spacings of 0.2 to 0.5 foot. However, permeability tests indicate that fractures become much tighter with depth. Seepage within this bedrock zone will not affect the stability of the dam and no grouting of the bedrock is proposed.

3. Channel Section

The channel section of the foundation occupies the valley bottom below the steeper side slopes that form the abutments. This area along the axis of the proposed new dam is below elevation 9235 feet. Generally, the alluvial valley of Adelaide Creek has the typical U-shape of a glaciated canyon. In micro-relief however, the channel section is irregular in shape because of three abandoned and present stream channels. It is unclear which channel was active prior to the construction of the present Adelaide Dam, but the present channel near the left abutment, receiving the discharge from the outlet works, appears to be a subsidiary channel (artificial or natural). The more deeply incised and widest channel is located on the right central portion of the channel section. The third incised channel is located on the right side of the channel section and receives the discharge of the present spillway and may be partially or entirely artificial. The valley bottom has a sparse to moderate cover of trees and brush.

The channel section is underlain by glacial till that has been slightly reworked surficially along the stream channels within the valley bottom. The depth of till encountered in bore holes varies from 8.5 feet (at DH-9) on the right side of the area to maximums of 40 to 48 feet in the center and left side of the channel section. The sections shown on Figure III-2 indicate the depth of till overlying bedrock. The till consists of a heterogeneous mixture of boulders, gravel, sand and silt that is relatively dense. Trenches into the upper part of the till indicate little stratification. Largely non-plastic fines range from 3 to 32 percent. Cobbles to large boulders comprise about 40 to 50 percent of the deposits, the rest is mostly sand sizes. Large boulders range up to 15 to 20 feet in diameter, but most are 2 to 4 feet in diameter. Within drill hole DH-8, a nest of large boulders was encountered from 18 to 40 feet. The lower 10 to 15 feet of these boulders appear to be residual in nature suggesting rubblization more or less in place. Their origin may be similar to the boulder field on the right abutment except that the matrix is filled with sand and silt with some gravel.

Multiple well pumping tests were conducted in the central channel section foundation area as described in detail in Section III-E. Results of these tests indicate that hydraulic conductivities of the till range from 300 to 1400 feet per year. For design purposes an average hydraulic conductivity of 1200 feet per year was selected which is considered conservative. Geologic data and single hole tests in DH-1 and DH-3 indicate that the upper half of the till is generally more permeable. Also, multiple well pumping tests indicate that maximum transmissivities occur through the left portion of the channel section in the vicinity of DH-1. While the till foundation generally has a low hydraulic conductivity, there may be buried channels of more transmissive gravels, but their extent should be small.

The granitic bedrock beneath the till is generally massive with widely spaced joints. The joints are relatively tight with hydraulic conductivities in the range of 1 to 300 feet per year with the fracture permeability decreasing with depth in all cases. Drill Hole DH-9, located near the right side of the channel section, encountered moderately fractured rocks with hydraulic conductivities ranging from 100 to 300 feet per year, again decreasing with depth. This more permeable bedrock zone will partially offset the lower transmissivities of the till on the right side of the channel section.

4. Left Abutment

The left abutment area is formed by rather uniform side slopes that are slightly concave; steepening slightly near the new dam crest. Above about elevation 9315 the side slope of the left abutment ridge becomes more gentle as shown on Figure III-2, Section A-A. The abutment area has a moderate tree cover of young to mature conifers except for a narrow opening along an access trail to the present discharge of the outlet works.

The left abutment area is underlain by a cover of glacial till combined with decomposed granite that is 40 feet thick at the base of the abutment (DH-3) and gradually thins to 0 at about elevation 9305 feet. DH-2, located at a surface elevation of 9274 feet, encountered 18 feet of till above bedrock. The glacial till generally consists of well graded, gravelly silty sand. The gravel ranges up to large boulders. Organic top soils are very thin (± 6 inches), but the root zone of the timber covering the area probably extends 1-2 feet in depth. The till probably contains some slope wash or reworked till. Near the base of the deposit, the till grades to decomposed granite, a silty sand. The extent and thickness of decomposed granite is variable ranging from a possible 15 feet thick at DH-3 to approximately 1 foot thick at TP-1. Decomposed granite is typically erratic in extent and thickness and there are probably zones where it is absent. The hydraulic conductivity of the till and decomposed granite is probably similar to or lower than the till in the channel section. This forested slope is more conducive to chemical weathering and results in more clay and silt fines in the till and decomposed granite.

Massive granite bedrock underlies the till and decomposed granite. The granitic rock has widely spaced joints at about 4 feet spacings. The permeability of the bedrock will be controlled by these fractures which tend to be tighter with depth. Tests in bore holes DH-2 and DH-3 indicate hydraulic conductivities ranging from about 1 to 33 feet per year. This is about two orders of magnitude lower than the overlying till.

5. Left Abutment Spillway Area

As an alternative to a glory hole service spillway, an uncontrolled ogee section or a side channel configuration were studied. These spillway alternatives would be located around the left end of the embankment in a cut through the left abutment ridge. Above elevation 9305 feet, the spillway cut will be entirely in massive granitic rock. Below this elevation, there will be a cover of glacial till and decomposed granite up to 10 vertical feet deep at the end of the new dam. The weir structure and approaches would be founded on bedrock. The alternative spillway chute would be excavated partially in rock and partially in glacial till. The bouldery nature of the glacial till will prevent excessive erosion of the portion of the channel not in bedrock which will eventually have an appearance similar to the existing spillway channel. The area along the spillway axis has a moderate tree cover of conifers.

Excavation of the granitic bedrock will require blasting. Presplitting will be required to develop uniform side slopes for the spillway channel. The granitic bedrock will stand on vertical to

1/2:1 slopes. Common excavation methods can be used to strip the till. However, occasional large boulders may be encountered up to 20 feet in diameter that will require secondary blasting for excavation.

6. Auxiliary Spillway Area

If the dam is raised to a crest elevation of 9287, the option of an auxiliary spillway in the borrow area as proposed in the Level II study becomes much more feasible. Bedrock elevations of up to 9280 occur in the spillway as shown on Section C-C', Figure III-4. Thus, it appears that relatively expensive rock excavation can be avoided if the dam crest is at elevation 9287. In this case, the auxiliary spillway would be cut in till and decomposed granite and the excavated materials used as borrow for dam construction. In other words, auxiliary spillway construction would be a matter of shaping the south part of the borrow area. The shallow bedrock surface will limit downward erosion during emergency spills. The location of the auxiliary spillway, along with a more detailed discussion of spillway requirements at the site, is presented in Section VI.A.

C. CONSTRUCTION MATERIALS

The proposed source of materials for constructing the new embankment is shown by the area designated as "Borrow Area Limits" in Figure III-4. This area encompasses the original borrow site and the location for the proposed auxiliary spillway. A total of 10 exploratory trenches and two drill holes were excavated during this investigation to provide samples of the borrow materials, and define the extend and geometry of the borrow and spillway area. In addition, a seimsic survey was conducted across the area, as designated by lines SR 7-1 through SR 7-4 on Figure III-4. The information obtained by the trenches, drill holes and seismic survey, in conjunction with the Level II study site investigation, was used to develop three cross sections, shown in Figures III-5 and III-6.

Cross Section C-C' (Figure III-5) shows that bedrock extends up to approximately 9280 feet along a line approximately parallel with the proposed auxiliary spillway. Thus the layout of the borrow area was selected partially on the basis of forming an approach apron, or bay in the vicinity of the emergency spillway entrance. At high water level equal to 9280 feet elevation, most of the ground disturbed in the borrow area during construction will be submerged.

Throughout the borrow area, as shown by sections C-C′ (Figure III-5), and D-D′ and E-E′ (Figure III-6), the material above bedrock consists of two visibly distinct layers. The surficial layer, below about 6 inches to 1 foot of topsoil, consists of silty sand till. This material will be the source for the earthfill in the upstream portion of the new embankment. Most of the silty sand till material is non-plastic, but some low plasticity materials were observed near the reservoir (e.g. near T-8, T-15 and T-16). This more plastic silty, clayey material will be used in the embankment at the left and right abutment contacts. Beneath approximately 5 to 20 feet of till, is a variable zone of arkosic, gravelly sand designated for convenience in this report as "decomposed granite". Much of this material has probably been reworked and is not a true decomposed granite. This material varies from clean coarse sand to silty, well graded sand. It is intended that the decomposed granite material be used in the filter zones of the embankment and possibly as a more pervious downstream embankment material.

Both the till and the decomposed granite materials are gravelly. Cobbles and boulders up to 2 to 3 feet occur throughout the borrow area, but they are more prevalent in the upper till layer. Based on visual observations during the trench excavations, it was estimated that boulders and cobbles over 6 inches comprise approximately 25 percent of the borrow volume. Since it is intended to limit the maximum particle size for the primary earthfill zone to 6 inches, this material will have to be processed to separate the oversize rocks from the primary embankment earth fill. The Alternative B design configuration discussed in Section V provides the contractor with the option of using the oversize borrow material in a rock fill section, whereas the other two design alternatives, A and C (see Section V), use only the right abutment rock source for a mandatory rock toe, and waste the oversize borrow materials.

All materials above the granitic bedrock in the borrow area are easily excavated with conventional earth moving equipment. The residual "decomposed granite" material can be readily distinguished from the till material in the field, facilitating the borrow area development. Due to a relatively high water table level in the borrow area northeast of the existing access road (on the reservoir side), the borrow area will probably be developed from the low end to the high end (northeast to southwest) to allow drainage.

D. RESERVOIR RIM STABILITY

The rim of the present and enlarged Lake Adelaide reservoir is exceptionally stable. The glacial till and the granitic bedrock are both very resistant materials against slope failure. During geologic mapping of the reservoir rim, only one small area of possible slope instability was mapped. This area in glacial till and located on the northeastern reservoir rim, is less than one half acre in extent. No conditions were identified that could threaten the operation or integrety of the reservoir. Wave erosion of the till could locally result in low oversteepened slopes, particularly on the southeastern shore. Although none are contemplated to our knowledge, facilities in general should be set back from this high water line to preclude damage by under cutting or small slip failures.

E. SEEPAGE CONDITIONS

1. Exploration and Field Testing

The Level II studies reported an extremely wide range of hydraulic conductivities for the till deposits. These hydraulic conductivities ranged over 3.5 orders of magnitude or from less than 100 feet per year to 55,000 feet per year. Because of this wide range, a multiple well pumping test was conducted to evaluate the permeability of the till in the maximum section area of the foundation. This testing procedure avoids the errors that are difficult to overcome with the USBR Designation E-18 procedures in unconsolidated materials. The multiple well pumping test provides a better definition of average hydraulic conductivity or transmissivity at observation wells, remote from the pumped well, and avoids problems such as formation plugging, formation disturbances and short circuiting around packers or casing that are pervasive problems with single hole tests.

Two multiple well pumping tests were performed in the foundation area. Constant discharge tests were run using DH-7 as the pumped well and DH-8 and DH-1 as observation wells. The pumping cone of depression from which water-level responses were measured, covered at least one third of the channel section of the foundation area. All of the wells fully penetrated the till. Also, the highest hydraulic conductivities were reported from tests run on the DH-1 observation well during the Level II study.

A third multiple well pumping test was performed in the area of the auxiliary spillway (and borrow area). DH-11 was used as the pumped well and DH-12 was used to measure water-level responses. Both wells fully penetrated the till and decomposed granite aquifer. Table III-1 is a summary of the results of the three pumping tests.

The primary method of interpretation of the three pumping tests was to match logrithmic plots of field data (drawdowns versus time over the radial distances to the observation well squared) with families of type curves. Unconfined type curves were used along with a family of type curves for well storage effects. Observation well DH-8 indicated some well storage effects and a combination of type curves was used. At both test sites, the aquifer is unconfined and in all cases water-level responses deviated from the artesian type curve due to delayed drainage effects. The matching curve or Theis method of interpretation provides the most reliable results because the field data plots were in the region where u (the Boltzman constant) is greater than 0.01. The Jacob method of analysis was used as a check using both drawdown and recovery data. However, this method is subject to fairly large errors where u is greater than 0.01. This accounts for the consistently higher transmissivity and hydraulic conductivity values for this method shown on Table III-1. Field data plots and interpretations are presented in Appendix A.

The hydraulic conductivity selected for seepage analysis through the till at the foundation of the dam was 1200 feet per year. This value is in the upper part of the range using the Theis or matching curve method of interpretation and is considered to be conservative or on the high side. At DH-1 the hydraulic conductivity may be slightly higher, but at DH-8, data indicates the hydraulic conductivity is considerably lower than 1200 feet per year.

At the auxiliary spillway site, the hydraulic conductivity selected for seepage analysis is 6000 feet per year, based on the Theis interpretation. DH-12 penetrated a considerable amount of low

TABLE III-1

SUMMARY OF PUMPING TEST ANALYSIS

Test No.	Q gpm	Observ. Well No.	Saturated Thickness	JACOB ANALYSIS METHOD: Drawdown Per Log Cycle ft	JACOB ANALYSIS METHOD: Transmissivity T sq ft/yr	JACOB ANALYSIS METHOD: Permeability K ft/yr (cm/s)	THEIS ANALYSIS METHOD: Match Point W(u)	THEIS ANALYSIS METHOD: Match Point S ft	THEIS ANALYSIS METHOD: Transmissivity T sq ft/yr	THEIS ANALYSIS METHOD: Permeability K ft/yr (cm/s)
1 Pump	1.2	DH-1	30	0.32	4.83×10^{4}	1610 (1.6×10^{-3})	10	1.6	4.20×10^{4}	1400 (1.4×10^{-3})
		DH-8	36	0.36	4.29×10^{4}	1190 (1.2×10^{-3})	1	0.2	3.36×10^{4}	930 (9.0×10^{-4})
1 Recovery	1.2	DH-1	30	0.37	4.18×10^{4}	1390 (1.1×10^{-3})				
		DH-8	36	0.38	4.06×10^{4} (1.1×10^{-3})	1130				
2 Pump	3.0	DH-1	30	0.86	4.49×10^{4}	1500 (1.5×10^{-3})	1	0.49	3.43×10^{4}	1140 (1.1×10^{-3})
		DH-8	36	0.86	4.49×10^{4}	1250 (1.2×10^{-3})	1	1.50	1.12×10^{4}	311 (3.0×10^{-4})
2 Recovery	3.0	DH-1	30	0.80	4.83×10^{4}	1610 (1.6×10^{-3})				
		DH-8	36	0.75	5.15×10^{4}	1430 (1.4×10^{-3})				
3 Pump	3.0	DH-12	15	0.27	1.43×10^{5}	9540 (9.2 x 10-3)	1	0.20	8.39×10^{4}	5590 (5.4 x 10-3)
3 Recovery	3.0	DH-12	15	0.24	1.61×10^{5}	10720 (1.0 x 10-2)				

density silty sand believed to be decomposed granite beneath a layer of denser till. The low density sand accounts for the significantly higher hydraulic conductivity at this location.

Table III-2 shows the results of packer tests performed in DH-8, DH-9, and DH-10 after granitic bedrock was penetrated. Hydraulic conductivities in the granitic rocks are controlled by fractures. The hydraulic conductivities vary from 0 to about 300 feet per year which generally agrees with the bedrock tests performed during the Level II study. Level II drill holes DH-5 and DH-6 reported higher bedrock hydraulic conductivities than encountered in the present study, but they are outside the proposed foundation area (see Figure III-1). These two holes probably penetrated the same fracture zone as penetrated in DH-9 near the base of the right abutment. Packer tests indicate that the higher hydraulic conductivities are limited to the upper 10 to 15 feet of bedrock where fractures are relatively open. As is common, the fractures become significantly tighter with depth. Field data from the packer tests are presented in Appendix A.

2. Foundation Area

The most transmissive material in the foundation of the dam are the till deposits. These deposits are thickest (40 to 50 feet) beneath the central and left channel section, and exist as shallow cover on the left abutment. Beneath the right side of the channel section, the till thins to 8.5 feet at DH-9. As a result, seepage will be concentrated more toward the left side of the foundation area. Seepage through the granitic rocks beneath the till will be minor in comparison because the hydraulic conductivity is generally 1 to 3 orders of magnitude lower even in the upper 10 feet of the bedrock, and for practical purposes is negligable in terms of reservoir losses. An exception is the zone of moderately fractured rocks in the vicinity of DH-9. Seepage through the upper 10 to 15 feet along this fracture zone will partially offset the lower transmissivity in the till near the base of the right abutment.

A finite element seepage model was developed to analyse seepage through the embankment and foundation. The results from these seepage analyses indicate reservoir losses through the foundation would not be a critical factor and that a full or partially penetrating cutoff, as proposed in the Level II study, would not be required. The primary

TABLE III-2

SUMMARY OF BEDROCK PERMEABILITY
(PACKER TEST) ANALYSIS

Drill Hole	Location	Depth Interval (ft)	Flow Test Pressure (psi)	Flow Test Qave (gpm)	Flow Test K (ft/yr)	Holding Test T (sq ft/yr)	Holding Test K (ft/yr)
DH-8	Near centerline of proposed dam, approx. mid valley.	42.0-47.0	10 to 40	0	0	0	0
		47.0-57.0	30	0.02	2	0	0
			50	0.02	1	0	0
DH-9	Base of right abutment/rock pile.	11.0-19.0	10	1.06	224	1950	243
			20	2.43	281	2086	260
		19.0-29.0	20	1.11	106	635	64
			30	1.42	94	609	61
DH-10	On right abutment rock pile.	23.0-36.0	10	0.13	13	0	0
			20	0.53	34	509	40
			30	0.60	29	288	20
		35.8-48.8	30	0.03	1	0	0
			40	0.05	2	0	0
			50	0.08	3	0	0

purpose of the seepage model was to analyse pore water pressures to support stability analyses and embankment design. These analyses and results are described in detail in Section V. They indicate that the maximum seepage through the embankment and foundation under full reservoir head was at a rate of up to 250 acre-feet per year. However, full reservoir heads will only exist for a short time at the end of the runoff season before the reservoir is drawn down. Also, this model was developed along the maximum section where losses are greatest and the rate was applied along the full centerline length. In light of these conservative assumptions, actual losses will probably be closer to 100 acre-feet per year.

Seepage through the foundation will remain essentially uncontrolled except that the portion of the old dam to be left in place will act as a partial upstream blanket. Also, filters and drains will be provided to control migration of fines and pore pressures in the lower portion of the embankment. Even if seepage losses are higher than expected, they will be greatest when the reservoir is full and during normal reservoir releases. Most of the seepage should appear as surface flow within about 1500 feet downstream from the dam. These extra flows, if of any significance, should be credited to reservoir operation as fish or normal releases depending on the season. Some seasonal monitoring will be required to quantify these releases at a reasonable distance downstream. The seepage analyses are not sensitive enough to provide this information since they are based on a steady state condition, whereas, actual seepage will be transient or variable.

3. <u>Reservoir Rim Seepage</u>

Under present conditions, ground-water flow is mostly toward Lake Adelaide in surficial materials except at the dam. The ground-water surface closely parallels topography and is restricted by the underlying bedrock surface. There is undoubtedly some deeper flow southward from Lake Adelaide to Shell Reservoir through fractures in the granitic rocks. However, the magnitude of this flow is minor and will not increase significantly with the enlargement of Lake Adelaide. The lake is situated in a bedrock bowl closed on all sides except at the dam. Therefore, there is no potential for significant reservoir seepage until the reservoir is raised enough to spill over the bedrock surface, through the till and decomposed granite deposits.

Due south of the upper end of Lake Adelaide there is a low divide between Adelaide Creek and the Shell Reservoir Watershed. A small unnamed lake occupies a depression in this divide. This lake appears to be fed by ground water from upper Adelaide Creek, and in turn, there appears to be ground-water flow from the lake southwest to a small bowl shaped depression that is swampy or wet from ground water discharges. This bowl is the head of a small canyon that drains into Shell Reservoir. In this manner, perhaps a few tens of gallons per minute is lost from upper Adelaide Creek (and diversions from Buckley Creek) to Shell Reservoir. It is possible that seepage losses could be increased in this area by the enlargement of Lake Adelaide depending on the buried, but unknown configuration of the bedrock surface. In our judgement, however, the increased losses over natural conditions should not be significant.

Geologic mapping and exploration in the borrow area/auxiliary spillway area, suggest that the buried granitic bedrock surface reaches close to an elevation of 9280 feet which will be the maximum operational reservoir pool elevation assuming the maximum increase in reservoir size is adopted. In this case, reservoir rim seepage will be essentially negligible for practical purposes. However, there is the possibility of a bedrock low undetected during the exploration program. The most likely location would be through the till ridge south from the upper end of the reservoir to the bowl shaped depression described earlier, that drains to Shell Reservoir. The other area is in the vicinity of the borrow area/auxiliary spillway. Exploration in this latter area indicates a bedrock low would have to follow a rather irregular path, either to the southeast along the present access road, or along the auxiliary spillway to the west.

Because of these unknowns, a hypothetical bedrock low was analysed for seepage. This bedrock low was assumed to be 800 to 1000 feet wide with a bedrock surface at elevation 9350 feet or 30 feet below the maximum operational reservoir level. A hydraulic gradient of 0.025 was estimated from the possible seepage paths and a hydraulic conductivity of 6000 feet per year was used based on the pumping test at the auxiliary spillway site. Using a Darcian seepage formula, the maximum seepage rate was estimated at 100 acre-feet per year. Since the reservoir would be full only about two months, the annual seepage loss would be 25 to 50 acre-feet per year. While substantial, even two bedrock lows combined with foundation seepage losses should not seriously effect reservoir operation or yield.

F. BUCKLEY CREEK DIVERSION

The diversion point of Buckley Creek is situated at the mouth of a small canyon at about elevation 9320 feet. The stream is flowing across a small alluvial cone of till and reworked till. The present stream flows on the northwest side of the cone and there is an abandoned channel on the southeast side. Further, the creek splits just above the present diversion so that approximately one half of the flow currently bypasses the diversion structure (August, 1985). The alluvial cone area has a moderate cover of trees (conifers) with many blow-downs. The split in the stream is mainly due to log jams and other vegetative debris.

The diversion canal is cut in till that contains about 20 percent non-plastic fines, 5 to 10 percent cobbles and boulders, and 65 to 70 percent fine gravel and sand. It classifies as a gravelly silty sand. The alluvial cone along Buckley Creek at the diversion is probably similar in composition, but probably has less fines because of reworking by the stream.

The stream course on the alluvial cone appears fairly stable on the right (NW) side except for small debris diversions. There is a low ridge in the middle of the cone that presently blocks access to the abandoned channel on the left (SE) side of the cone. Also, the mature forest cover helps the general stream stability except for local diversions from log jams. However, during a major flood event, the stream could easily develop a new channel. For this reason, it is recommended that the new diversion be a minimal structure, realizing that some maintenance and/or reconstruction will be required after major floods.

The alluvial cone material is probably more permeable than the till that covers most of the area. There is probably more underflow past the diversion point than normal for the area. This extra underflow should reappear about 400 to 500 feet downstream at the head of a small meadow. Therefore, it would be reasonable to account for the extra underflow in the downstream release requirements past the diversion. This could be done by measuring flows at the head of the small meadow before the confluence of a small tributary from the southeast.

G. ACCESS ROAD

The proposed new access road will be an improvement of the existing road from Shell Reservoir to Lake Adelaide. In anticipation of this possibility, geologic mapping was performed along this route. The primary reason this route was selected is that the route described in the Level II studies encroaches on the new boundary of the Cloud Peak Wilderness Area. An important advantage of following the existing road is that it will minimize new disturbance within the National Forest. The route will deviate from the existing road at the north end of Shell Dam to gain access to the first rock bench at a reasonable grade. Also, the improved road will require a new switch-back to gain the top of the ridge before entering the Lake Adelaide drainage basin.

The southern slope of the ridge has moderate to heavy timber cover. From the top of the ridge to Adelaide Dam, the route follows open alpine meadow except for sparse to moderate timber near the left abutment of the dam. On the south slopes of the ridge, some timber will have to be removed to widen the road and trees will have to be removed along the deviations from the existing road.

The route between Shell Reservoir and Lake Adelaide is underlain by a veneer of till which in turn is underlain by granitic rocks. The south slope of the ridge above (north) Shell Reservoir has many small knolls and points which are surface expressions of granitic rock outcrops (or areas with little soil cover). Till of variable depth occurs between these small topographic highs. Till thickness on the lower slopes of the ridge may vary from one or two feet up to about 10 feet. Composition of the till is similar to that described earlier; basically a bouldery gravelly silty sand. Also, as elsewhere, there are probably irregular zones of decomposed granite (silty sand) at the base of the till.

Above about elevation 9200 feet the remainder of the route is underlain by till. However, cuts may still encounter bedrock at a shallow depth.

Excavation of the granitic rock for cuts will require blasting. The till can generally be handled by common excavation. Large boulders however may be encountered that will require secondary blasting or avoidance by minor rerouting.

The till will provide excellent road base material when over sized rock is removed. Side slope cuts will probably provide most of the fill material required. Supplemental borrow can be obtained from the borrow source for the dam and from the old borrow source for Shell Dam. The Lake Adelaide borrow area will provide a flat or down hill haul to the road between Shell and Lake Adelaide.

The road from the base of Crooked Creek Hill to Shell Reservoir was not examined in detail. However, the route appears adequate in general with some improvements required. The improvements will be primarily to remove or cover with fill the many boulders and to widen some of the tighter switch-backs to accommodate heavy equipment. Geologic conditions are generally similar to those north of Shell Reservoir. Some small existing borrow pits were noted along the route. Also, there is a large deposit of till capping the ridge south of Shell Reservoir. This would provide an excellent source of borrow with down-hill hauls.

H. SEISMICITY

1. Regional Tectonic Setting

The project area is located in the Middle Rocky Mountain Geomorphic Province. The major tectonic features of this province developed during the Laramide orogeny, a period of intense mountain building, lasting from approximately 80 to 50 million years before the present. Major structural blocks such as the Big Horn Uplift, were differentially uplifted along major fault systems controled, in part, by pre-existing zones of weakness. The oldest Laramide features formed in response to east-west compression resulting in north-south trending faults and folds bounding mountain ranges. The final stage of Laramide tectonism resulted from north-south compression which formed prominant east-west uplifts, including the Owl Creek Mountains and the southern terminus of the Big Horn uplift.

2. Historic Seismicity

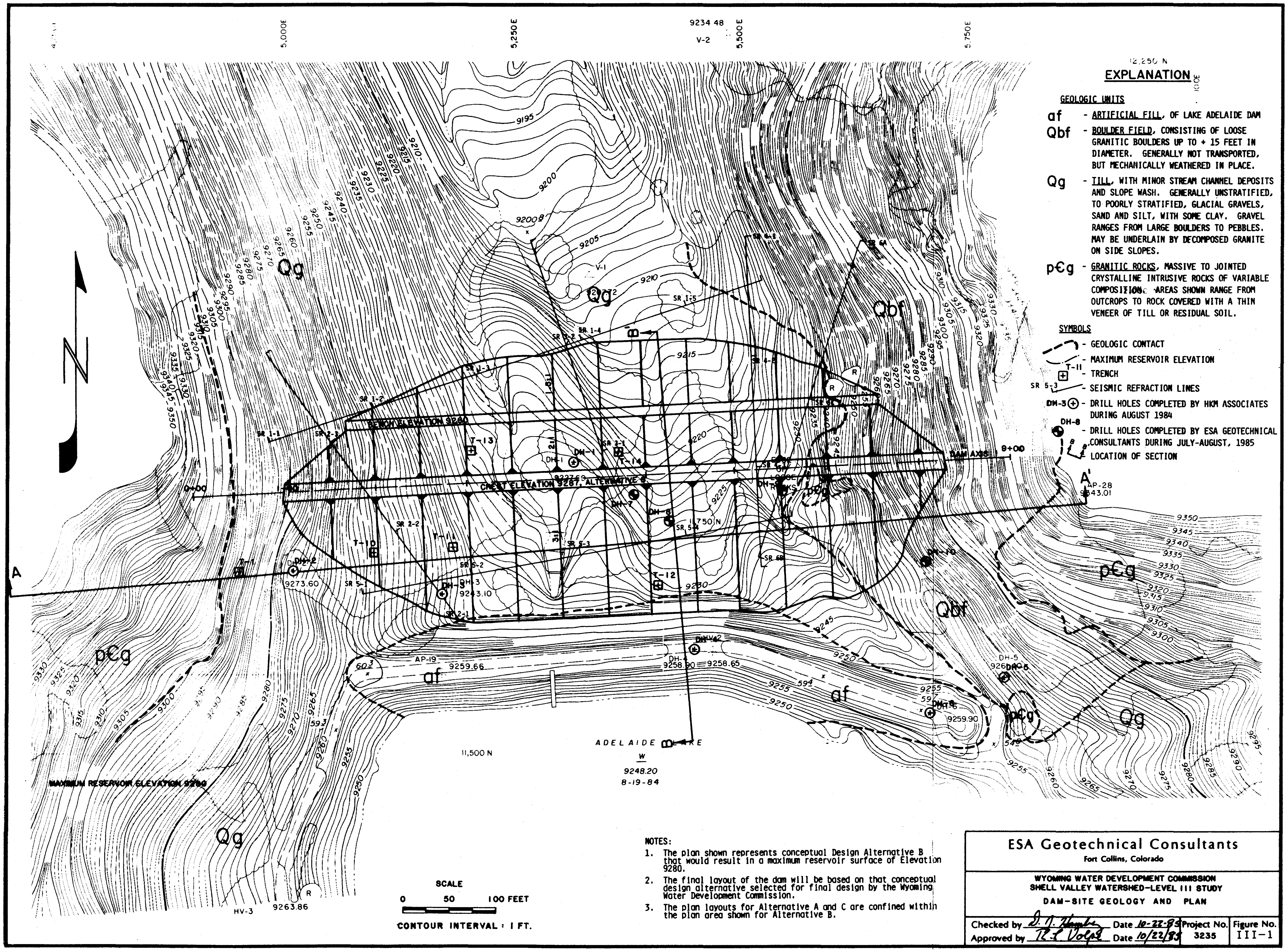
Wyoming is generally considered to be an area of low to moderate seismicity. The western edge of the state, however, borders the Intermountain Seismic Belt (ISB), an active zone of intraplate deformation characterized by relatively high strain rates, normal faulting, episodic large earthquakes and associated ground rupture. The closest part of the ISB to the project site is the Yellowstone Park area, about 150 miles to the west. Wyoming east of the ISB exhibits a rate of seismic activity about 1.5 times lower than the central part of the ISB in Utah. Locations of recorded earthquakes from 1915 through March, 1985 within 200 miles of the project site are shown on Figure III-7. The largest earthquake recorded within this radius was the August 18, 1959 Hebgen Earthquake of magnitude 7.1 with an epicenter about 180 miles west of the site.

Investigation of active and potentially active faults in Wyoming has been slow to develop outside the ISB. More detailed fault mapping may reveal more evidence of Holocene fault dispacement than was generally considered logical by past investigators. The perception of earlier geologists was based largely on the clearly indicated Laramide origin of the major tectonic features in the central and eastern parts of the state. However, in the last 10 years investigators have reported potentially active faults in the central part of the state, including a fault near Badwater at the south end of the Big Horn Mountains, about 80 miles south of Lake Adelaide. This fault may have experienced historic surface rupture (Witkind, 1975).

3. Design Earthquake

Standard procedures for developing a design earthquake from known seismic sources was considered to be unrealistic and very unconservative for this project. Because of the limited knowledge of active faults in Wyoming and the geologically short record, the concept of a "floating earthquake" was used for design purposes. It was considered reasonable to assume that a 5.0 magnitude event could occur under or in the near vicinity of the site considering the tectonic setting and the magnitude of recorded earthquakes in central Wyoming. It was assumed that the design earthquake would occur at a relatively shallow focal depth of 10 km.

Numerous researchers (Campbell, 1981; Joyner and Boore, 1981; and Bold and Abrahamson, 1982) have correlated peak horizontal bedrock acceleration as a function of distance from the seismic source. Based on these relationships, the expected peak horizontal bedrock acceleration at the site is between 0.08g and 0.13g due to a magnitude 5.0 earthquake occurring 10 km from the site. These values were further evaluated with the seismic risk map of the Contiguous U.S. by Algermissen, et al. (1982). This map indicates a peak horizontal acceleration in bedrock of 0.09g with 90 percent probability of not being exceeded in 250 years. Based on the above information, a peak horizontal bedrock acceleration of 0.15g would be a conservative design value. The influence of these maximum peak bedrock accelerations on the overall safety of the dam is discussed in more detail in Section V.E of this report.



EXPLANATION

GEOLOGIC UNITS

af - ARTIFICIAL FILL, OF LAKE ADELAIDE DAM

Qbf - BOULDER FIELD, CONSISTING OF LOOSE GRANITIC BOULDERS UP TO + 15 FEET IN DIAMETER. GENERALLY NOT TRANSPORTED, BUT MECHANICALLY WEATHERED IN PLACE.

Qg - TILL, WITH MINOR STREAM CHANNEL DEPOSITS AND SLOPE WASH. GENERALLY UNSTRATIFIED, TO POORLY STRATIFIED, GLACIAL GRAVELS, SAND AND SILT, WITH SOME CLAY. GRAVEL RANGES FROM LARGE BOULDERS TO PEBBLES. MAY BE UNDERLAIN BY DECOMPOSED GRANITE ON SIDE SLOPES.

pЄg - GRANITIC ROCKS, MASSIVE TO JOINTED CRYSTALLINE INTRUSIVE ROCKS OF VARIABLE COMPOSITION. AREAS SHOWN RANGE FROM OUTCROPS TO ROCK COVERED WITH A THIN VENEER OF TILL OR RESIDUAL SOIL.

SYMBOLS

- GEOLOGIC CONTACT
- MAXIMUM RESERVOIR ELEVATION
T-11 - TRENCH
SR 5-3 - SEISMIC REFRACTION LINES
DH-3 - DRILL HOLES COMPLETED BY HKM ASSOCIATES DURING AUGUST 1984
DH-8 - DRILL HOLES COMPLETED BY ESA GEOTECHNICAL CONSULTANTS DURING JULY-AUGUST, 1985
B B - LOCATION OF SECTION

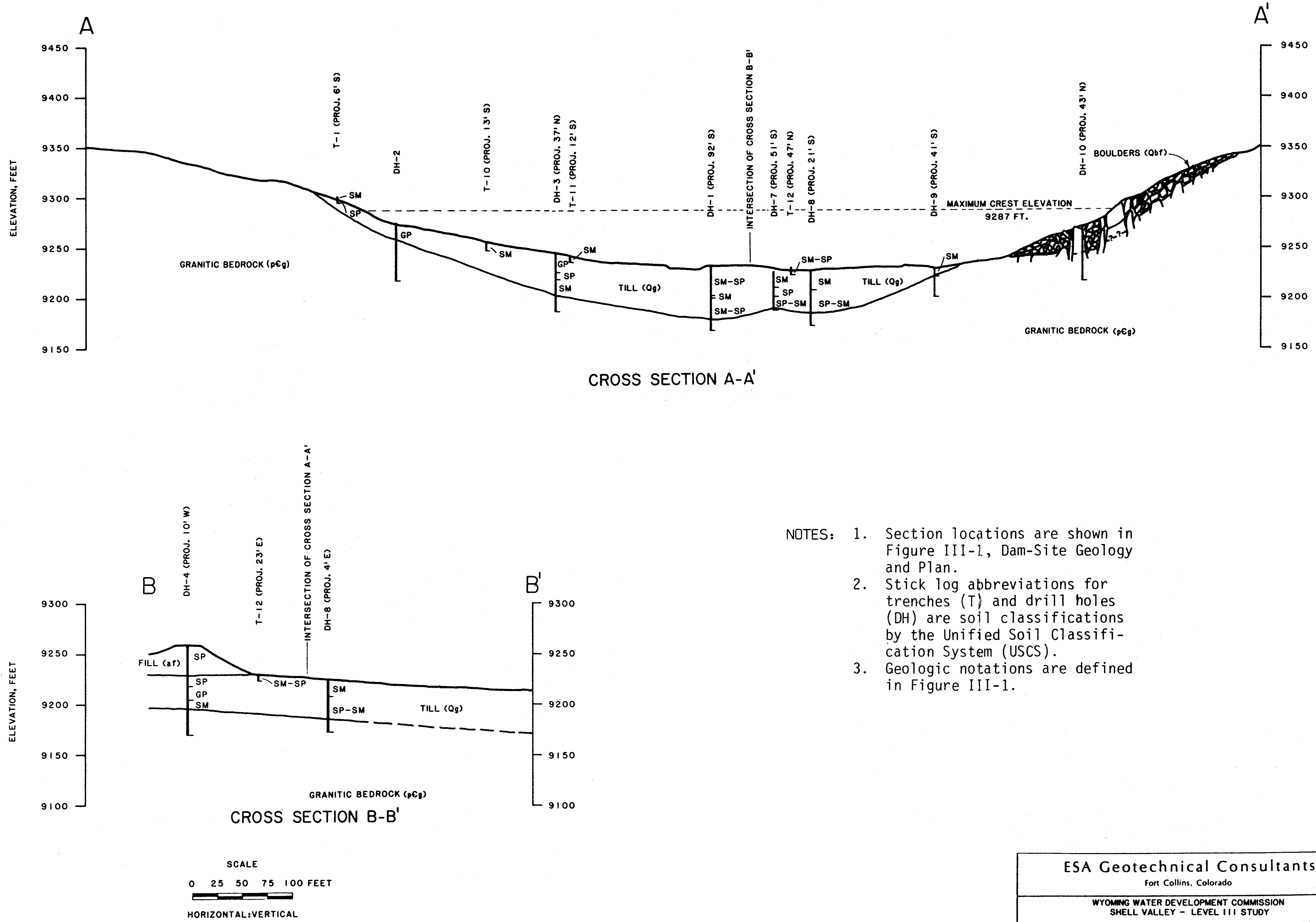
NOTES:

1. The plan shown represents conceptual Design Alternative B that would result in a maximum reservoir surface of Elevation 9280.
2. The final layout of the dam will be based on that conceptual design alternative selected for final design by the Wyoming Water Development Commission.
3. The plan layouts for Alternative A and C are confined within the plan area shown for Alternative B.

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
DAM-SITE GEOLOGY AND PLAN

Checked by D. J. Hamby	Date 10-22-85	Project No. 3235	Figure No. III-1
Approved by R. L. Volpe	Date 10/22/85		



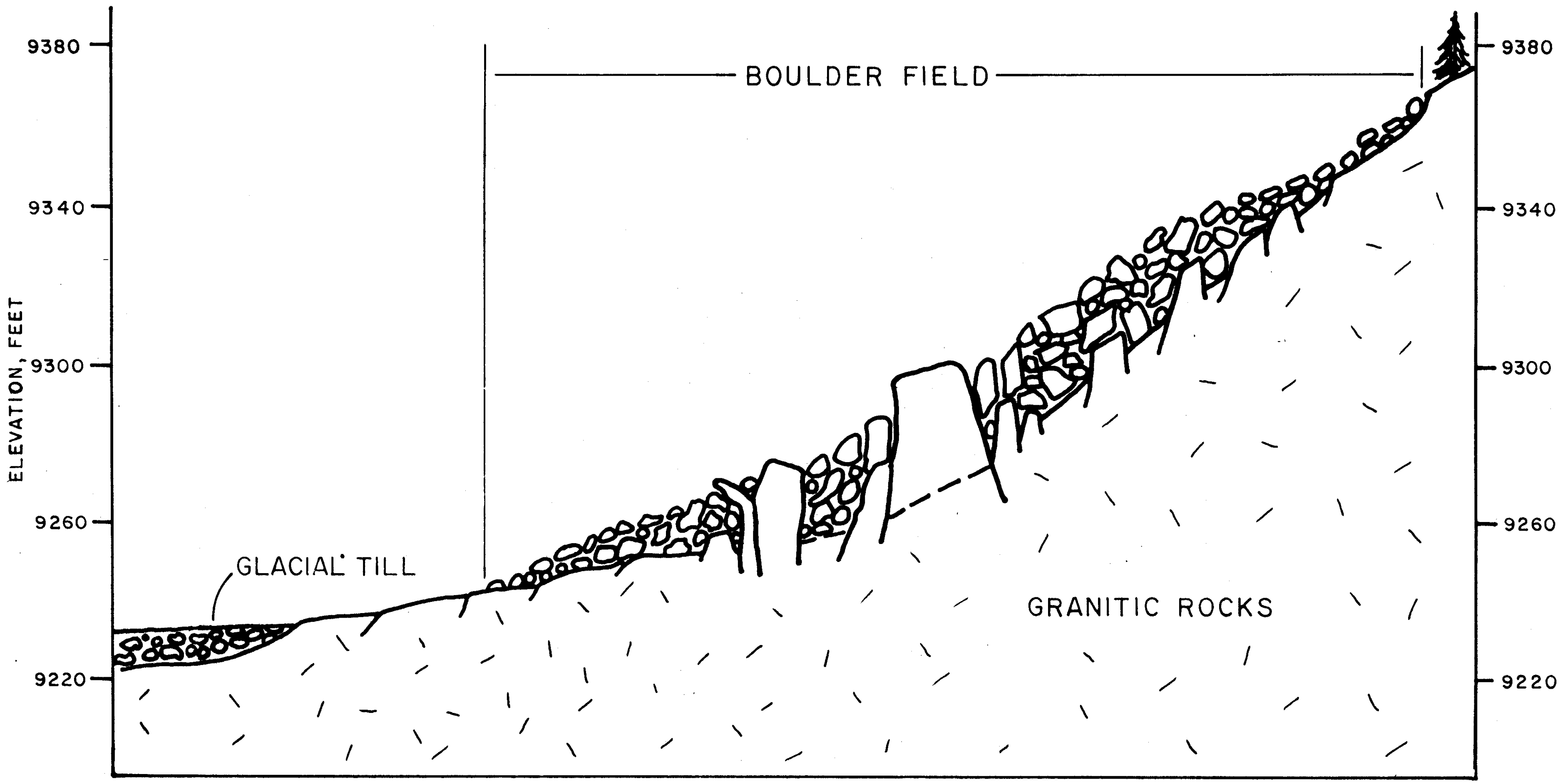
NOTES:
1. Section locations are shown in Figure III-1, Dam-Site Geology and Plan.
2. Stick log abbreviations for trenches (T) and drill holes (DH) are soil classifications by the Unified Soil Classification System (USCS).
3. Geologic notations are defined in Figure III-1.

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY - LEVEL III STUDY

CROSS SECTIONS A-A' AND B-B'

Checked by D. J. Hynek	Date 10-22-85	Project No. 3235	Figure No. III-2
Approved by R.P. Volpe	Date 10/22/85		

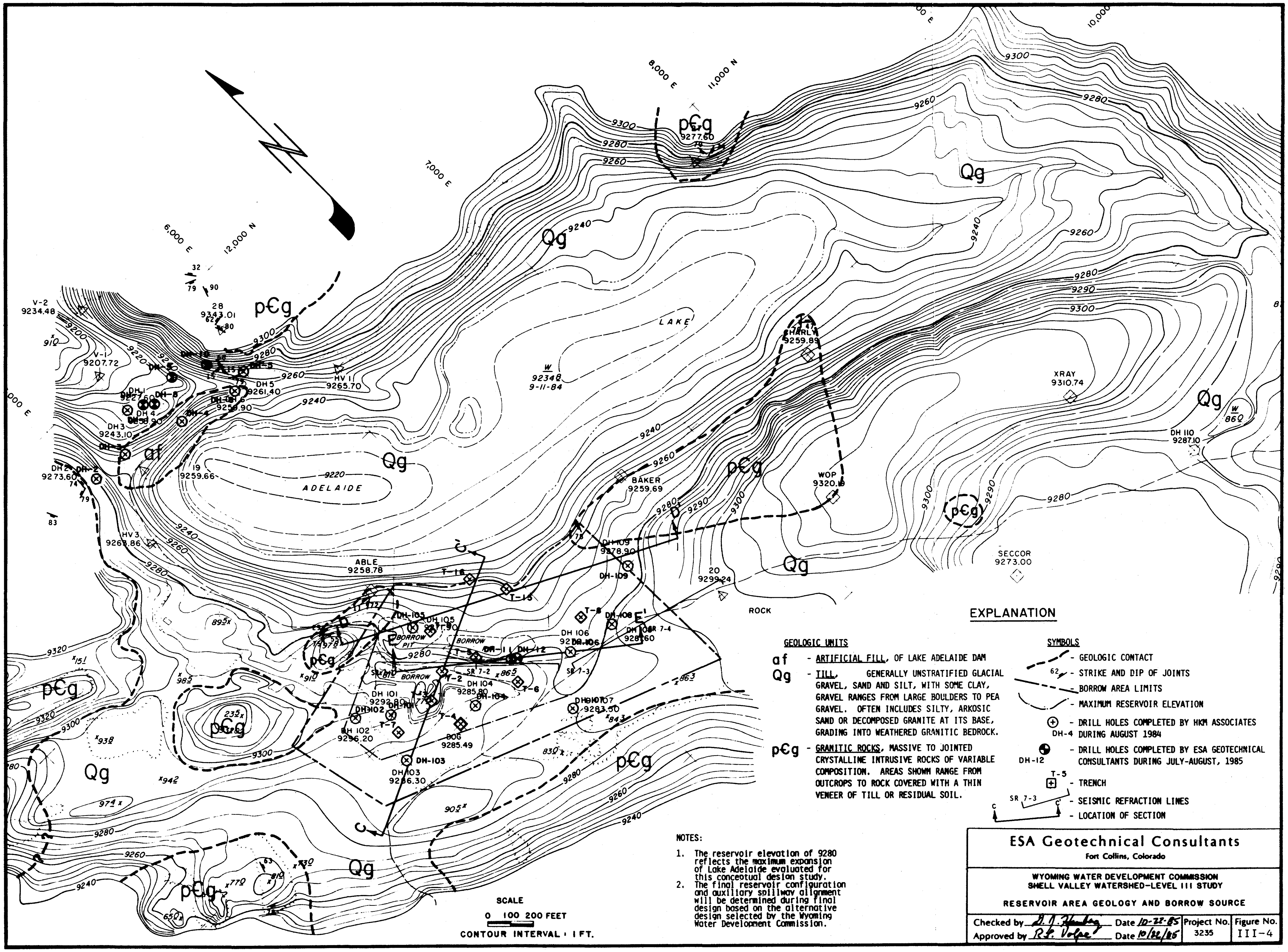


SCALE

0 20 40 FEET

HORIZONTAL = VERTICAL

ESA Geotechnical Consultants Fort Collins, Colorado			
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY PROJECT SCHEMATIC SECTION-RIGHT ABUTMENT			
Checked by D. J. Hamlin Approved by T. L. Volpe	Date 10-22-85 Date 10/22/85	Project No. 3235	Figure No. III-3



EXPLANATION

GEOLOGIC UNITS

af - ARTIFICIAL FILL, OF LAKE ADELAIDE DAM

Qg - TILL, GENERALLY UNSTRATIFIED GLACIAL GRAVEL, SAND AND SILT, WITH SOME CLAY, GRAVEL RANGES FROM LARGE BOULDERS TO PEA GRAVEL. OFTEN INCLUDES SILTY, ARKOSIC SAND OR DECOMPOSED GRANITE AT ITS BASE, GRADING INTO WEATHERED GRANITIC BEDROCK.

pЄg - GRANITIC ROCKS, MASSIVE TO JOINTED CRYSTALLINE INTRUSIVE ROCKS OF VARIABLE COMPOSITION. AREAS SHOWN RANGE FROM OUTCROPS TO ROCK COVERED WITH A THIN VENEER OF TILL OR RESIDUAL SOIL.

SYMBOLS

- GEOLOGIC CONTACT
- 62 - STRIKE AND DIP OF JOINTS
- BORROW AREA LIMITS
- MAXIMUM RESERVOIR ELEVATION
- DH-4 - DRILL HOLES COMPLETED BY HKM ASSOCIATES DURING AUGUST 1984
- DH-12 - DRILL HOLES COMPLETED BY ESA GEOTECHNICAL CONSULTANTS DURING JULY-AUGUST, 1985
- T-5 - TRENCH
- SR 7-3 - SEISMIC REFRACTION LINES
- C C' - LOCATION OF SECTION

NOTES:

1. The reservoir elevation of 9280 reflects the maximum expansion of Lake Adelaide evaluated for this conceptual design study.
2. The final reservoir configuration and auxiliary spillway alignment will be determined during final design based on the alternative design selected by the Wyoming Water Development Commission.

SCALE

0 100 200 FEET

CONTOUR INTERVAL : 1 FT.

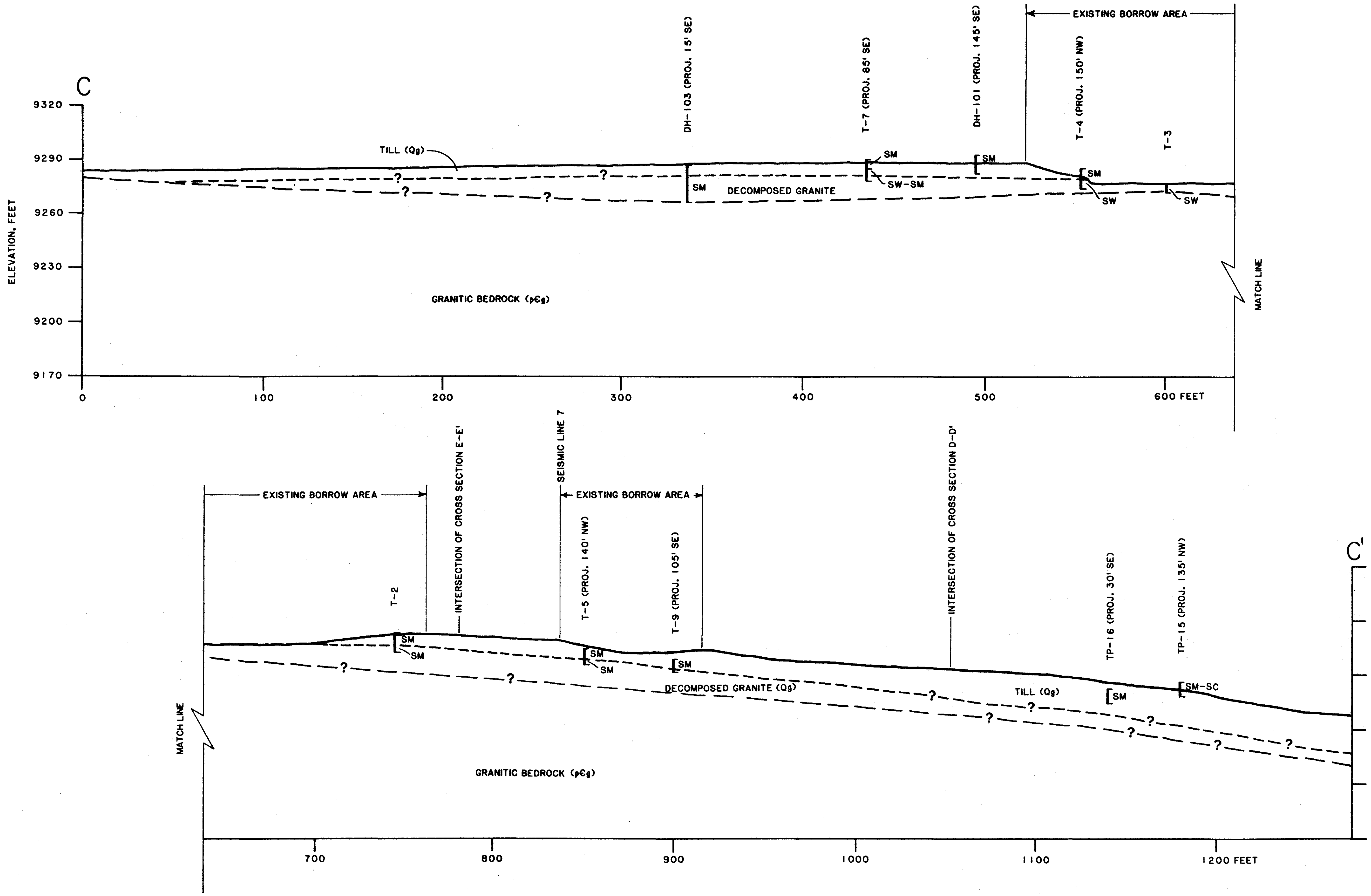
ESA Geotechnical Consultants

Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY

RESERVOIR AREA GEOLOGY AND BORROW SOURCE

Checked by D. J. Hamburg	Date 10-28-85	Project No. 3235	Figure No. III-4
Approved by R. F. Volpe	Date 10/28/85		

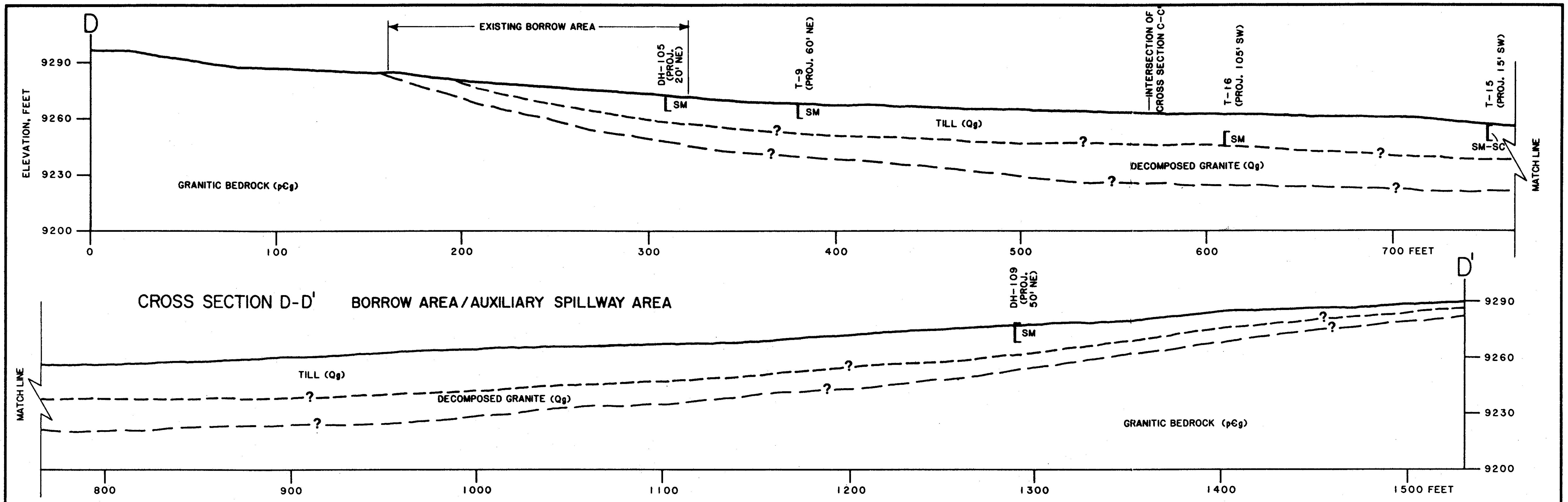


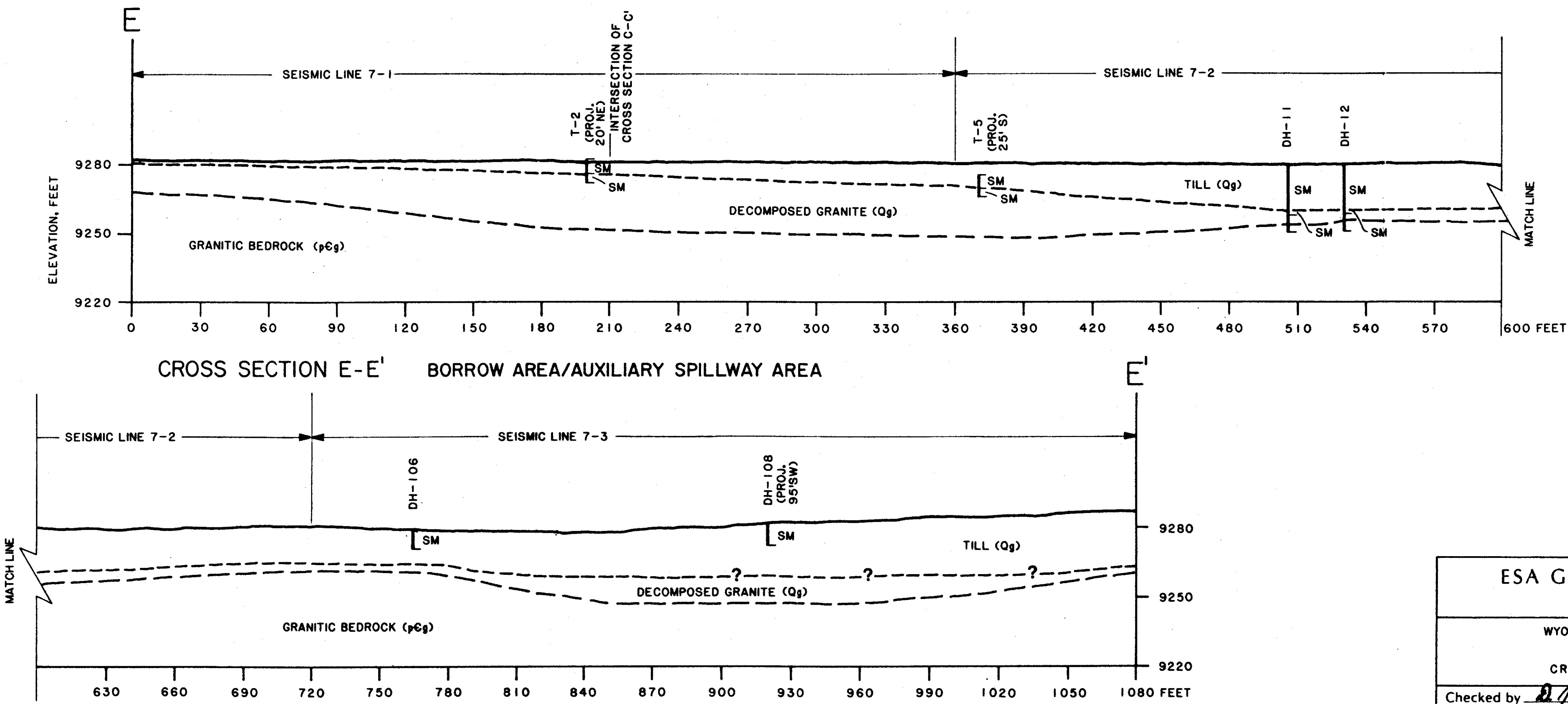
CROSS SECTION C-C'

BORROW AREA / AUXILIARY SPILLWAY AREA

NOTES:
1. Section location is shown in Figure III-4, Reservoir Area Geology and Borrow Source.
2. Stick log abbreviations for trenches (T) and drill holes (DH) are soil classifications by the Unified Soil Classification System (USCS).
3. Geologic notations are defined in Figure III-4.

ESA Geotechnical Consultants Fort Collins, Colorado		
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY- LEVEL III STUDY CROSS SECTION C-C'		
Checked by S. J. Hamby Date 10-22-85 Approved by T.R. Volpe Date 10/22/85	Project No. 3235	Figure No. III-5





NOTES:

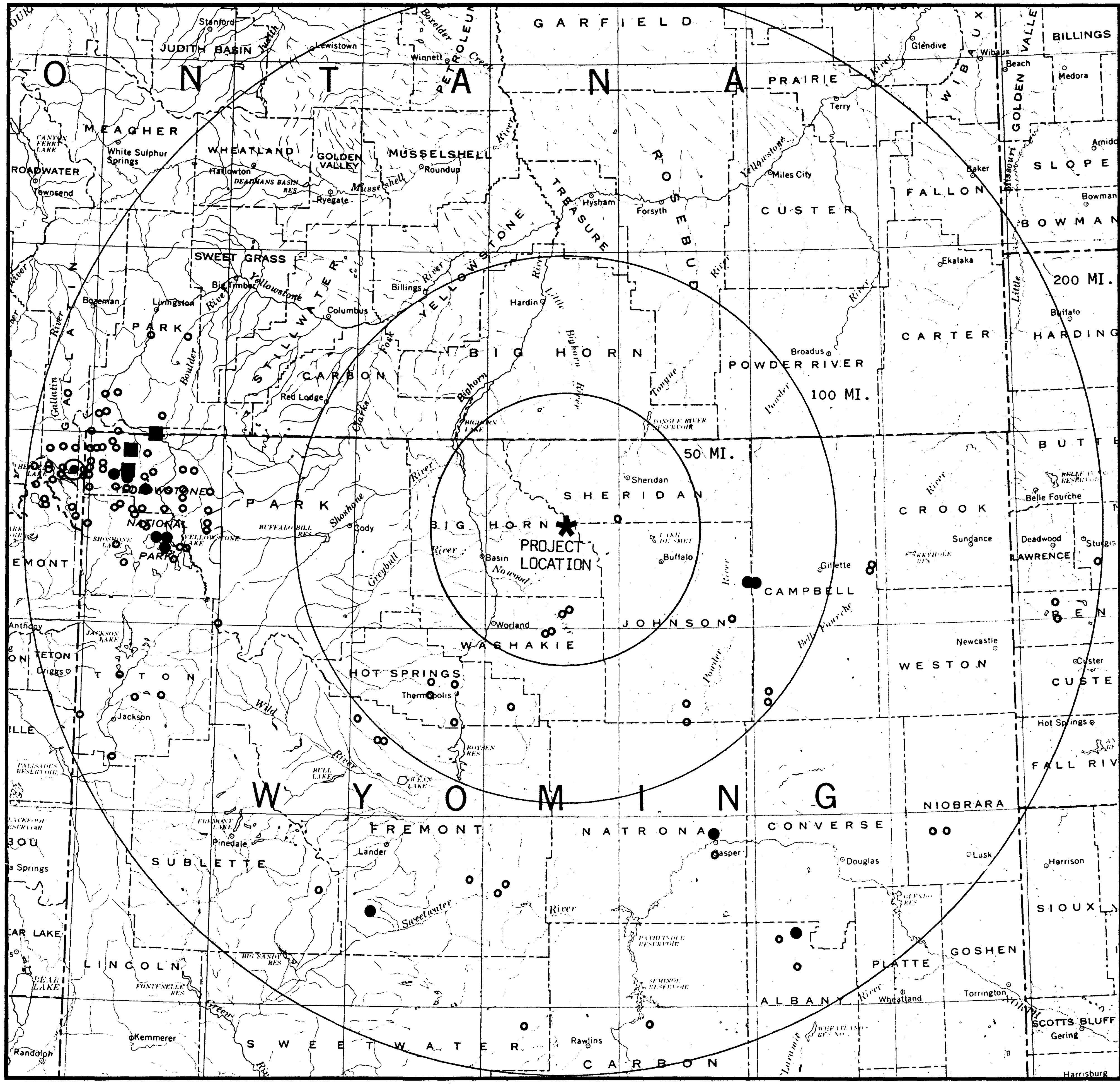
1. Section location is shown in Figure III-4, Reservoir Area Geology and Borrow Source.
2. Stick log abbreviations for trenches (T) and drill holes (DH) are soil classifications by the Unified Soil Classification System (USCS).
3. Geologic notations are defined in Figure III-4.

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY-LEVEL III STUDY

CROSS SECTIONS D-D' AND E-E'

Checked by D. A. Hoenke	Date 10-22-85	Project No. 3235	Figure No. III-6
Approved by R. E. Volpe	Date 10/22/85		



EXPLANATION

SYMBOLS	MAGNITUDE
small open circle	4.0 - 4.9
solid circle	5.0 - 5.9
solid square	6.0 - 6.9
circled dot	7.0 - 7.9

NOTE: Where more than one earthquake has occurred at the same location, only the highest magnitude earthquake is shown.

SOURCE: National Geophysical Data Center
NOAA
Boulder, Colorado

N

SCALE
0 10 25 50 100 MILES

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
LOCATION OF EARTHQUAKE EPICENTERS

Checked by D. J. Hamberg	Date 10-22-85	Project No.	Figure No.
Approved by R. L. Volpe	Date 10/22/85	3235	III-7

IV. WATERSHED HYDROLOGY AND RESERVOIR OPERATION

An integral part of this Level III study for the enlargement of Lake Adelaide included hydrological studies to determine water requirements in the Shell Valley, watershed yield, and optimum reservoir enlargement capacity. In addition, a flood analysis was performed to provide hydrologic design criteria for the spillway. This section presents the results of these analyses.

A. GENERAL DESCRIPTION

The Shell Creek watershed, having a drainage area of approximately 560 square miles (sq. mi.), extends from the higher elevations of the Bighorn Mountains to Shell Creek confluence with the Bighorn River at Greybull (Figure IV-1). Elevations range from 11,000 feet in the mountain high country to 3800 feet at Greybull. Precipitation ranges from approximately 36 inches (mostly snowfall) in the upper elevations to 6 to 8 inches at Greybull.

The majority of the agricultural lands within the Shell Valley are currently under irrigation. Shell Creek provides water for a large percent of these Lands. Major irrigation canals from Shell Creek include Shell Canal, Whaley Ditch and Porter Ditch. When streamflow is at a minimum during the summer months, many shortages occur. To help alleviate these shortages, two reservoirs were constructed; Lake Adelaide in 1915 and Shell Reservoir in 1956. Their total reservoir capacities are approximately 1700 and 1950 acre-feet, respectively. The yield of Lake Adelaide was increased by the construction and operation of a diversion headgate and canal on Buckley Creek, an adjacent tributary to Shell Creek. This canal delivers streamflows from Buckley Creek to Mud Lake which feeds Lake Adelaide. The enlargement of Lake Adelaide is now under consideration as a means of storing additional spring runoffs for summer irrigation use.

B. STREAMFLOW DATA

1. Data Availability

There are three streamflow monitoring stations (gages) located on Shell Creek. Gage number 06278300, Shell Creek above Shell Reservoir, measures all inflows into Shell Reservoir from Shell Creek. It is located just below the confluence with Buckley Creek at the upper end of Shell Reservoir. This gage was installed in October 1956 by the U.S. Geological Survey (USGS) and is still in continuous operation.

Gage number 06278500, Shell Creek near Shell, is located just below the mouth of Shell Canyon and measures all streamflows produced by Shell Creek above the mouth of the canyon. The gage has continuous daily records for the summer and early fall months from October 1941 to the present. Gage number 06279090, Shell Creek near Greybull, is a water quality monitoring station. Periodic streamflow measurements were recorded for a period of January - September 1951, and from July 1965 - September 1983. These measurements were taken on a random and instantaneous basis.

2. Historical Flows

The operational analyses were performed to evaluate the potential yield of Lake Adelaide and the availability of water in meeting the demands in the Shell Valley. The analyses required streamflow data at a number of locations along Shell Creek. The period of record used in this analysis was 1943 through 1982. This corresponds to the longest period of record for the streamflow gages and the most available historic precipitation and snow data.

a. Shell Creek Near Shell - Shell Creek near Shell gage has a total drainage area of approximately 148 square miles. As mentioned earlier, this gage has continuous data available from 1941 with the exception of the winter months for the period 1971 to 1982. In order to complete the data set, flow information for Shell Creek near Shell and precipitation data for Burgess Junction and Basin were compiled and analyzed. Missing historic precipitation data for Burgess Junction and Basin was calculated using the HEC-4 model (COE, 1971). HEC-4 is a generalized computer program which evaluates the statistical relationships between data from several stations. Using these relationships, it estimates values for missing data at one station based on actual data at the other stations.

A regression analysis was then performed on streamflow and precipitation data. This analysis resulted in the equation:

$$Q_{o-m} = 8434 + 0.787 \times T_f - 183 \times M_{pw} + 274 \times M_{ps}$$

Where:

Q_{o-m} = Total flow at Shell near Shell Creek for October through March in acre-feet

T_f = Total summer flow (April-September) at Shell Creek near Shell in acre-feet

M_{pw} = Precipitation for the winter months (October-March) taken as an average of precipitation at Basin and Burgess Junction in inches

M_{ps} = Precipitation for the summer months (April-September) taken as an average of precipitation at Basin and Burgess Junction in inches

This equation resulted in a correlation coefficient of .77. This total calculated winter flow was then distributed over the months October through March by following the historical distribution pattern. Table IV-1 presents the completed record of streamflows at Shell Creek near Shell, Wyoming. The average annual flow was estimated to be approximately 88,300 acre-feet.

b. <u>Shell Creek Above Shell Reservoir</u> - The upper Shell Creek drainage area is approximately 23.1 square miles and is gaged by a station above Shell Reservoir. This station has recorded streamflows for the period of 1956 through 1983, and therefore required synthetic streamflows to be generated for the period of 1943 through 1955 in order to complete the period of record. Correlation analyses were performed to generate these synthetic streamflows. Correlation developed between Shell Creek above the Shell Reservoir gage and the downstream Shell Creek near Shell gage was poor. This lack of correlation is probably due to different streamflow characteristics caused by elevation and upstream regulation by Shell Reservoir and Lake Adelaide.

TABLE IV-1

SHELL VALLEY LEVEL III STUDY

SHELL CREEK NEAR SHELL
(AC-FT)

NODE #054000

WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEP	TOTAL
1943	3050	2740	2660	2380	2060	2320	4100	13200	41290	13000	5240	4180	96220
1944	3030	2700	2260	2160	1970	2100	2220	21660	38020	13790	5210	4470	99590
1945	3380	2860	2420	2330	2030	2120	2090	12100	35670	18950	6560	6240	96750
1946	4370	3320	3020	2620	2340	2950	8220	17710	36360	10010	5310	5830	102060
1947	4900	3830	3270	2770	2480	2510	2790	21800	32160	15650	6130	4990	103280
1948	4110	3620	3420	3000	2410	2460	2600	23620	18400	7670	5340	3180	79830
1949	3000	2710	2600	2280	1850	2050	3170	24140	29100	7510	4500	3840	86750
1950	3070	2600	2300	2140	1860	1900	2620	10120	32190	10660	5980	4480	79920
1951	4200	3030	2520	2040	1790	2020	2350	19690	21610	12560	7100	4170	83080
1952	3380	2740	2520	2390	2020	2040	5460	23020	23290	10020	5800	3480	86160
1953	2840	2380	2300	2350	2030	2220	2100	5700	41040	8650	5470	3590	80670
1954	2680	2520	2580	2350	1980	2070	2450	20200	16950	6360	4200	2140	66480
1955	2170	1880	2000	1800	1740	2020	2270	17720	34370	9620	4810	2940	83340
1956	2600	2430	2410	2270	2270	2370	2560	24250	22100	5660	3930	2650	75500
1957	2290	2180	2090	1870	1690	1910	2000	9950	13830	9500	5750	3960	57020
1958	3080	2570	2390	2100	1830	2090	2000	24540	14530	7510	6310	4130	73080
1959	2760	2440	2350	2160	1760	1980	2250	7010	34010	7800	6010	4240	74770
1960	3230	2400	2350	1960	1850	1800	2520	11190	16610	4970	4710	2860	56450
1961	2870	2020	1980	1770	1490	1590	1720	18120	15520	4250	4160	2990	58480
1962	3250	2900	2480	2030	1860	2000	5410	19630	34660	10600	5460	4200	94480
1963	3380	3070	2660	2150	1950	2030	2360	16810	39870	7580	5210	3490	90560
1964	2880	2620	2310	2180	1940	2080	2020	14560	44950	17500	7040	5250	105330
1965	4130	2960	2880	2570	2070	2180	2600	8030	54500	15840	6940	5530	110230
1966	4220	2860	2620	2360	1990	2220	2190	17470	9700	4510	3550	2270	55960
1967	2730	2130	2130	1740	1560	1980	2050	13420	48180	17300	5760	5470	104450
1968	4950	3600	2980	2550	2200	2260	2230	7150	58910	13310	8280	7990	116410
1969	5760	4540	3710	2860	2320	2550	5440	26920	16150	12840	6220	4260	93570
1970	3820	3000	2720	2410	2120	2190	2190	13940	41260	10860	6180	4430	95120
1971	3060	2540	2530	2390	2040	2200	2410	17220	37970	8700	6900	4140	92100
1972	3720	2470	2300	2050	1790	2620	2750	15990	38590	10730	7660	5340	96010
1973	3370	2970	2830	2430	2160	2530	2510	20340	36230	8480	5820	4580	94250
1974	3480	3000	2802	2440	2150	2320	3210	18030	38260	9960	6980	5100	97732
1975	3330	2870	2680	2340	2060	2220	2150	5760	42510	29070	7770	5800	108560
1976	3800	3270	3060	2670	2350	2530	2730	17840	39780	11710	7490	5130	102360
1977	3550	3050	2860	2490	2200	2360	4220	19210	17140	6110	4840	3280	71810
1978	2890	2480	2320	2030	1780	1920	2590	13510	46350	20410	8600	6930	111810
1979	3860	3320	3100	2710	2380	2560	3210	21710	27700	10850	9710	7070	98180
1980	3310	2850	2670	2330	2050	2200	3280	18030	22130	7880	5740	4460	76930
1981	3220	2780	2600	2260	2000	2140	4240	21120	25400	7820	5980	3840	83400
1982	3360	2890	2700	2360	2080	2230	1950	11810	36020	15707	5740	5510	92357
AVE	3427	2829	2610	2302	2013	2196	2931	16606	31833	11048	6010	4461	88263

As an alternative approach to estimate the missing flow data, an attempt was made to develop a relationship between historical streamflows and precipitation data. The analyses resulted in seven separate equations to estimate streamflows for the months of October through April as follows:

Month	Equation	Correlation Coefficient
Oct	$Q_{oct} = .283\ Q_{sep} + 120\ P_{oct} + 18.5\ T_{oct} - 487$	.65
Nov	$Q_{nov} = .396\ Q_{oct} + 7.58\ P_{nov} - 2.42\ T_{nov} + 166$	.99
Dec	$Q_{dec} = 1.09\ Q_{nov} + 3.52\ T_{dec} + 13.20\ P_{dec} - 190$	.86
Jan	$Q_{jan} = .215\ Q_{dec} + 17.8\ P_{jan} + 70$	.79
Feb	$Q_{feb} = .543\ Q_{jan} + 1.545\ T_{feb} - 3.08\ P_{feb} + 7$	.91
Mar	$Q_{mar} = 1.286\ Q_{feb} - 25$	.98
Apr	$Q_{apr} = 5.194\ Q_{mar} + 11.3\ T_{apr} - 9.176\ P_{apr} - 658$	.84

Where:

Q_x = The Flow for month "x" in acre-feet

P_x = The precipitation for month "x" at Burgess Junction in inches

T_x = The average monthly temperature for month "x" at Burgess Junction in degrees farenheit.

It appears that the streamflows for the months of May through September rely very heavily on snow melt. The analyses indicate that the total volume of runoff for these months could be estimated from the snow pack as measured in April and summer precipitation. The equation developed by regression analysis is:

$$Q_{m-s} = .880\ WE_{sp} + 1715\ P_{bj} - 10427$$

Where:

WE_{sp} = Calculated water equivalent of snow pack of the drainage basin for the Shell above Shell Reservoir gage in inches

P_{bj} = Precipitation at Burgess Junction during the Summer months (May - September) in inches

The correlation coefficient between the recorded flows and the estimated flows based on the above equation was .89. The total summer flow was distributed over the months of May through September by following historical flow patterns.

Table IV-2 presents the completed record of streamflows at Shell Creek above Shell Reservoir. The average annual flow was estimated to be approximately 25,500 acre-feet.

c. <u>Shell Creek at Greybull</u> - The Shell Creek drainage area between the communities of Shell and Greybull accounts for approximately 75 percent of the total 650 square mile drainage area, but contributes no flows most of the time. This is a result of a very low annual precipitation rate (6-8 inches) and a high evaporation rate.

Several attempts were made to correlate precipitation and daily streamflows at Shell with instantaneous streamflows at Greybull. The most favorable selection was a logarithmic regression equation that expressed the streamflows at Greybull as a function of monthly precipitation at Basin and average daily streamflows at Shell. The estimated monthly streamflows at Greybull were generally lower than the streamflows at Shell. The lower flows are due to irrigation diversions between Shell and Greybull. There were three significant inflows between Shell and Greybull during the period of study, 1943 to 1982. These flows were found to be in the month of June and were approximately 369 acre-feet in 1964, 1642 acre-feet in 1968 and 1015 acre-feet in 1976. These flows correspond to months when precipitation was greater than 1.5 inches. The inflows in the other months during the period of study were found to be zero or negligible for this study.

d. <u>Buckley Creek</u> - Buckley Creek is an ungaged stream that drains approximately 3.42 square miles. There exists a diversion on Buckley Creek that diverts some waters into the Adelaide basin while allowing a minimum of 1-2 cubic feet per second (cfs) to remain in Buckley Creek below the diversion. These diversions have not been measured and hence, no records of actual amounts diverted exist. An analysis of the drainage basin indicates that the basin is very comparable to the upper Shell Creek Basin in both topography and

TABLE IV-2

SHELL VALLEY LEVEL III STUDY

SHELL CREEK STREAMFLOWS
ABOVE SHELL RESERVOIR
(AC-FT)

NODE #014000

WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1943	526	322	261	162	120	129	340	9350	15970	2910	791	625	31506
1944	568	337	276	160	115	124	180	7360	12570	2290	623	492	25095
1945	571	341	270	166	120	129	163	9300	15880	2900	787	622	31249
1946	209	213	138	156	116	124	322	5600	9560	1740	473	374	19025
1947	679	386	330	107	86	85	123	8230	14060	2560	696	550	27892
1948	496	304	218	180	129	141	407	6264	13388	3263	838	636	26264
1949	597	347	259	153	99	102	201	5750	9820	1790	487	384	19989
1950	544	331	264	155	98	102	185	8880	15170	2770	751	593	29843
1951	487	318	241	130	98	101	109	9130	15600	2840	772	610	30436
1952	412	275	194	146	101	105	148	4370	7460	1360	370	292	15233
1953	442	286	208	120	99	102	114	5920	10110	1840	501	396	20138
1954	459	298	216	152	108	114	234	5170	8830	1610	437	346	17974
1955	738	409	324	152	107	113	267	7250	12400	2260	614	485	25119
1956	639	366	299	167	122	131	342	5090	8690	1580	431	340	18197
1957	384	268	246	184	167	184	238	2550	15540	3060	585	683	24089
1958	560	455	394	263	194	184	149	17780	5910	2450	1330	433	30102
1959	362	356	277	215	167	154	149	1910	15080	2080	505	316	21571
1960	323	244	246	194	179	227	454	5340	9650	1240	468	327	18892
1961	266	245	185	102	84	70	114	8800	6500	718	251	500	17835
1962	1080	566	641	212	181	209	1050	4610	15290	2980	898	526	28243
1963	387	266	218	136	102	100	117	13000	16880	1200	297	304	33007
1964	246	224	168	104	87	101	127	7090	15860	5420	765	795	30987
1965	453	303	209	210	127	140	201	3850	17980	5070	746	389	29678
1966	506	275	185	141	123	145	147	6240	4090	942	413	270	13477
1967	274	230	172	131	85	85	192	8860	17810	5340	764	650	34593
1968	902	438	206	121	113	126	101	3740	21000	4230	2800	2670	36447
1969	988	521	397	197	137	137	474	15720	6090	3770	666	388	29485
1970	411	296	120	107	86	83	73	5280	16680	2880	599	326	26941
1971	272	236	142	101	80	77	110	5790	16180	2420	765	342	26515
1972	381	236	199	132	91	94	145	5540	14040	2760	1790	1060	26468
1973	726	488	269	162	87	85	130	4360	14430	2060	781	1170	24748
1974	529	317	264	164	121	151	357	4590	15580	2520	729	446	25768
1975	313	267	192	158	114	91	82	937	14090	11530	805	257	28836
1976	219	173	147	106	80	81	100	5610	14470	3770	802	537	26095
1977	612	305	168	129	92	99	812	7470	10600	1180	600	575	22642
1978	713	437	267	219	131	167	224	3500	18960	7370	1530	1210	34728
1979	934	422	327	190	148	155	179	6870	10310	2500	1260	704	23999
1980	392	250	150	95	63	95	323	7090	8540	1470	549	980	19997
1981	910	443	206	132	89	90	504	6950	10060	1410	490	311	21595
1982	403	252	170	133	83	91	141	3730	13920	4360	895	628	24806
AVE.	523	326	242	154	113	121	246	6622	12876	2911	766	589	25488

vegetative cover. This allowed for a straight drainage area ratio, between the two basins, to estimate historic monthly streamflows. The results are depicted in Table IV-3. The average annual flow was estimated to be approximately 3800 acre-feet.

e. Adelaide Creek - The Adelaide Creek drainage basin covers approximately 3.58 square miles and ajoins the Buckley Creek basin at its eastern border. The basin is almost identical to the Buckley Creek basin except that it has less vegetation, a large marsh above the lake, and a more southern exposure. Although the precipitation in the Adelaide Creek basin approximates that in the Buckley Creek basin, it appears that the runoff rate, per a unit drainage area, is smaller. The streamflows for Adelaide basin were estimated using a drainage basin ratio with the upper Shell Creek basin. The results were then adjusted downward to account for evapotranspiration and evaporative losses associated with the 90 acre marsh above the lake, drainage basin losses associated with existing Mud Lake and Arden Lake, drainage basin slope, vegetative cover and percent of the basin with a southern exposure. Utilizing available data and professional judgements, it was determined that the derived Adelaide Creek flows should be reduced by approximately 23%. The resulting estimated streamflows are shown in Table IV-4. The average annual runoff is estimated to be approximately 3000 acre-feet.

C. OPERATION STUDY MODEL

The Level II study presented an extensive evaluation of water use in the Shell Creek basin including diversion requirements and consumptive use. There are approximately 17,500 acres of land which have water rights in the Shell Creek watershed. Approximately 11,500 acres have diversion rights from the main stem of Shell Creek. The major irrigation diversions from Shell Creek are Shell, Whaley and Porter ditches which have decreed rights for the irrigation of approximately 8428 acres. The other major diversion is a decree for municipal water for the City of Greybull. The water availability for these four major diversions were analyzed through an operation study. Other water rights are relatively small and mostly located below Porter Ditch. It is considered that they are not impacted nor will they impact the results of this operation study.

TABLE IV-3

SHELL VALLEY LEVEL III STUDY

BUCKLEY CREEK STREAMFLOWS
(AC-FT)

WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	JULY	AUG	SEP	TOTAL
1943	78	48	39	24	18	19	50	1384	2364	431	117	93	4663
1944	84	50	41	24	17	18	27	1089	1860	339	92	73	3714
1945	85	50	40	25	18	19	24	1376	2350	429	116	92	4625
1946	31	32	20	23	17	18	48	829	1415	258	70	55	2816
1947	100	57	49	16	13	13	18	1218	2081	379	103	81	4128
1948	73	45	32	27	19	21	60	927	1981	483	124	94	3887
1949	88	51	38	23	15	15	30	851	1453	265	72	57	2958
1950	81	49	39	23	15	15	27	1314	2245	410	111	88	4417
1951	72	47	36	19	15	15	16	1351	2309	420	114	90	4505
1952	61	41	29	22	15	16	22	647	1104	201	55	43	2254
1953	65	42	31	18	15	15	17	876	1496	272	74	59	2980
1954	68	44	32	22	16	17	35	765	1307	238	65	51	2660
1955	109	61	48	22	16	17	40	1073	1835	334	91	72	3718
1956	95	54	44	25	18	19	51	753	1286	234	64	50	2693
1957	57	40	36	27	25	27	35	377	2300	453	87	101	3565
1958	83	67	58	39	29	27	22	2631	875	363	197	64	4455
1959	54	53	41	32	25	23	22	283	2232	308	75	47	3193
1960	48	36	36	29	26	34	67	790	1428	184	69	48	2796
1961	39	36	27	15	12	10	17	1302	962	106	37	74	2640
1962	160	84	95	31	27	31	155	682	2263	441	133	78	4180
1963	57	39	32	20	15	15	17	1924	2498	178	44	45	4885
1964	36	33	25	15	13	15	19	1049	2347	802	113	118	4586
1965	67	45	31	31	19	21	30	570	2661	750	110	58	4392
1966	75	41	27	21	18	21	22	924	605	139	61	40	1995
1967	41	34	25	19	13	13	28	1311	2636	790	113	96	5120
1968	133	65	30	18	17	19	15	554	3108	626	414	395	5394
1969	146	77	59	29	20	20	70	2327	901	558	99	57	4364
1970	61	44	18	16	13	12	11	781	2469	426	89	48	3987
1971	40	35	21	15	12	11	16	857	2395	358	113	51	3924
1972	56	35	29	20	13	14	21	820	2078	408	265	157	3917
1973	107	72	40	24	13	13	19	645	2136	305	116	173	3663
1974	78	47	39	24	18	22	53	679	2306	373	108	66	3814
1975	46	40	28	23	17	13	12	139	2085	1706	119	38	4268
1976	32	26	22	16	12	12	15	830	2142	558	119	79	3862
1977	91	45	25	19	14	15	120	1106	1569	175	89	85	3351
1978	106	65	40	32	19	25	33	518	2806	1091	226	179	5140
1979	138	62	48	28	22	23	26	1017	1526	370	186	104	3552
1980	58	37	22	14	9	14	48	1049	1264	218	81	145	2960
1981	135	66	30	20	13	13	75	1029	1489	209	73	46	3196
1982	60	37	25	20	12	13	21	552	2060	645	132	93	3671
AVE.	77	48	36	23	17	18	36	980	1906	431	113	87	3772

TABLE IV-4

SHELL VALLEY LEVEL III STUDY

ADELAIDE CREEK
(AC-FT)

NODE #020000

WATER YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1943	63	38	31	19	14	15	41	1116	1906	347	94	75	3759
1944	68	40	33	19	14	15	21	878	1500	273	74	59	2994
1945	68	41	32	20	14	15	19	1110	1895	346	94	74	3728
1946	25	25	16	19	14	15	38	668	1141	208	56	45	2270
1947	81	46	39	13	10	10	15	982	1678	305	83	66	3328
1948	59	36	26	21	15	17	49	748	1598	389	100	76	3134
1949	71	41	31	18	12	12	24	686	1172	214	58	46	2385
1950	65	39	32	18	12	12	22	1060	1810	331	90	71	3562
1951	58	38	29	16	12	12	13	1090	1862	339	92	73	3634
1952	49	33	23	17	12	13	18	521	890	162	44	35	1817
1953	53	34	25	14	12	12	14	706	1206	220	60	47	2403
1954	55	36	26	18	13	14	28	617	1054	192	52	41	2146
1955	88	49	39	18	13	13	32	865	1480	270	73	58	2998
1956	76	44	36	20	15	16	41	607	1037	189	51	41	2173
1957	46	32	29	22	20	22	28	304	1854	365	70	82	2874
1958	67	54	47	31	23	22	18	2122	705	292	159	52	3592
1959	43	42	33	26	20	18	18	228	1800	248	60	38	2574
1960	38	29	29	23	21	27	54	637	1152	148	56	39	2253
1961	32	29	22	12	10	8	14	1050	776	86	30	60	2129
1962	129	68	76	25	22	25	125	550	1825	356	107	63	3371
1963	46	32	26	16	12	12	14	1551	2014	143	35	36	3937
1964	29	27	20	12	10	12	15	846	1893	647	91	95	3697
1965	54	36	25	25	15	17	24	459	2146	605	89	46	3541
1966	60	33	22	17	15	17	18	745	488	112	49	32	1608
1967	33	27	21	16	10	10	23	1057	2125	637	91	78	4128
1968	108	52	25	14	13	15	12	446	2506	505	214	319	4229
1969	118	62	47	24	16	16	57	1876	727	450	79	46	3518
1970	49	35	14	13	10	10	9	630	1990	344	71	39	3214
1971	32	28	17	12	10	9	13	691	1931	289	91	41	3164
1972	45	28	24	16	11	11	17	661	1675	329	214	126	3157
1973	87	58	32	19	10	10	16	520	1722	246	93	140	2953
1974	63	38	32	20	14	18	43	548	1859	301	87	53	3076
1975	37	32	23	19	14	11	10	112	1681	1376	96	31	3442
1976	26	21	18	13	10	10	12	669	1727	450	96	64	3116
1977	73	36	20	15	11	12	98	891	1265	141	72	69	2703
1978	85	52	32	26	16	20	27	418	2263	879	183	204	4205
1979	111	50	39	23	18	18	21	820	1230	298	150	84	2862
1980	47	30	18	11	8	11	39	846	1019	175	66	143	2413
1981	108	53	25	16	11	11	60	829	1200	168	58	37	2576
1982	48	30	20	16	10	11	17	445	1661	520	107	75	2960
AVE	62	39	29	18	14	14	29	790	1537	347	88	72	3041

In the operation study, various sizes of Lake Adelaide were analyzed to determine an optimum enlargement capacity. The enlargement of Lake Adelaide will supplement the water supply needed for irrigation by the Shell Creek users. Shell reservoir was not included in the operation study.

The operation study was performed using the Wyoming Integrated River System Operation Study (WIRSOS) model. The model was originally developed for use in connection with the adjudication of water rights in the Bighorn River Basin. The WIRSOS model is essentially an accounting model based on the prior appropriation doctrine and the "one-fill" rule for reservoir storage.

1. Operation Study Nodes

The WIRSOS model utilizes nodal points for the accounting procedure. Nodes were established in the drainage basin at all points where inflow, diversions and/or return flows occurred (See Figures IV-2 - IV-5). A description of each node is as follows:

Node 10000 - This node is located on Buckley Creek above the confluence with Shell Creek. The streamflows passing through this node are the estimated Buckley Creek streamflows less diversions to Adelaide Creek.

Node 11000 - This node is located on Shell Creek above the confluence with Buckley Creek. Streamflows at this node are actual Shell Creek streamflows, as measured at the existing USGS stream gage above Shell Reservoir, less historic Buckley Creek inflows.

Node 14000 - This node represents the existing USGS streamflow gaging station at the upper end of Shell Reservoir.

Node 20000 - This node represents the estimated historical streamflows into Lake Adelaide. The model locates this point just upstream of any Buckley Creek contributing diversions.

Node 21000 - Diversions from the Buckley Creek watershed to Lake Adelaide are accounted for at this node. This node is located just upstream of the diversion canal confluence with Adelaide Creek.

Node 25000 - This node represents Lake Adelaide. All inflows to Lake Adelaide from all sources are measured at this node.

Node 26000 - This node reflects all releases from Lake Adelaide. In addition, all instream flow requirements for Adelaide Creek are called at this node.

Node 30000 - This node is located just downstream of the Shell Creek/Adelaide Creek confluence. All streamflows and reservoir releases upstream of this node are measured at this node.

Node 40000 - Flows defined for this node are to account for all streamflows generated in the reach downstream of nodes 14000 and 26000 and upstream of the Shell Creek streamflow gaging station near Shell.

Node 50000 - The Shell Creek streamflow gaging station near Shell is represented by this node. The total streamflow at this node is the resultant of all streamflows generated in the upper Shell Creek drainage basin.

Node 54000 - This node represents the Shell Canal diversion point and is used for the City of Greybull diversions. It also reflects a portion of the Shell Canal return flows.

Node 58000 - All Whaley Ditch diversions are made at this node. It also reflects some return flows from Whaley Ditch.

Node 62000 - This node is utilized for Porter Ditch diversions and return flows.

Node 63000 - Remaining return flows from Shell Canal and Whaley Ditch are accounted for at this node, which is located below the Porter Ditch diversion.

Node 65000 - All precipitation-generated streamflows occurring between the Shell gage near Shell and the Greybull gaging station at Greybull enter the system at this node.

Node 66000 - This node represents the streamflows entering the Bighorn River from Shell Creek at Greybull.

2. Operation Study Inflows

The model used in this study is an accounting type model which accounts for all inflows from the upper to the lower basin. It was set up to accept six inflow stations. These stations were defined as follows:

Upper Shell Creek (Node 11000) - The inflows from this stream were the actual and estimated gaged records at the gaging station above Shell Reservoir less the estimated historic Buckley Creek streamflows. Historic Buckley Creek streamflows were assumed to be 2 cfs or the natural flow, whichever was less.

Buckley Creek (Node 10000 and 20000) - The Buckley Creek streamflows were split into two parts. The first part is a minimum flow required for the Buckley Creek downstream of the diversion as defined at Node 10000. The remainder is specified at Node 20000 as a diversion to Adelaide Creek. Historically, the minimum flow passing through the diversion was 2 cfs or the natural flow, whichever was less. For the operational analyses of the enlarged Lake Adelaide, the minimum flow is assumed to be 1.3 cfs as specified by the Wyoming Game and Fish Department (1985).

Adelaide Creek (Node 20000) - Inflows at this node are the actual streamflows as previously estimated.

Shell Canyon (Node 40000) - Inflows at this node refer to the inflows downstream of Shell and Adelaide reservoirs to the Shell Creek gage near Shell. To estimate these inflows, Adelaide Creek (Node 25000) and Upper Shell Creek (Node 011000) flows were subtracted from flows for Shell Creek near Shell (Node 50000). The inflows were then modified to account for the historical impacts of Lake Adelaide storage. Historic records for Lake Adelaide operation are not available. However, based on interviews with local residents, it was assumed that the reservoir is opened in the middle of July and closed at the end of September. A reservoir simulation was conducted to determine the effects of the lake on the downstream flows. The resulting impacts were then applied to the estimated Shell Canyon inflows.

It was assumed that Shell Reservoir would be operated in a manner similar to past operations and that its impacts were accounted for in the historic record.

Shell/Greybull - The inflows developed for this reach in the regression analysis were used (Node 65000).

3. Streamflow Losses

Lake Adelaide was included in the operation study to analyze its water supply capability. Net evaporation losses were assumed to be 14.3 inches/year based on evaporation losses estimated for the Bighorn basin (Rice, 1965). The monthly distribution of Lake Adelaide evaporation losses are given below:

Month	Evaporation loss in inches
January	0
February	0
March	0
April	0
May	.6
June	2.6
July	4.5
August	4.6
September	2.0
October	0
November	0
December	0
TOTAL	14.3

The amount of evaporation loss (acre-feet) from the reservoir for a given month was computed as reservoir surface area (acres) times evaporation loss (feet) as given in the above table.

Substantial losses for the Adelaide outflow have been observed for the reach between the dam and the diversion point at Shell Canal, especially at the beginning of reservoir release. However, these losses were not separately considered in the operation analyses, assuming that channel losses were accounted in the historic records.

4. Diversions

The diversions included in the operation study were Shell, Whaley and Porter Ditches and the city of Greybull diversion. Table IV-5 gives a summary of the information used in the model for each water right. The "Acreage" column lists the number of acres that can be irrigated with each water right, and the "Diversions" column lists the amount of water needed for an average year on the land. The "efficiency" in the heading indicates a ratio of consumptive use to diversion requirement.

TABLE IV-5
SHELL CANAL
(Node 54000, Efficiency 35%)

Permit Number	Priority Date	Acreage (acres)	Diversions (ac-ft/yr)
TERR	04/01/1886	245	688
TERR	04/01/1887	235	660
430	03/07/1893	65	183
271E	09/18/1897	537	1509
462E	09/18/1899	278	781
1330E	01/06/1905	591	1660
1439E	05/22/1905	190	534
1938E	04/10/1907	20	56
8290	03/20/1908	876	2462
2084E	06/01/1909	2405	6758
5312E	05/24/1941	140	393
5986E	06/23/1959	42	118
6091E	01/18/1963	20	56
	Subtotals	5644	15,860

WHALEY DITCH
(Node 58000, Efficiency 40%)

Permit Number	Priority Date	Acreage (acres)	Diversions (ac-ft/yr)
TERR	04/01/1889	142	399
42E	06/17/1893	692	1945
462E	09/18/1899	156	438
650E	04/22/1901	269	756
1330E	01/06/1905	24	67
1730E	12/10/1906	150	422
5420E	04/19/1945	40	112
5472E	07/26/1948	15	42
6196E	12/06/1967	3	8
	Subtotals	1491	4189

PORTER DITCH
(Node 62000, Efficiency 42%)

Permit Number	Priority Date	Acreage (acres)	Diversions (ac-ft/yr)
365	11/18/1892	1050	2950
322E	01/21/1898	146	410
1464E	11/03/1905	54	153
1726E	04/22/1907	43	120
	Subtotals	1293	3633

CITY OF GREYBULL
(Node 54000, Efficiency 100%)

Permit Number	Priority Date	Acreage (acres)	Diversions (ac-ft/yr)
430	03/07/1893	-	958
19279	09/21/1938	-	160
	Subtotals	-	1118
	Totals	8,428	24,800

The actual irrigation requirements vary by the month of the year. Table IV-6 gives an average monthly distribution of irrigation requirements (HKM, 1985).

Table IV-6
Average Monthly Irrigation Requirements
(Percent of Total)

April	May	June	July	August	September	October
2.1	8.4	18.2	32.8	25.0	10.4	3.2

The diversions for the City of Greybull were considered to be a continuous demand at the City's decreed withdrawal rate.

5. Return Flows

Return flows from irrigation diversions were asusmed to be 50 percent of the amount of diversion less consumptive use. It was further assumed that half of the return flow would be returned to the stream as surface runoff without time delay. The remainder would be returned to the stream as ground water flow. The ground water return flow is assumed to be exponentially decreasing with time as governed by Darcy's ground water flow equation. The diffusivity required in the equation was assumed to be 3×10^6 gpd/ft. With these assumptions, two return flow patterns were developed as presented in Table IV-7. Return flow pattern #1 is applicable for a distance of approximately 1 mile between the canal and the point of return, while return flow pattern #2 is used for return distances of approximately 2 miles or greater.

Table IV-7
Irrigation
Return Flow Pattern
(Percentage)

MONTHS

Pattern	1	2	3	4	5	6	7	8	9	10	11	12
#1	76	10	4	4	2	1	1	1	1	0	0	0
#2	57	15	8	5	4	3	2	2	1	1	1	1

Approximately thirty-three percent of the water diverted by Shell Canal is returned to Shell Creek at two locations. Twenty three percent of the return flow is returned to the diversion point (Node 54000). This represents the amount of return flow water that enters the system above the Whaley Ditch diversion point (HKM, 1985). Return flow pattern #1 was used for these return flows. The remaining return flow is returned below the Porter Ditch diversion (Node 63000) utilizing return flow pattern #2.

Whaley Ditch is forty percent efficient and approximately thirty percent of the water diverted is returned to Shell Creek. Fifty percent of the return flows are returned below Porter Ditch diversion (Node 63000) utilizing return flow pattern #2, and are not available for use by Porter Ditch. The remaining fifty percent of the return flows reenter the system at the point of diversion (Node 58000) while utilizing return flow pattern #1.

Porter Ditch is forty-two percent efficient and returns twenty-nine percent of the waters diverted to Shell Creek using return flow pattern #1. All of these return flows are accounted for at the diversion point (Node 62000).

The diversions made by the City of Greybull (Node 54000) are for municipal use in Greybull, and no return flows are considered.

7. Results

The main purpose of the operation study is to analyze the water supply capability of Lake Adelaide, thereby providing information on an optimum size of the reservoir enlargement.

The existing Lake Adelaide has a total storage capacity of approximately 1700 acre-feet with a dead storage of 219 acre feet. However, the decreed storage water right is 1448 acre-feet (HKM, 1985). In analyzing the existing reservoir, the decreed storage right was used to limit the maximum amount of water storable in a given year by the direction of Wyoming Water Development Commission.

In addition to analyzing the existing reservoir, several sizes of the reservoir were analyzed to determine an optimum reservoir enlargement. The optimum enlarged reservoir capacity (total storage capacity) was determined to be approximatelty 4500 acre-feet. As the capacity is increased above 4500 acre-feet, the inflows are not adequate to fill the reservoir on a normal basis. In addition, physical constraints of the site does not support further enlargements past 4500 acre-feet. The incremental costs for the extra capacity would increase rapidly. The actual enlarged reservoir capacity (total storage capacity) was estimated to be 4548 acre-feet with 234 acre-feet of dead storage. However, the hydrologic analysis is based on the total storage capacity of 4500 acre-feet.

Five different case study results are presented in this report. All case studies assumed that Lake Adelaide would provide supplemental water only for Shell Canal. Also, the dead storage was assumed to be 219 acre-feet for all cases. Actual dead storage will be 234 acre-feet due to the downstream alignment of the new dam compared with the existing dam.

The computer output of each case study is volumenous and is bound separately. The results are summarized in subsequent paragraphs.

Case #1 was conducted assuming existing Lake Adelaide conditions with the 1448 acre-feet storage right. The results are summarized in Table IV-8. Column 2 under "Shell Demand on Reservoir" indicates a call made on the reservoir by Shell Canal users when their demands are not met by non-stored flows. In those years which show zero in Column 2, natural flows were sufficient to meet the Shell Canal demand and

TABLE IV-8

CASE #1

LAKE ADELAIDE YIELD
BASE CASE

YEAR	SHELL DEMAND ON RESERVOIR (1) (AC-FT)	RESERVOIR SUPPLY (AC-FT)	RESERVOIR SURPLUS (AC-FT)	SHELL CANAL SHORTAGE (AC-FT)	RESERVOIR YIELD (2) (AC-FT)
1943	0	0	1120	0	1120
1944	0	0	1119	0	1119
1945	0	0	1119	0	1119
1946	1954	1205	0	749	1205
1947	0	0	1120	0	1120
1948	4184	1205	0	2979	1205
1949	5084	1205	0	3879	1205
1950	787	787	350	0	1137
1951	0	0	1120	0	1120
1952	2046	1205	0	841	1205
1953	3286	1205	0	2081	1205
1954	6651	1205	0	5446	1205
1955	2968	1205	0	1763	1205
1956	6715	1205	0	5510	1205
1957	2120	1205	0	915	1205
1958	4286	1205	0	3081	1205
1959	4249	1205	0	3044	1205
1960	5757	1205	0	4552	1205
1961	6894	1205	0	5689	1205
1962	793	793	343	0	1136
1963	4507	1205	0	3302	1205
1964	0	0	1120	0	1120
1965	0	0	1119	0	1119
1966	7995	1205	0	6790	1205
1967	0	0	1120	0	1120
1968	0	0	1119	0	1119
1969	0	0	1119	0	1119
1970	553	553	574	0	1127
1971	3083	1205	0	1878	1205
1972	719	719	416	0	1135
1973	3396	1205	0	2191	1205
1974	1794	1205	0	589	1205
1975	0	0	1124	0	1124
1976	0	0	1119	0	1119
1977	4960	1205	0	3755	1205
1978	0	0	1121	0	1121
1979	670	670	465	0	1135
1980	4274	1205	0	3069	1205
1981	4274	1205	0	3069	1205
1982	0	0	1121	0	1121
TOTALS	93999	28827	17828	65172	46655
AVERAGES	2350	721	446	1629	1166

NOTE:

1) The amount of calls by the Shell Canal on the reservoir, ie., shortage of the natural flow in meeting the average annual demand of 15,860 AC-FT for the Shell Canal.

2) Reservor supply for the Shell Canal (col. 3) plus surplus water left in Lake Adelaide at the end of the irrigation season (col. 4)

there was no need to release water from the reservoir. Column 3 under "Reservoir Supply" indicates reservoir releases to meet the calls by Shell Canal. Column 4 under "Reservoir Surplus" indicates surplus water left in the reservoir at the end of the irrigation season, not including dead storage. Column 5 under "Shell Canal Shortage" indicates shortage in meeting the irrigation demand by Shell Canal users as implied by Column 2 minus Column 3. Column 6 under "Reservoir Yield" was computed as the sum of Column 3 plus Column 4. Values in Column 6 indicate reservoir yield that can be expected for each year.

The operation analyses presented herein, was performed under the assumption that reservoir water would be released only when needed, that is, when the natural flow does not satisfy the Shell Canal demand. The historical operation is different from this assumption. Normally, water is released from the reservoir in the middle of July, regardless of downstream water requirements.

From Table IV-8, the reservoir yield is approximately 1200 acre-feet on an average annual basis. The water supply is short in meeting the Shell Canal demand approximately half of the time. It should be noted that this estimate of water shortage is based on the assumption that 5644 acres of land which have water rights on the Shell Canal are fully irrigated. Based on the latest estimate, 3488 acres are currently irrigated (HKM, 1985). Thus, water shortage occurrances under presently irrigated land conditions would be smaller than estimated above.

Cases #2 - #5 presented below are comparisons of predicted conditions utilizing Adelaide Reservoir at 4500 acre-feet under different scenarios. Acres irrigated, shortages, and reservoir surpluses, etc., are the same as previously described in Case #1.

Case #2 considered enlarging Adelaide Lake to 4500 acre-feet and allowed for no minimum instream flows in Adelaide Creek. Instream flow requirements of 1.3 cfs or the natural inflow, whichever was less (Wyoming Game and Fish Department, 1985) at the diversion point was considered for Buckley Creek. This resulted in shortages in Shell Canal in 11 of 40 years or 28 percent of the time (Table IV-9). Table IV-9 presents data in the same format as Table IV-8.

TABLE IV-9

CASE #2

LAKE ADELAIDE YIELD
WITHOUT MINIMUM FLOW (1)

YEAR	SHELL DEMAND ON RESERVOIR (2) (AC-FT)	RESERVOIR SUPPLY (AC-FT)	RESERVOIR SURPLUS (AC-FT)	SHELL CANAL SHORTAGE (AC-FT)	RESERVOIR YIELD (3) (AC-FT)
1943	0	0	4123	0	4123
1944	0	0	4119	0	4119
1945	0	0	4119	0	4119
1946	1709	1709	2424	0	4133
1947	0	0	4119	0	4119
1948	3807	3807	357	0	4164
1949	4943	4249	0	694	4249
1950	787	787	3343	0	4130
1951	0	0	4119	0	4119
1952	1802	1802	2332	0	4134
1953	3044	3044	1108	0	4152
1954	6510	4249	0	2261	4249
1955	2489	2489	1654	0	4143
1956	6411	4249	0	2162	4249
1957	1875	1875	2271	0	4146
1958	4286	4243	0	43	4243
1959	4255	4255	0	0	4255
1960	5562	3898	0	1664	3898
1961	6692	3886	0	2806	3886
1962	793	793	3338	0	4131
1963	4507	4243	0	264	4243
1964	0	0	4125	0	4125
1965	0	0	4119	0	4119
1966	7909	4243	0	3666	4243
1967	0	0	4124	0	4124
1968	0	0	4119	0	4119
1969	0	0	4119	0	4119
1970	553	553	3567	0	4120
1971	2841	2841	1309	0	4150
1972	719	719	3409	0	4128
1973	3154	3154	998	0	4152
1974	1550	1550	2584	0	4134
1975	0	0	4121	0	4121
1976	0	0	4119	0	4119
1977	4650	4243	0	407	4243
1978	0	0	4127	0	4127
1979	670	670	3456	0	4126
1980	4274	4243	0	31	4243
1981	4274	4249	0	25	4249
1982	0	0	4127	0	4127
TOTALS	90066	76043	89849	14023	165892
AVERAGES	2252	1901	2246	351	4147

NOTE:
1) No minimum flow for Adelaide Creek
2) The amount of calls by the Shell Canal on the reservoir, ie., shortage of the natural flow in meeting the average annual demand of 15,860 AC-FT for the Shell Canal.
3) Reservoir supply for the Shell Canal (col. 3) plus surplus water left in Lake Adelaide at the end of the irrigation season (col. 4).

Case #3 allowed for minimum instream flows in Adelaide Creek of 1.6 cfs or the natural inflow, whichever was less, (Wyoming Game and Fish Department, 1985) and 1.3 cfs or the natural inflow, whichever was less, for Buckley Creek. The results are very comparable to Case #2. Average annual shortages increased slightly and reservoir yield decreased slightly as compared to Case #2 (Table IV-10).

Case #4 utilized no minimum flow requirement for Adelaide Creek and 1.3 cfs or the natural flow, whichever was less, as a minimum flow requirement for Buckley Creek. In addition, a minimum pool of 838 acre-feet was maintained (Wyoming Game and Fish Department, 1985). This resulted in a reduction in reservoir yield of approximately 15 percent and increased average yearly shortages by approximately 50 percent as compared to Case #2 (Table IV-11).

Case #5 was conducted using the criteria described in Case #4, except that a minimum flow requirement of 1.6 cfs or the natural inflow, whichever was less, was imposed on Adelaide Creek. The results are very comparable to Case #4. (Table IV-12).

Table IV-13 presents a summary of the five cases previously discussed. The enlargement of Adelaide Creek to 4500 acre-feet will increase the yield from 1200 acre-feet to 3500 - 4100 acre-feet, depending upon the environmental operating constraints that may be required. The probability of obtaining the full (100% water supply) diversion requirements from Shell Canal will increase from 52 percent of the time to 68 - 73 percent. The enlargement will allow 80 percent of the demand to be met 98 percent of the time. The most apparent advantage of the enlargement is the reduction of average annual shortages from 1679 acre-feet/year to approximately 400 - 500 acre-feet/year.

These results are simulations only and are relative. Past operating practices have not allowed for an optimization of the system. The reservoir gate is normally opened in July and closed in September. The model used in this study allows for optimization and the only releases made are spills and calls by water users. The predicted yields associated with the enlargement requires that the releases be monitored and controlled.

TABLE IV-10

CASE #3

LAKE ADELAIDE YIELD
WITH MINIMUM FLOW REQUIREMENT (1)

YEAR	SHELL DEMAND ON RESERVOIR (2) (AC-FT)	RESERVOIR SUPPLY (AC-FT)	RESERVOIR SURPLUS (AC-FT)	SHELL CANAL SHORTAGE (AC-FT)	RESERVOIR YIELD (3) (AC-FT)
1943	0	0	4124	0	4124
1944	0	0	4119	0	4119
1945	0	0	4119	0	4119
1946	1709	1709	2424	0	4133
1947	0	0	4119	0	4119
1948	3807	3807	357	0	4164
1949	4948	4052	0	896	4052
1950	787	787	3344	0	4131
1951	0	0	4119	0	4119
1952	1802	1802	2332	0	4134
1953	3044	3044	1108	0	4152
1954	6510	4249	0	2261	4249
1955	2489	2489	1655	0	4144
1956	6411	4249	0	2162	4249
1957	1875	1875	2272	0	4147
1958	4286	4243	0	43	4243
1959	4248	4021	0	227	4021
1960	5591	3486	0	2105	3486
1961	6717	3566	0	3151	3566
1962	793	793	3340	0	4133
1963	4507	4243	0	264	4243
1964	0	0	4126	0	4126
1965	0	0	4119	0	4119
1966	7909	4243	0	3666	4243
1967	0	0	4125	0	4125
1968	0	0	4119	0	4119
1969	0	0	4119	0	4119
1970	553	553	3567	0	4120
1971	2841	2841	1309	0	4150
1972	719	719	3409	0	4128
1973	3154	3154	998	0	4152
1974	1550	1550	2585	0	4135
1975	0	0	4122	0	4122
1976	0	0	4119	0	4119
1977	4651	4243	0	408	4243
1978	0	0	4129	0	4129
1979	670	670	3456	0	4126
1980	4274	4243	0	31	4243
1981	4279	4046	0	233	4046
1982	0	0	4128	0	4128
TOTALS	90124	74677	89862	15447	164539
AVERAGES	2253	1867	2247	386	4113

NOTE:
1) Minimum bypass flow requirement of 1.6 cfs on Adelaide Creek.
2) The amount of calls by the Shell Canal on the reservoir, ie., shortage of the natural flow in meeting the average annual demand of 15,860 AC-FT for the Shell Canal.
3) Reservoir supply for the Shell Canal (col. 3) plus surplus water left in Lake Adelaide at the end of the irrigation season (col. 4).

TABLE IV-11

CASE #4

LAKE ADELAIDE YIELD
NO MINIMUM FLOW, MINIMUM POOL (1)

YEAR	SHELL DEMAND ON RESERVOIR (2) (AC-FT)	RESERVOIR SUPPLY (AC-FT)	RESERVOIR SURPLUS (AC-FT)	SHELL CANAL SHORTAGE (AC-FT)	RESERVOIR YIELD (3) (AC-FT)
1943	0	0	3502	0	3502
1944	0	0	3500	0	3500
1945	0	0	3500	0	3500
1946	1709	1709	1805	0	3514
1947	0	0	3500	0	3500
1948	4124	3628	0	496	3628
1949	4972	3628	0	1344	3628
1950	787	787	2723	0	3510
1951	0	0	3500	0	3500
1952	1802	1802	1713	0	3515
1953	3044	3044	489	0	3533
1954	5981	3628	0	2353	3628
1955	2489	2489	1033	0	3522
1956	6459	3628	0	2831	3628
1957	1875	1875	1649	0	3524
1958	4291	3628	0	663	3628
1959	4254	3634	0	620	3634
1960	5574	3628	0	1946	3628
1961	6711	3628	0	3083	3628
1962	793	793	2717	0	3510
1963	4512	3628	0	884	3628
1964	0	0	3504	0	3504
1965	0	0	3500	0	3500
1966	7945	3628	0	4317	3628
1967	0	0	3503	0	3503
1968	0	0	3500	0	3500
1969	0	0	3500	0	3500
1970	553	553	2948	0	3501
1971	2841	2841	690	0	3531
1972	719	719	2790	0	3509
1973	3154	3154	379	0	3533
1974	1550	1550	1965	0	3515
1975	0	0	3502	0	3502
1976	0	0	3500	0	3500
1977	4770	3628	0	1142	3628
1978	0	0	3506	0	3506
1979	670	670	2837	0	3507
1980	4279	3628	0	651	3628
1981	4279	3628	0	651	3628
1982	0	0	3506	0	3506
TOTALS	90137	69156	72761	20981	141917
AVERAGES	2253	1729	1819	525	3548

NOTE:
1) No minimum flow for Adelaide Creek, minimum reservoir pool of 838 AC-FT.
2) The amount of calls by Shell Canal on the reservoir, ie., shortage of the natural flow in meeting the average annual demand of 15,860 AC-FT for the Shell Canal.
3) Reservoir supply for the Shell Canal (col. 3) plus surplus water in Lake Adelaide at the end of the irrigation season (col. 4).

TABLE IV-12

CASE #5

LAKE ADELAIDE YIELD
MINIMUM FLOW & MINIMUM POOL (1)

YEAR	SHELL DEMAND ON RESERVOIR (2) (AC-FT)	RESERVOIR SUPPLY (AC-FT)	RESERVOIR SURPLUS (AC-FT)	SHELL CANAL SHORTAGE (AC-FT)	RESERVOIR YIELD (3) (AC-FT)
1943	0	0	3503	0	3503
1944	0	0	3500	0	3500
1945	0	0	3500	0	3500
1946	1709	1709	1805	0	3514
1947	0	0	3500	0	3500
1948	4124	3628	0	496	3628
1949	4978	3616	0	1362	3616
1950	787	787	2723	0	3510
1951	0	0	3500	0	3500
1952	1802	1802	1713	0	3515
1953	3044	3044	489	0	3533
1954	5724	3628	0	2096	3628
1955	2489	2489	1034	0	3523
1956	6459	3628	0	2831	3628
1957	1875	1875	1650	0	3525
1958	4291	3628	0	663	3628
1959	4254	3634	0	620	3634
1960	5586	3456	0	2130	3456
1961	6717	3542	0	3175	3542
1962	793	793	2719	0	3512
1963	4512	3628	0	884	3628
1964	0	0	3504	0	3504
1965	0	0	3500	0	3500
1966	7945	3628	0	4317	3628
1967	0	0	3504	0	3504
1968	0	0	3500	0	3500
1969	0	0	3500	0	3500
1970	553	553	2948	0	3501
1971	2841	2841	690	0	3531
1972	719	719	2790	0	3509
1973	3154	3154	379	0	3533
1974	1550	1550	1966	0	3516
1975	0	0	3503	0	3503
1976	0	0	3500	0	3500
1977	4770	3628	0	1142	3628
1978	0	0	3507	0	3507
1979	670	670	2837	0	3507
1980	4279	3628	0	651	3628
1981	4279	3628	0	651	3628
1982	0	0	3507	0	3507
TOTALS	89904	68886	72771	21018	141657
AVERAGES	2248	1722	1819	525	3541

NOTE:
1) Minimum bypass flow on Adelaide Creek of 1.6 cfs, minimum pool is 838 AC-FT.
2) the amount of calls by Shell Canal on the reservoir, ie., shortage of the natural flow in meeting the average annual demand of 15,860 AC-FT for the Shell Canal.
3) Reservoir supply for the Shell Canal (col. 3) plus surplus water in Lake Adelaide at the end of the irrigation season (col. 4).

Table IV-13
Lake Adelaide Yield
and Shell Canal Water Supply

	Case #1 Existing Reservoir	Case #2 Enlarged Reservoir	Case #3 Enlarged Reservoir	Case #4 Enlarged Reservoir	Case #5 Enlarged Reservoir
Capacity (ac-ft)	1700	4500	4500	4500	4500
Dead Storage (ac-ft)	219	219	219	219	219
Min. Flow Req.(cfs) for Adelaide Cr.	0	0	1.6	0	1.6
Min Flow Req.(cfs) for Buckley Cr.	2	1.3	1.3	1.3	1.3
Min. Pool Req. (ac-ft)	0	0	0	838	838
Annual Reservoir Yield (ac-ft)					
8 out of 10 yrs	1100	4100	4100	3500	3500
10 out of 10 yrs	1100	3900	3500	3500	3450
Average Annual	1200	4100	4100	3500	3450

Percent of time that Shell Canal can meet diversion requirements (15,680 ac-ft/yr).

	Case #1	Case #2	Case #3	Case #4	Case #5
100% Water Supply	52%	73%	70%	68%	68%
Greater than 90% Water Supply	60%	88%	88%	88%	88%
Greater than 80% Water Supply	80%	98%	98%	98%	98%
Greater than 70% Water Supply	90%	100%	100%	100%	100%

D. FLOOD STUDIES

1. Probable Maximum Flood

The Probable Maximum Precipitation (PMP) is the maximum precipitation, associated with a duration, that could occur over a given land area. Two types of storms can occur; local and general. Both were evaluated to determine the critical event.

The Adelaide Creek drainage basin is approximately 3.58 square miles in area and has an average elevation of 10,000 feet. It was determined that a local storm over Adelaide Basin would produce 10.6 inches in 6 hours while a general storm would produce 27 inches over a seventy-two hour period (NOAA, 1984). The general storm was determined to be the critical storm and all reservoir and stream flood routings utilized this storm.

The PMP storm is most likely to occur between the months of June and September. Should the event occur in June there is a high probability that a ground cover of snow may still be present. This condition would result in additional runoff. A three day snow melt value of 3.6 inches (HKM, 1985) was estimated. This results in a total event of 30.6 inches (Figure IV-6) for the general PMP. The temporal distribution of the PMP over a 72-hour period was made using a procedure specified in the HEC-1 flood hydrograph package (COE, 1979).

The Probable Maximum Flood (PMF) hydrograph that would result from the PMP was estimated using HEC-1, which includes a number of hydrograph simulation techniques. The SCS Dimensionless Unit Hydrograph approach which was used for this study. Inputs required for the model include an SCS curve number and lag time.

The SCS curve number is related to the infiltration characteristics of the various soil groups. Curve numbers range from 0 to 100 and, as the value of the curve number increases, the runoff excess increases. The SCS provides information on relating soil group type to the curve number as a function of soil cover, land use type and antecedent moisture conditions. The soils in the Shell drainage basin were evaluated and weighted by percent, utilizing the soil survey for Big Horn County (SCS, 1982). The analysis provided an SCS curve number with a value of 68.

The lag time is defined as the lag (hours) between the center of mass of the rainfall excess and the time of peak discharge as defined by the hydrograph. The lag time for this basin was computed as 1.35 hours, using the U.S. Army Corps of Engineers lag time formula shown below.

$$LAG = 24n\left(\frac{L\ Lc}{\sqrt{s}}\right)^{.38}$$

Where: n is the Manning's roughness coefficient,

L is the maximum travel distance along the main stream measured in miles.

L_c is the distance along the main stream to a point opposite the centroid of the basin measured in miles.

s is the weighted slope of the channel in feet per mile.

The PMP hydrograph developed is shown in Figure IV-7. The peak inflow was estimated to be approximately 6800 cfs. This can be compared to flows of 10,300 cfs developed by HKM (1985) and 7800 cfs developed by the COE (1979). Different results were largely due to the different model parameters used such as rainfall amount, rainfall distribution, and watershed characteristics. The present study is based on the most current information and the most detailed ivestigation of the watershed. In addition, a comparison of the value of 6800 cfs with PMP estimates in similar watersheds in Wyoming were found to be reasonable.

2. <u>100 Year Storm</u>

The Level II Study (HKM, 1985) estimated the 100-year peak flow to be 340 cfs using a formula developed by the USGS. This USGS formula was developed based upon a regional frequency analysis. When used on a specific region, i.e., a mountainous region like the present area of study, this USGS equation could produce erroneous results. A re-evaluation of the 100-year frequency storm event was conducted in this study. Utilizing the Precipitation-Frequency Atlas of the Western United States (National Weather Service, 1973), a storm with a 24-hour duration resulted in a total rainfall of 4.2 inches. Applying this storm over the basin and using the procedures previously described, a peak inflow of 670 cfs was developed (Figure IV-8).

3. Reservoir Routing

Both the 100-year and the PMF inflow hydrographs were applied as design floods to evaluate routing through Lake Adelaide. For flood routing purposes, the dam crest was assumed to be at elevation 9287. In the preliminary stages of the flood routing analysis, assumptions were made regarding the spillway arrangement in order to provide general outlfow criteria for the final spillway design. The preliminary spillway concept was a combination of a primary ogee spillway, 45 feet wide with a 9280 crest elevation, operating in tandem with a 120 foot wide auxiliary spillway with a crest elevation of 9283 feet. The final recommended spillway configuration is a side channel service spillway operating in tandem with an emergency spillway in the borrow area. Flood routing for the recommended spillway arrangement must be evaluated and optimized during final design.

Applying the PMP inflow hydrograph to the ogee-auxiliary combination, the peak outflow was approximately 5300 cfs at a stage of 9286.7 feet elevation, or 0.3 feet below the dam crest. The PMP inflow and outflow hydrographs are shown in Figure IV-7.

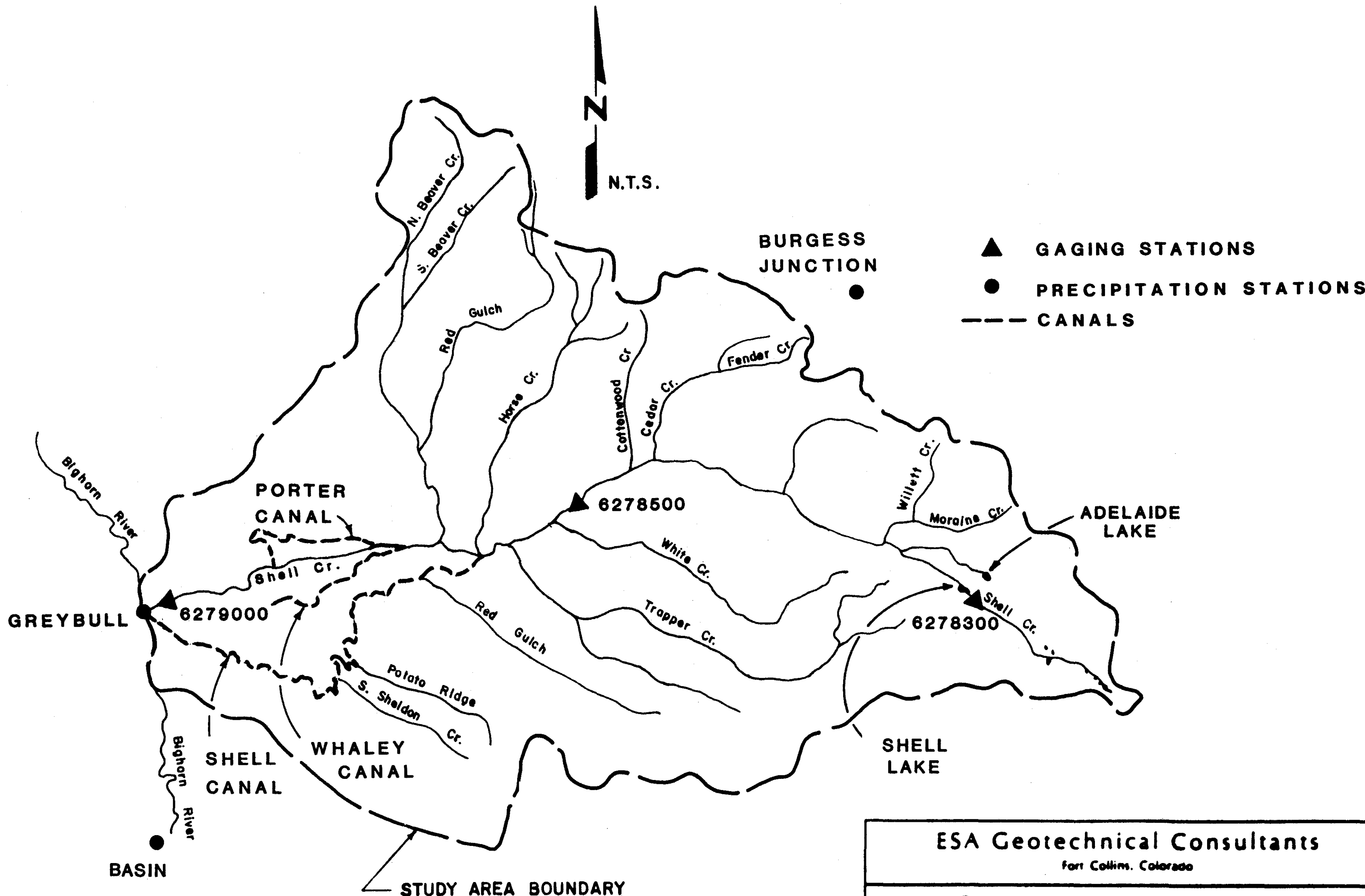
The peak outflow for the 100-year storm event was 212 cfs at an elevation of 9281 feet through the ogee spillway only. The 100-year hydrographs are shown in Figure IV-8.

4. Dam Break Analysis

An analysis of the effects of a catastrophic dam break was also considered. The PMP was routed into the reservoir and through a spillway capable of passing approximately 2800 cfs (1/2 PMP peak). It was assumed that the dam would start breaking if the water overtops the dam by 0.25 feet and approximately seventy percent of the dam would be eroded in 15 minutes. In reality, the erosion due to over topping would occur much more slowly. In addition, the dam would probably not be eroded below the elevation of the rock toe, which for the Alternative B configuration is estimated will be about 9260 feet. Thus, only about 34 percent of the total embankment height would be lost.

For comparison purposes, both the PMP outflow without a dam break and the dam break outflow were routed downstream of the dam. Channel cross-sections were taken along Adelaide and Shell Creeks to aid in the evaluation of flood evaluations (Figure IV-9 and IV-13).

Figures IV-11 - IV-13 compare the water depths along Shell and Adelaide Creeks associated with the PMP event and a dam failure event. A dam break does have considerable impact on stream depths as compared to a controlled PMP event. There are two reaches of the stream where a dam failure event would cause major damage. The first reach of concern is along Shell Creek at the Shell Creek campground. The estimated depth of water at this point would flood the campground and damage may result. The second area of concern is just below the highway bridge at the mouth of Shell Canyon. A residence is located in the floodplain and inundation would be highly likely.



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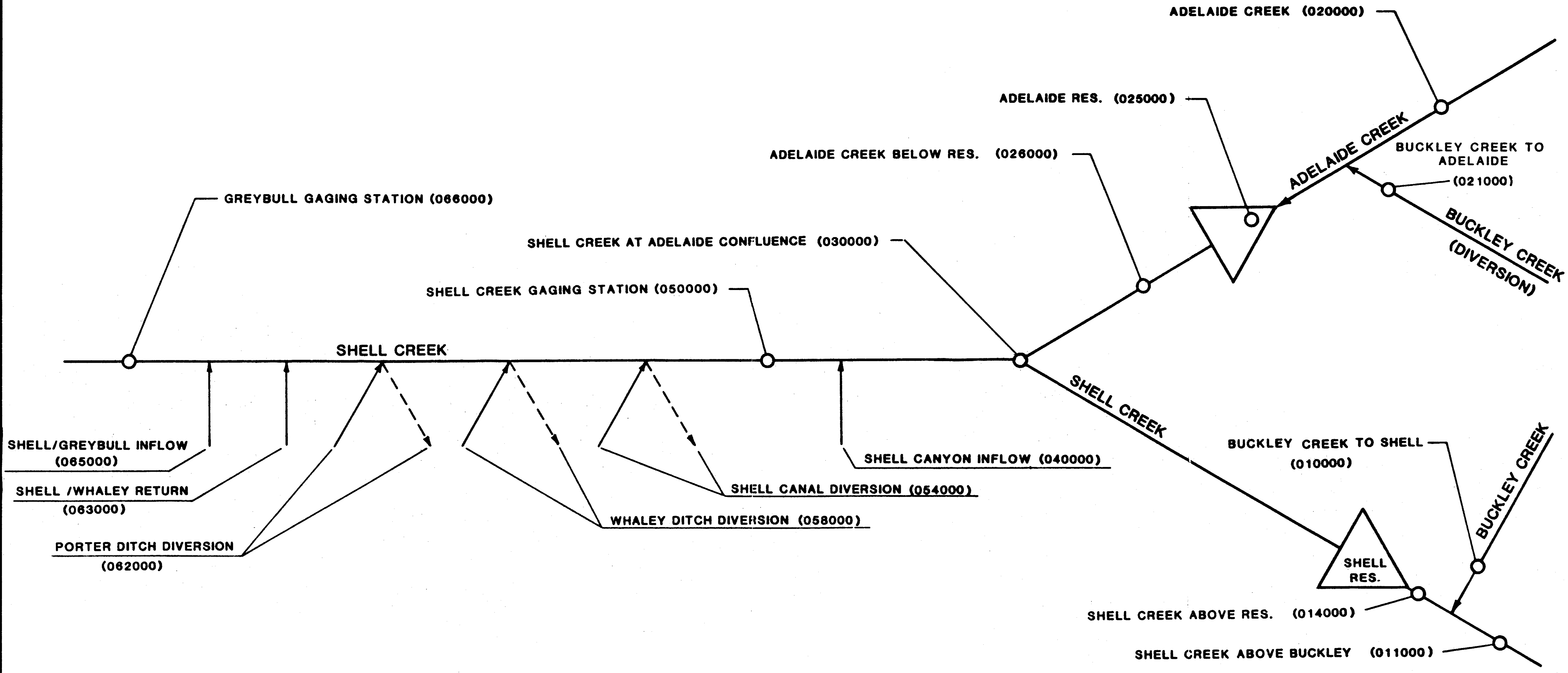
SHELL CREEK DRAINAGE BASIN

Checked by ______ Date ______	Project No.	Figure No.
Approved by ______ Date ______		IV-1



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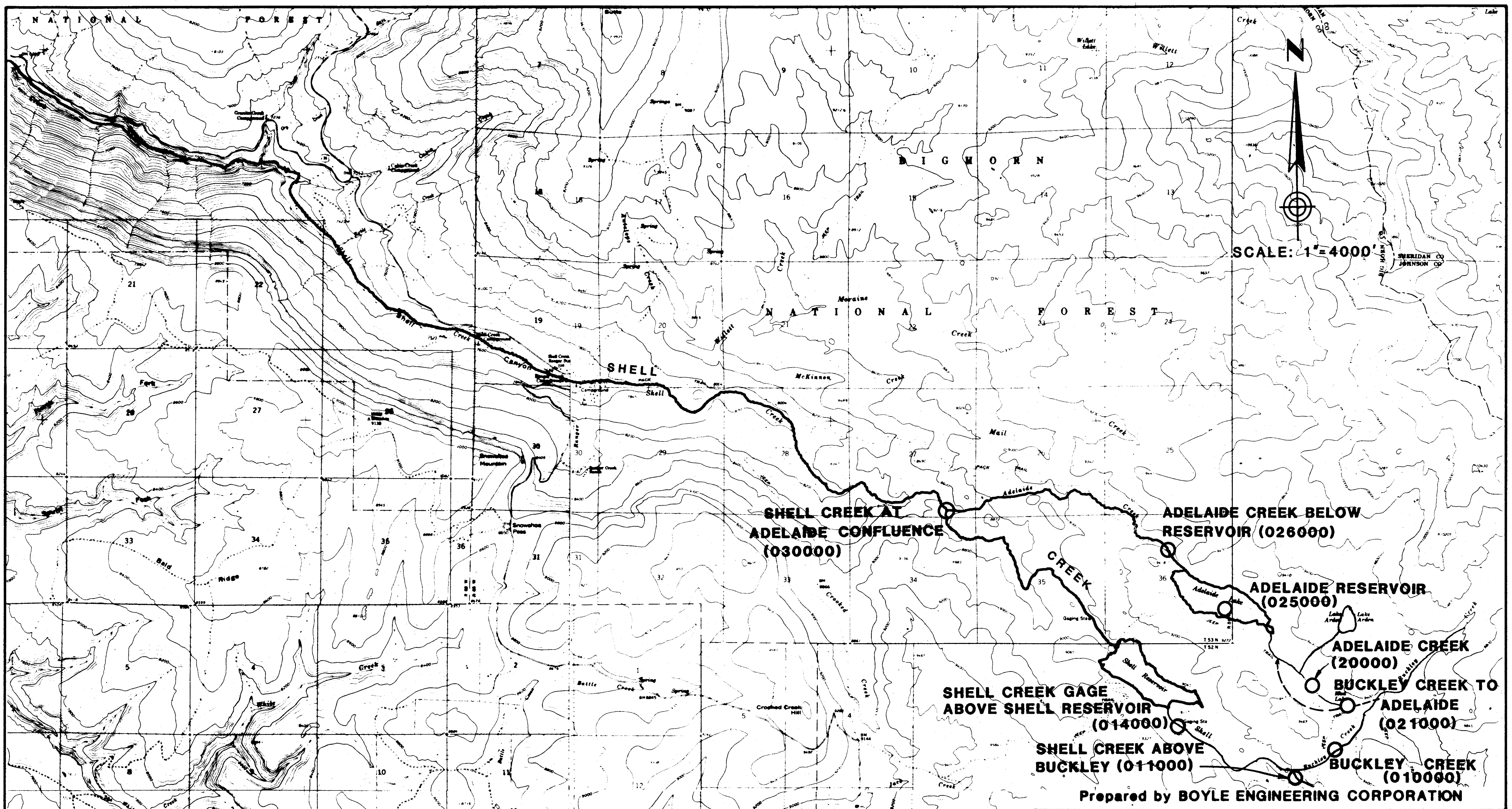
WYOMING WATER DEVELOPMENT COMMISSION

SHELL VALLEY WATERSHED-LEVEL III STUDY

OPERATION STUDY MODEL NODES

Checked by RJM	Date 10/22/85	Project No. 3235	Figure No. IV-2
Approved by Alm	Date 10/22/85		



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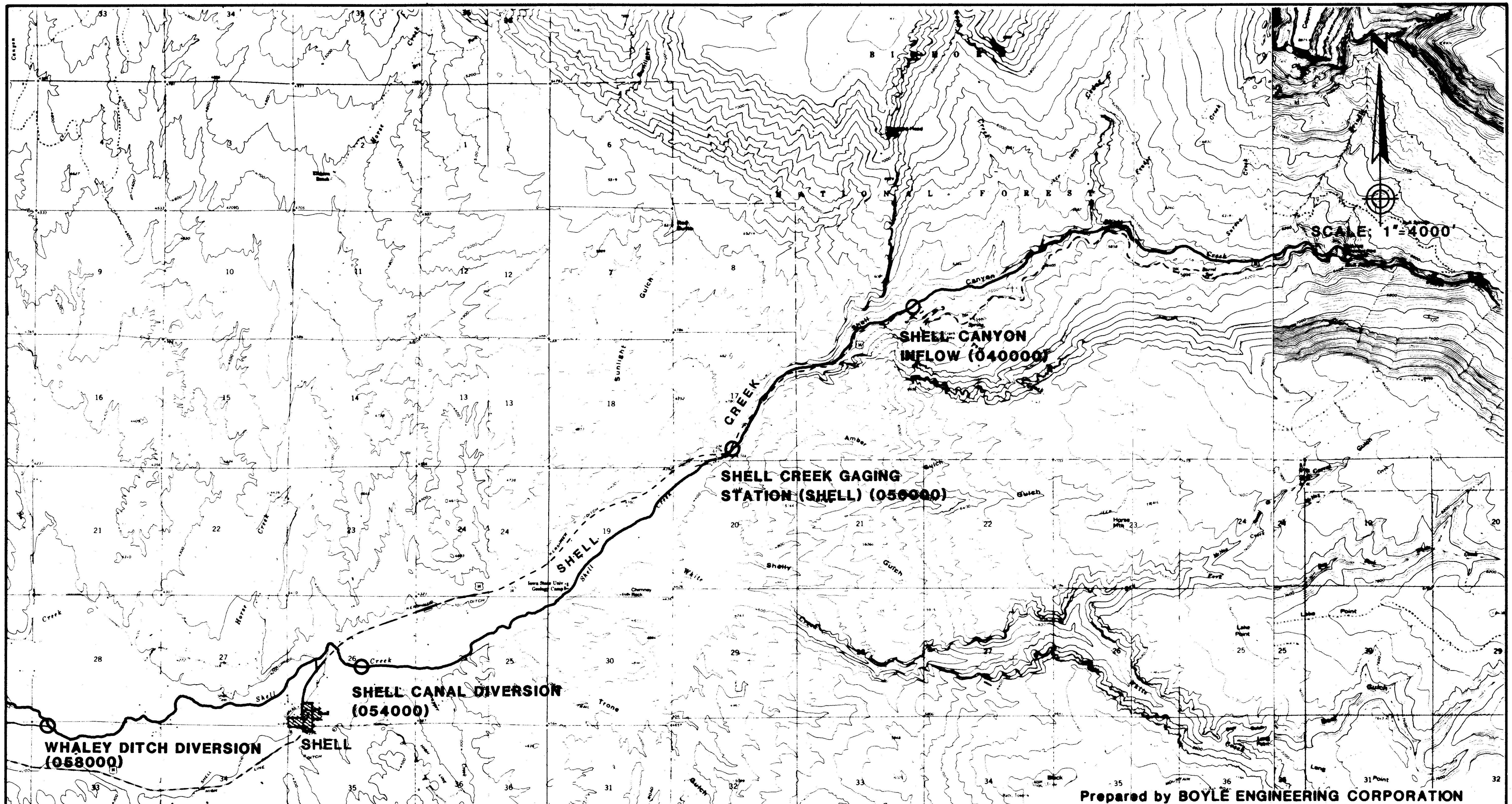
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY

OPERATION STUDY NODE LOCATIONS

Checked by RJM Date 10/22/85	Project No.	Figure No.
Approved by ALM Date 10/22/85	3235	IV-3



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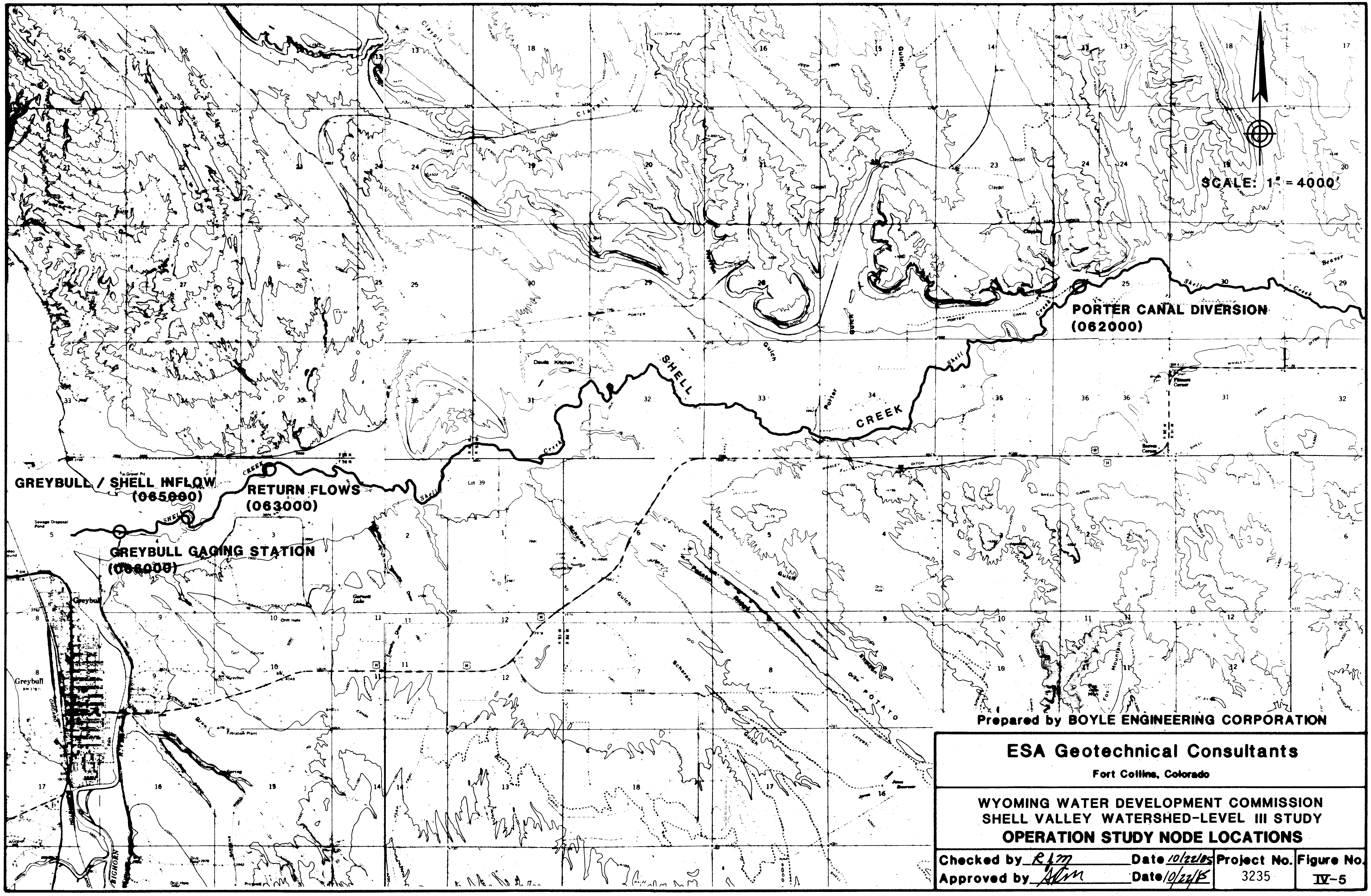
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
OPERATION STUDY NODE LOCATIONS

Checked by RJM	Date 10/22/85	Project No.	Figure No.
Approved by	Date 10/22/85	3235	IV-4



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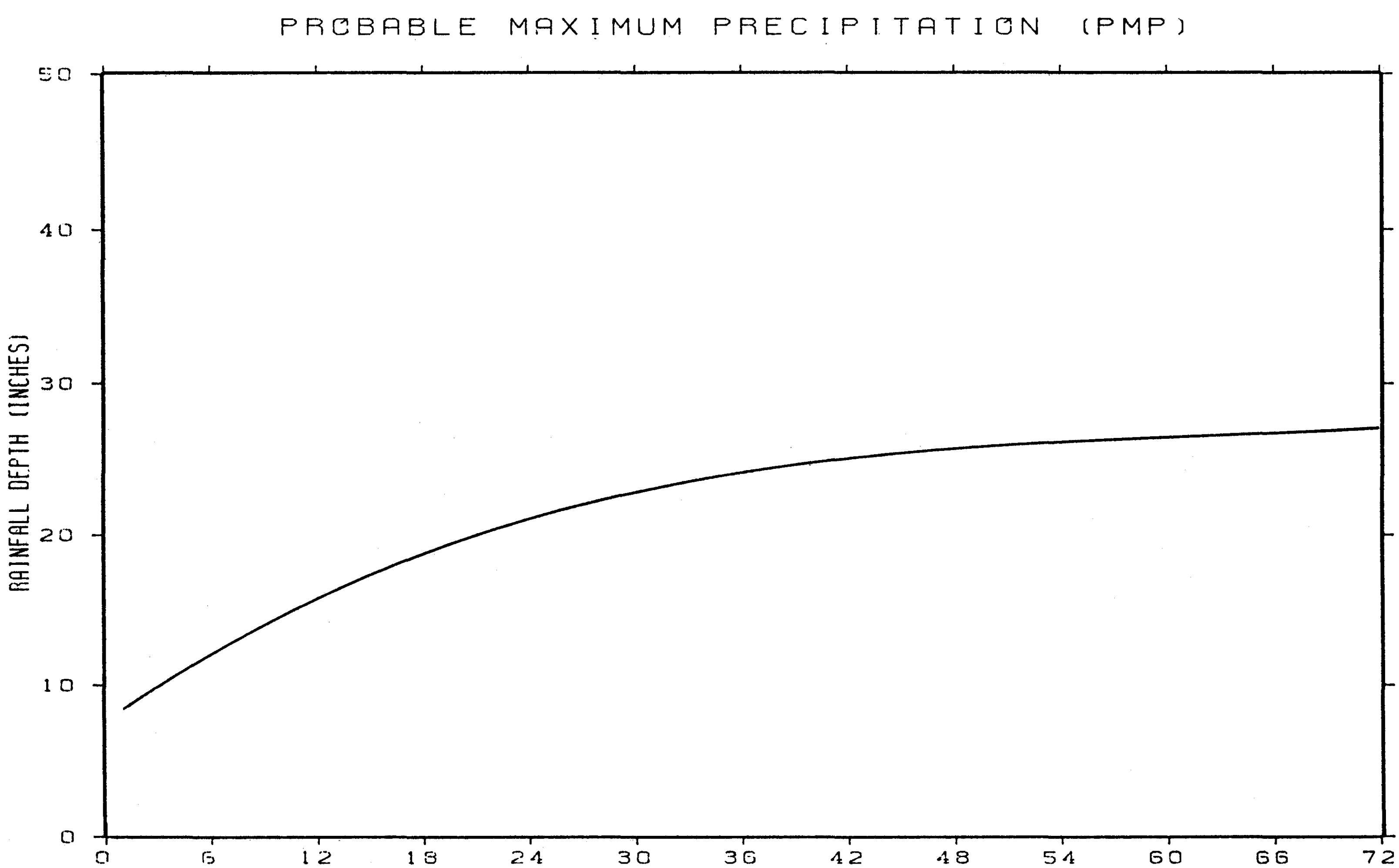
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY

OPERATION STUDY NODE LOCATIONS

Checked by RLM	Date 10/22/85	Project No.	Figure No.
Approved by [signature]	Date 10/22/85	3235	IV-5



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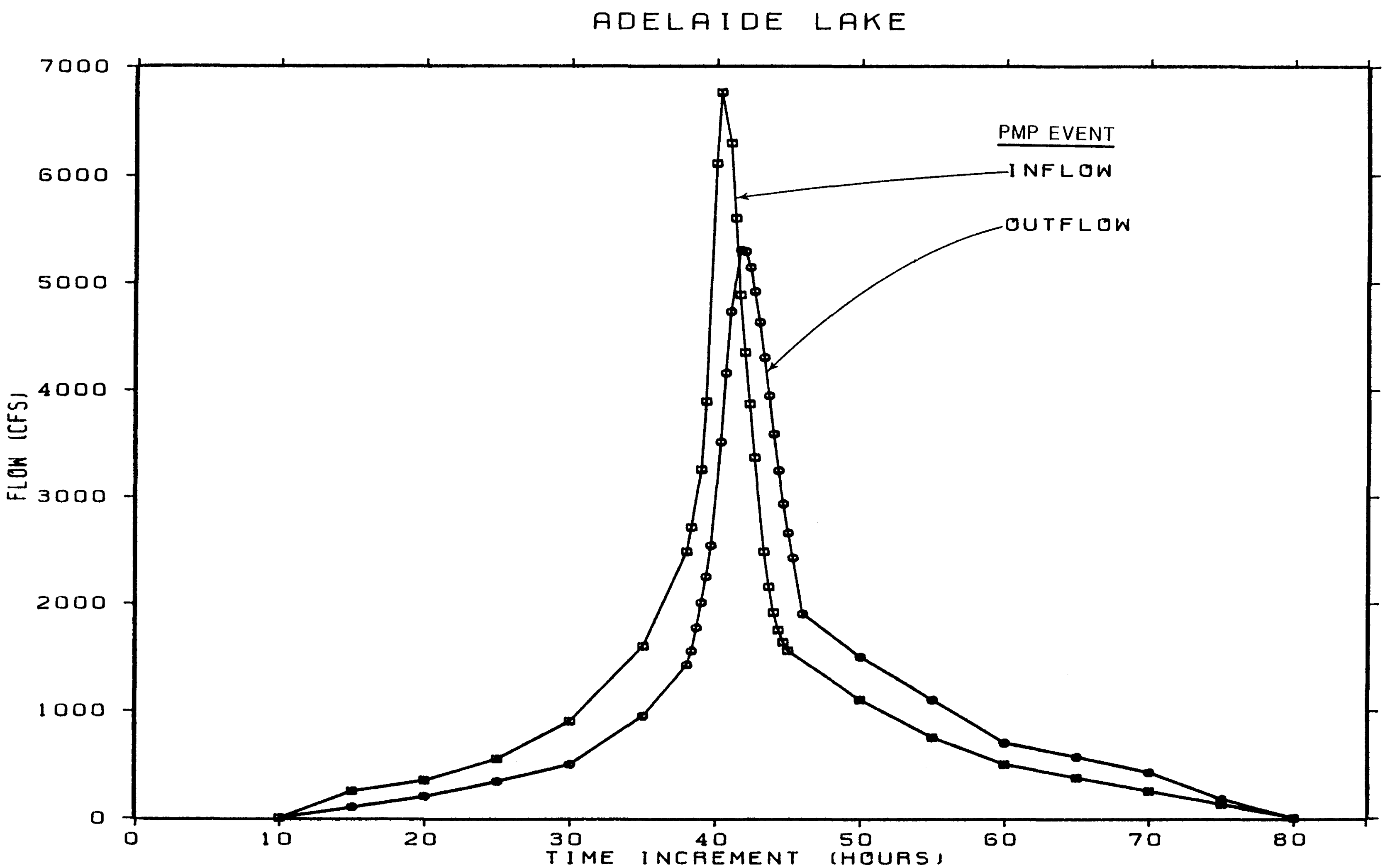
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
PROBABLE MAXIMUM PRECIPITATION

Checked by RJM	Date 10/22/85	Project No. 3235	Figure No. IV-6
Approved by RJM	Date 10/22/85		



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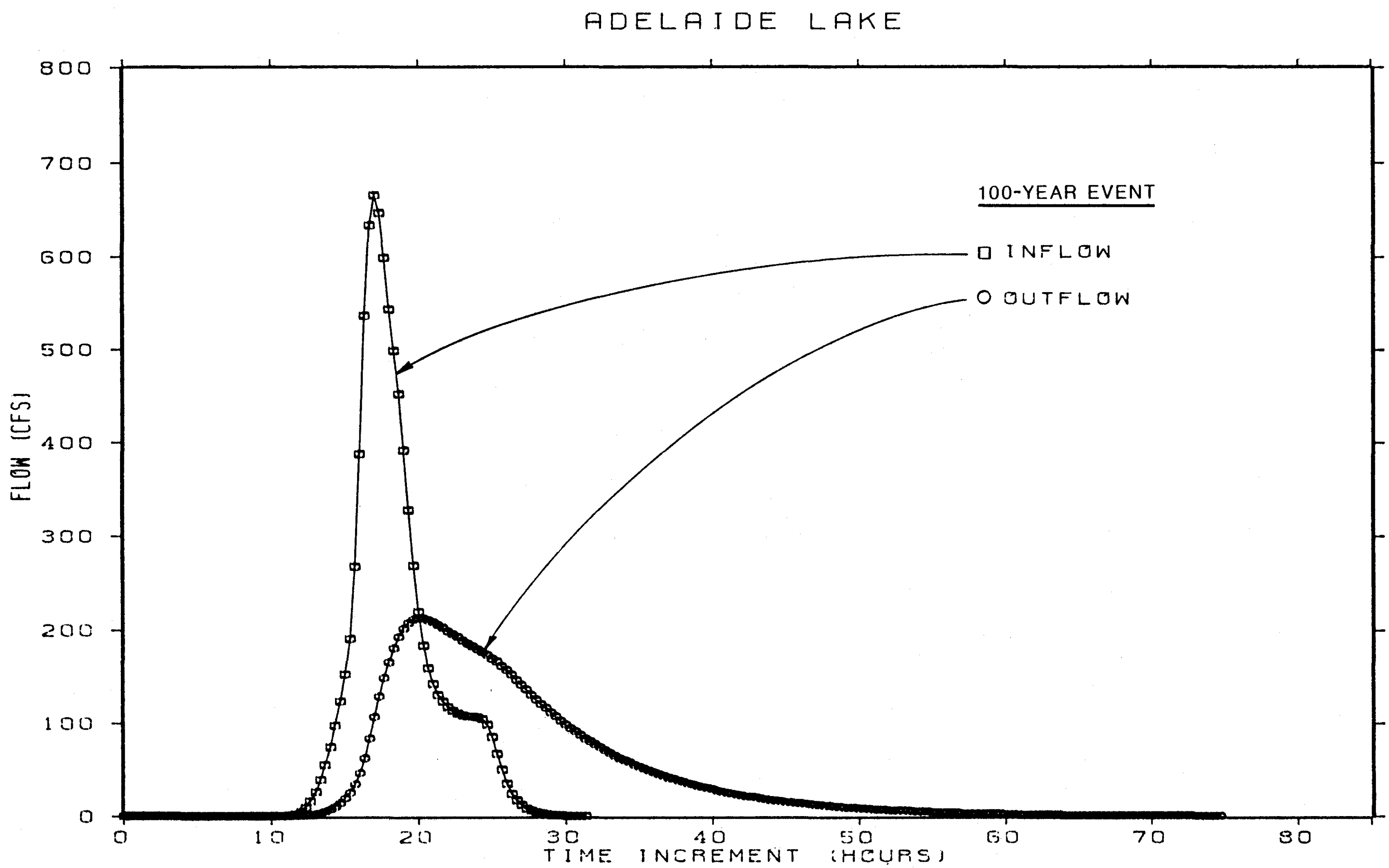
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
INFLOW/OUTFLOW HYDROGRAPHS

Checked by R/m	Date 10/22/85	Project No.	Figure No.
Approved by [illegible]	Date 10/22/85	3235	IV-7



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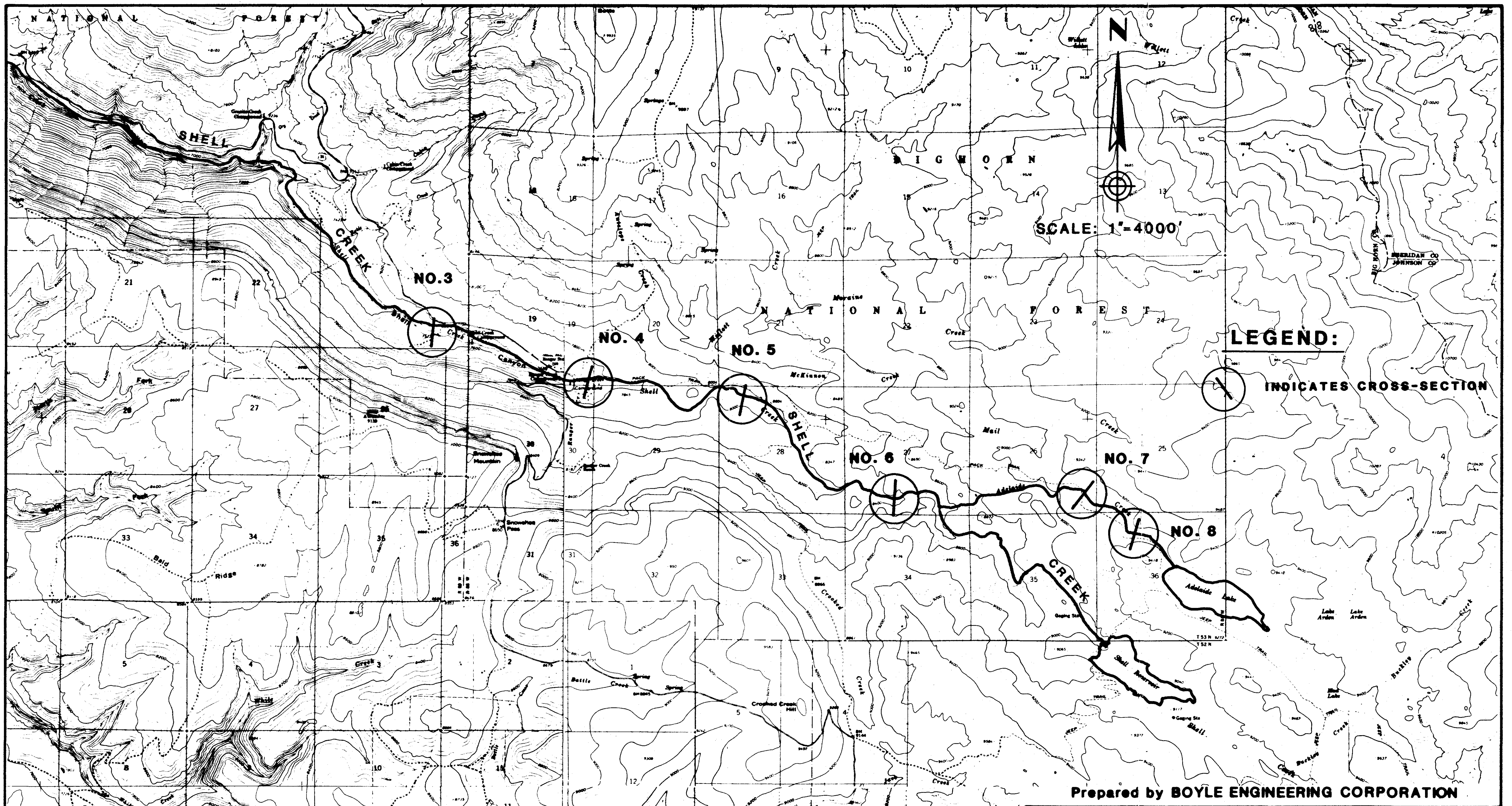
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
INFLOW/OUTFLOW HYDROGRAPHS

Checked by RLM	Date 10/22/85	Project No. 3235	Figure No. IV-8
Approved by AKim	Date 10/22/85		



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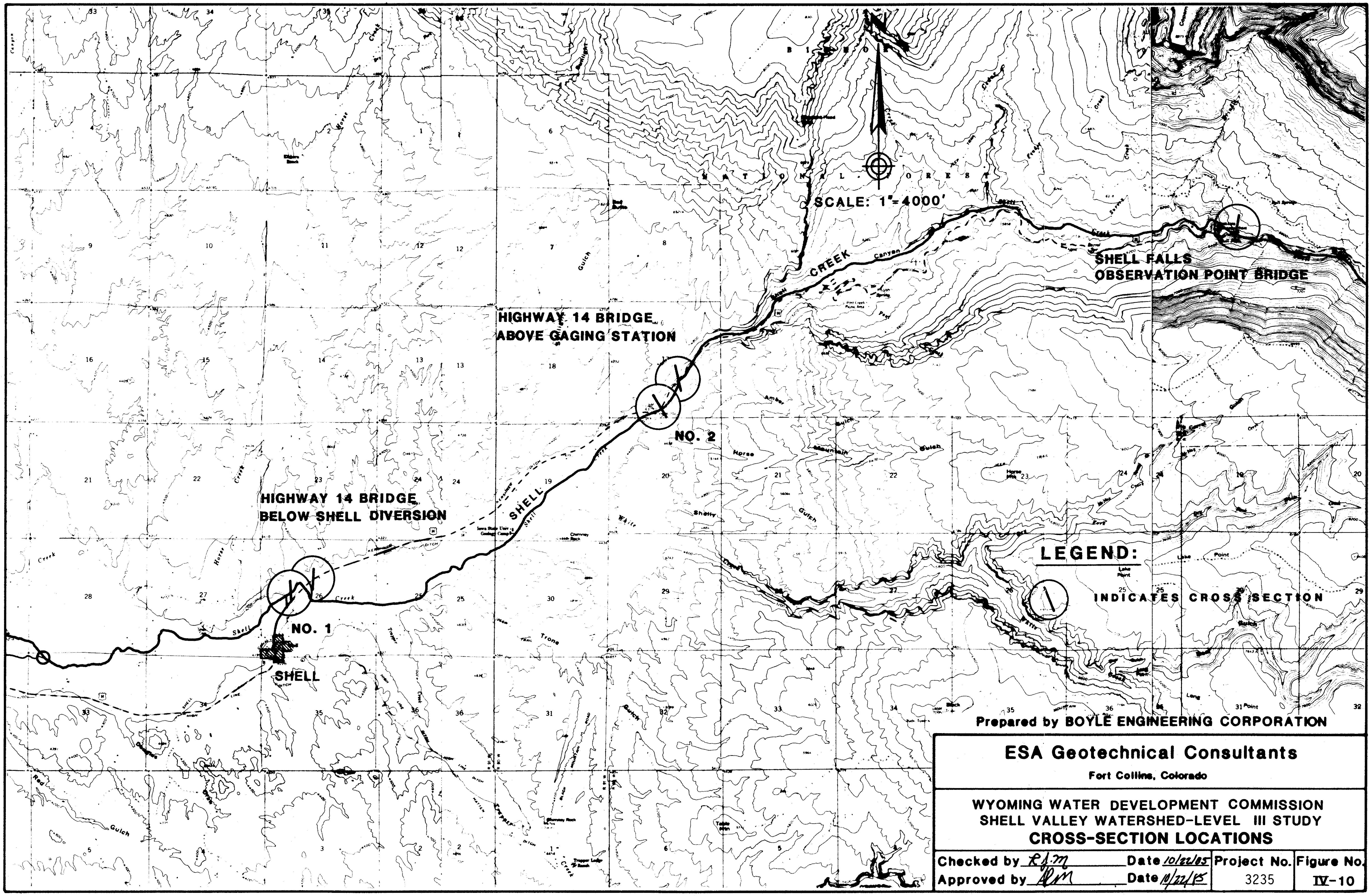
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
CROSS-SECTION LOCATIONS

Checked by RJM	Date 10/22/85	Project No.	Figure No.
Approved by HM	Date 10/22/85	3235	IV-9



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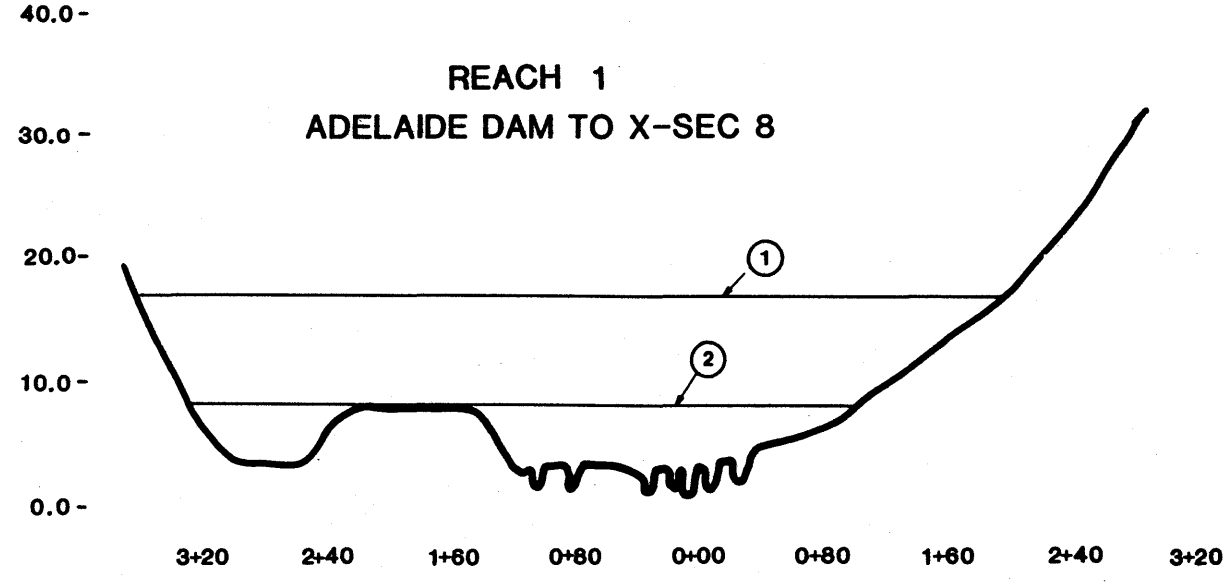
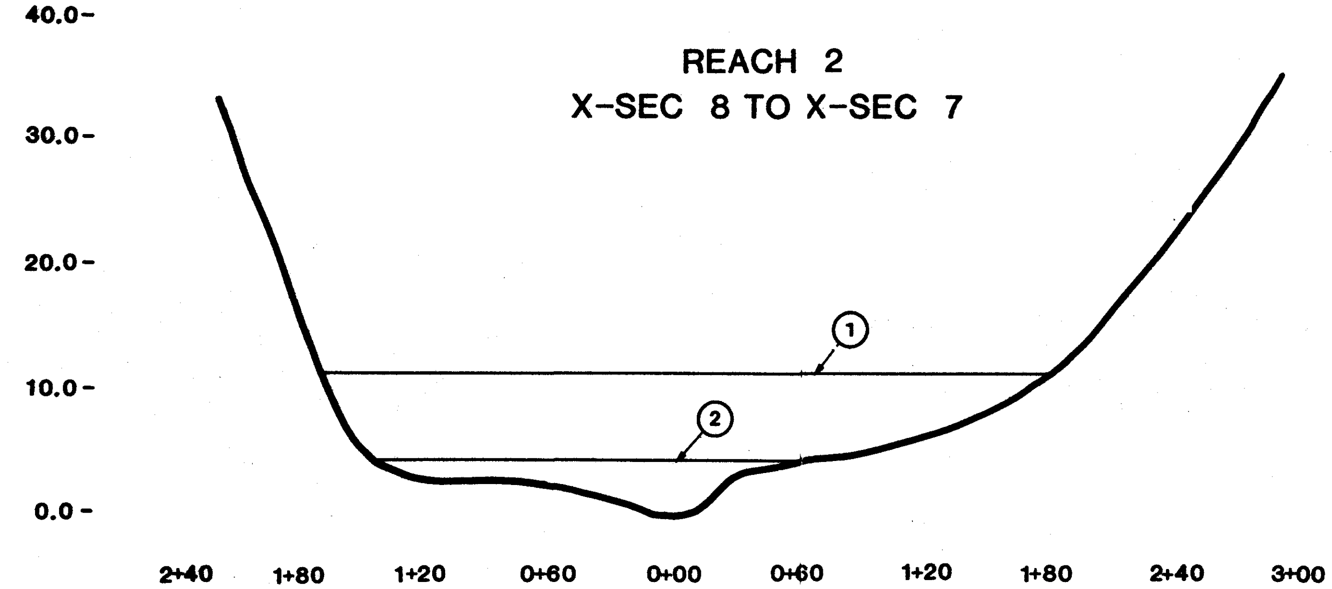
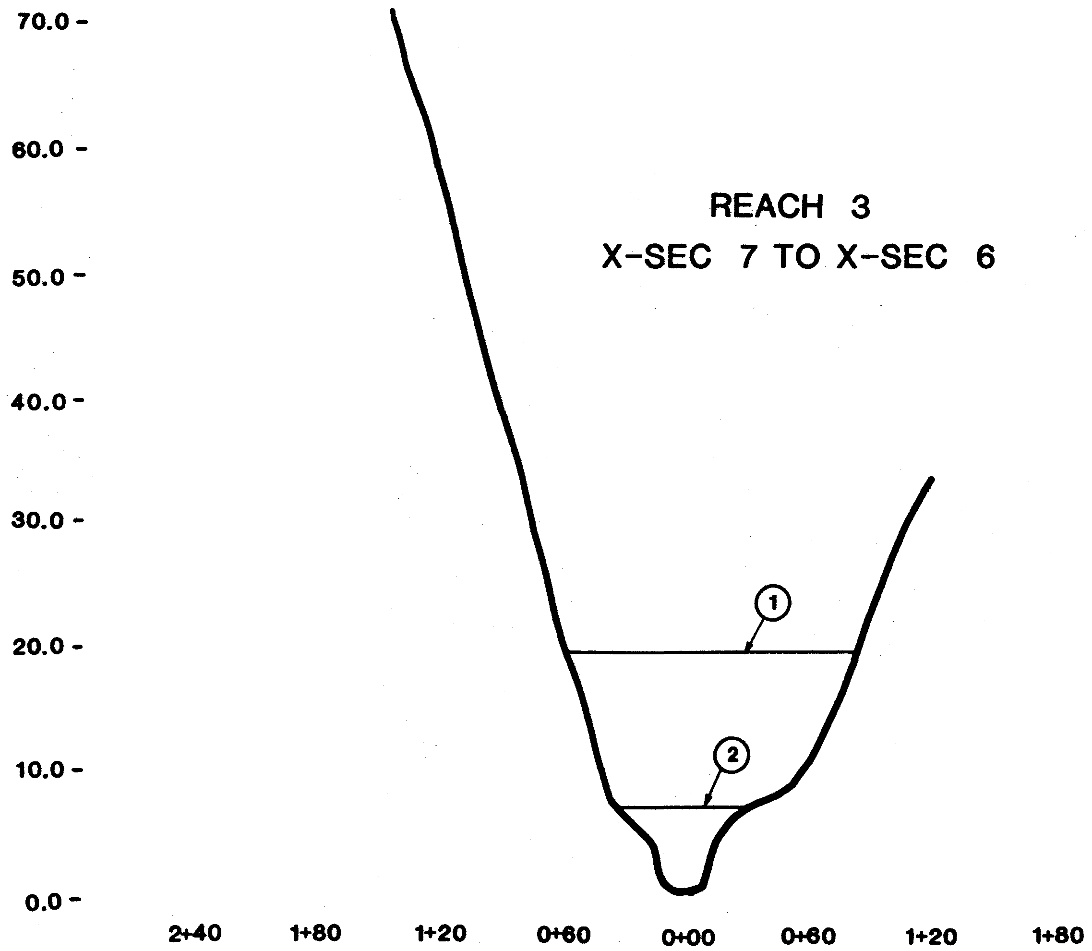
SHELL VALLEY WATERSHED-LEVEL III STUDY
CROSS-SECTION LOCATIONS

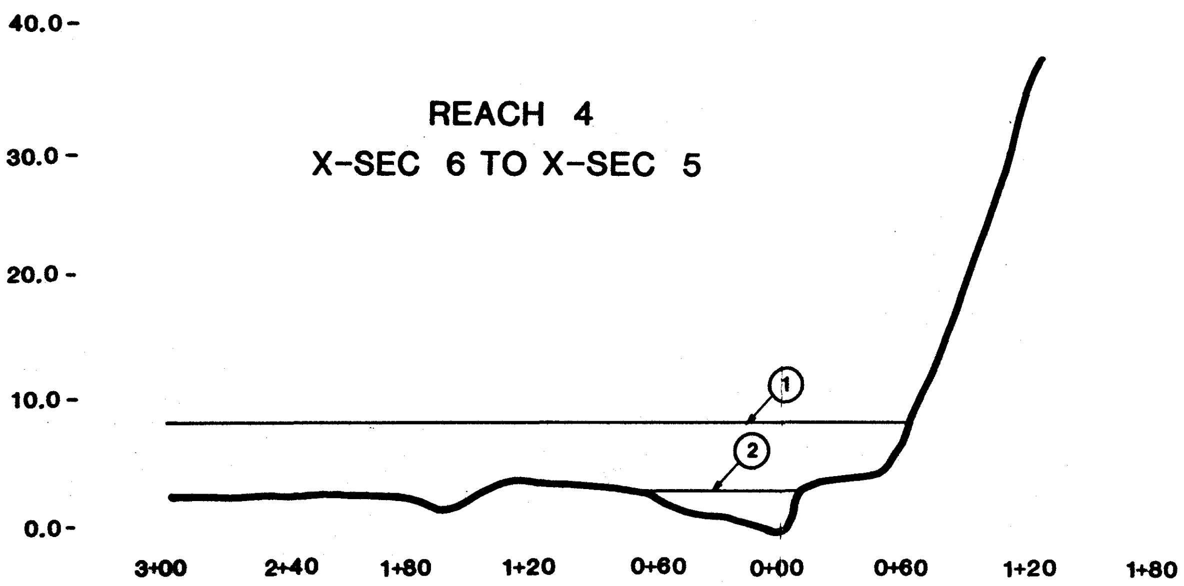
Checked by RJM	Date 10/22/85	Project No.	Figure No.
Approved by AM	Date 10/22/85	3235	IV-10

1 = 1/2 PMF SPILLWAY (DAM FAILURE)
2 = PMF SPILLWAY









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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
FLOOD PROFILES

Checked by RLM	Date 10/22/85	Project No.	Figure No.
Approved by ADM	Date 10/22/85	3235	IV-11

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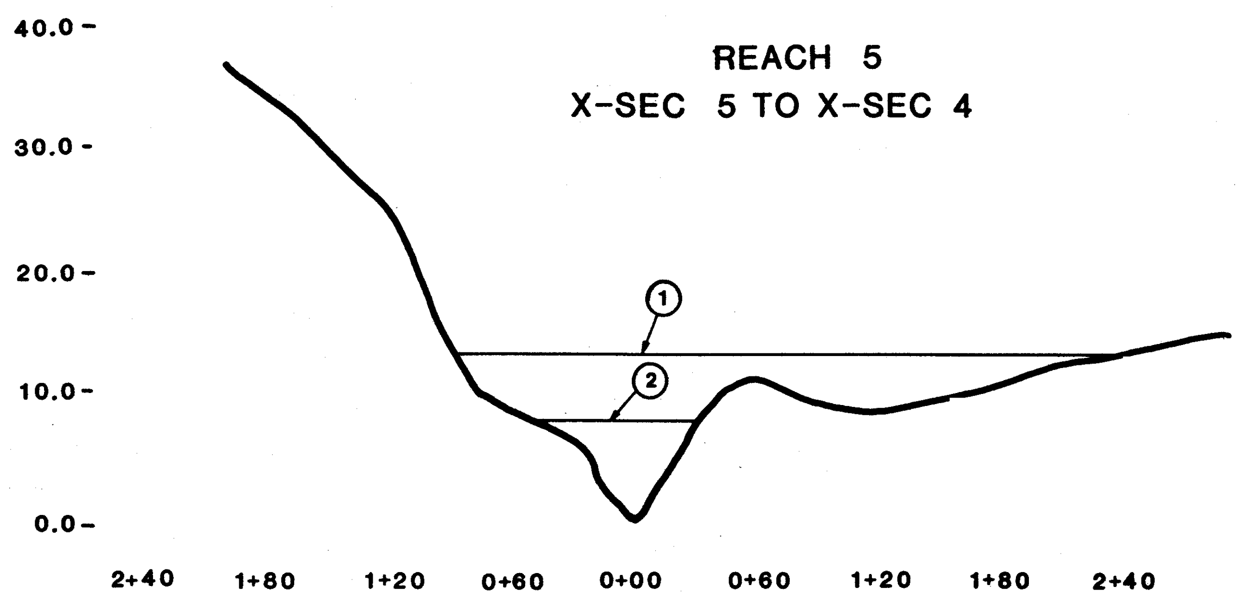
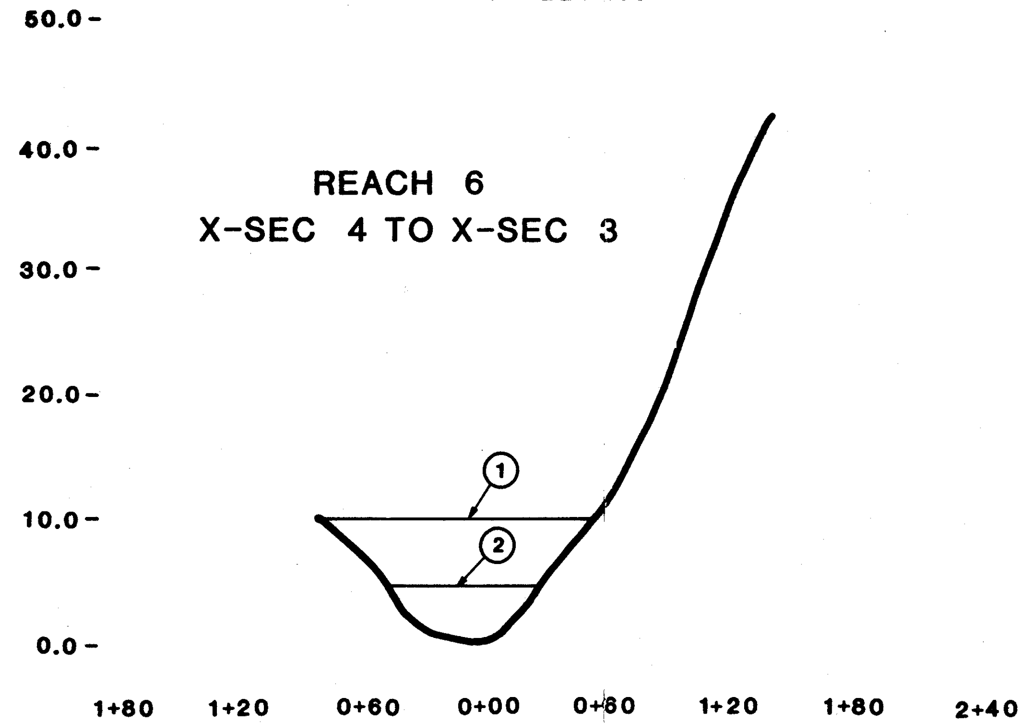
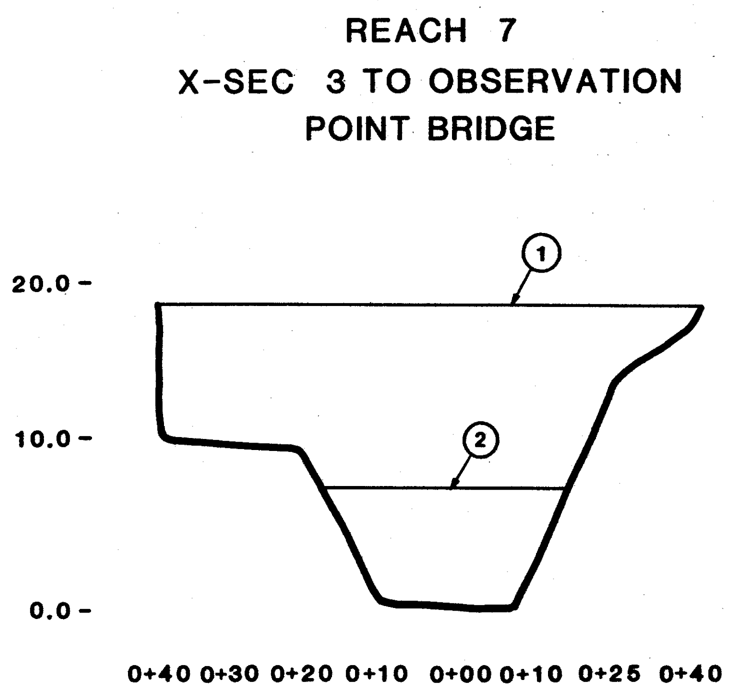
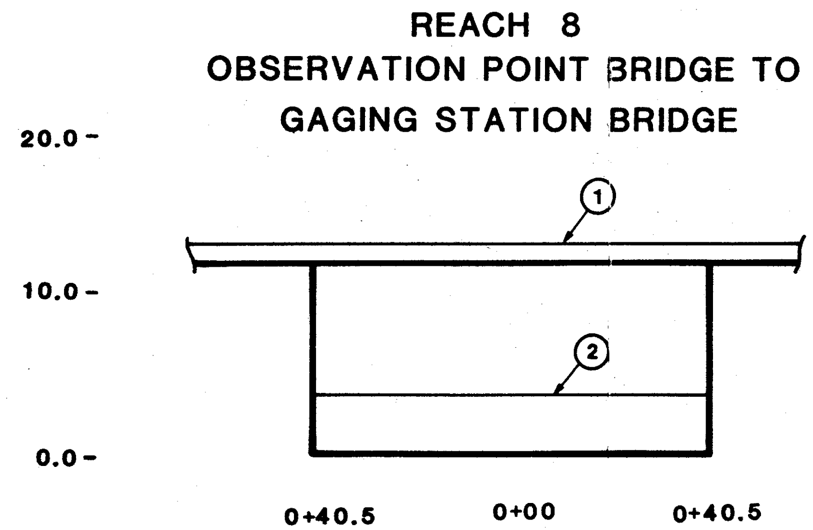
2=PMF SPILLWAY









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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY

FLOOD PROFILES

Checked by R.J.M.	Date 10/22/85	Project No.	Figure No.
Approved by ASM	Date 10/22/85	3235	IV-12

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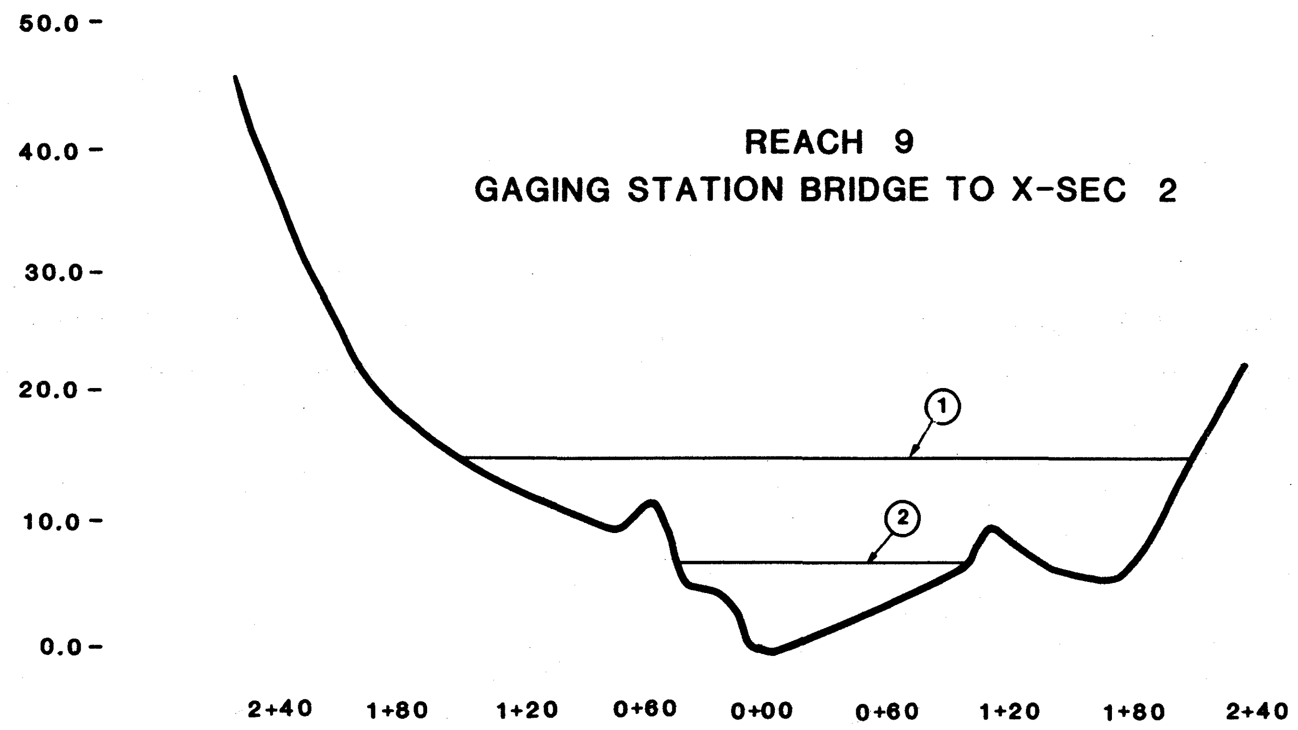

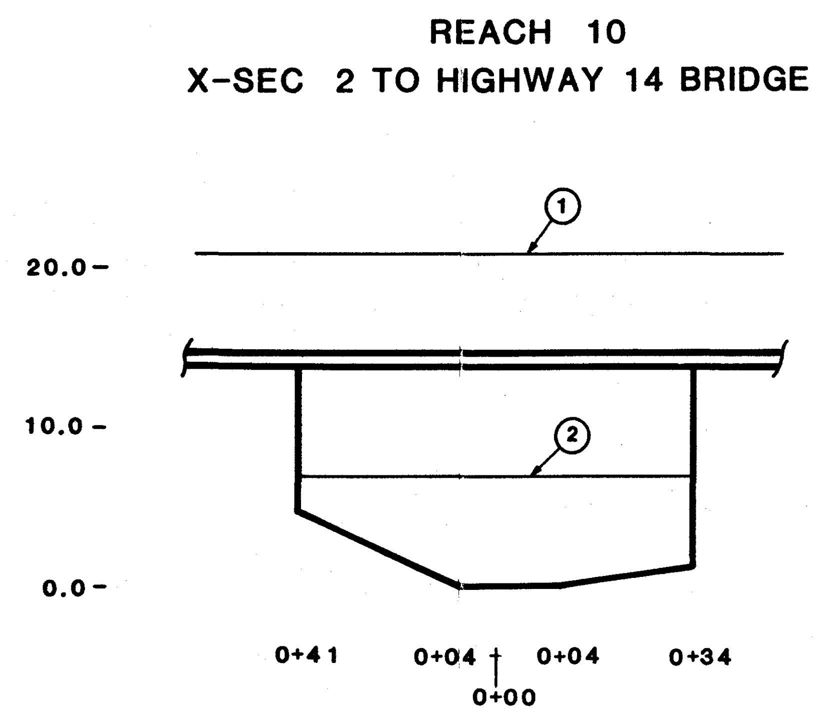

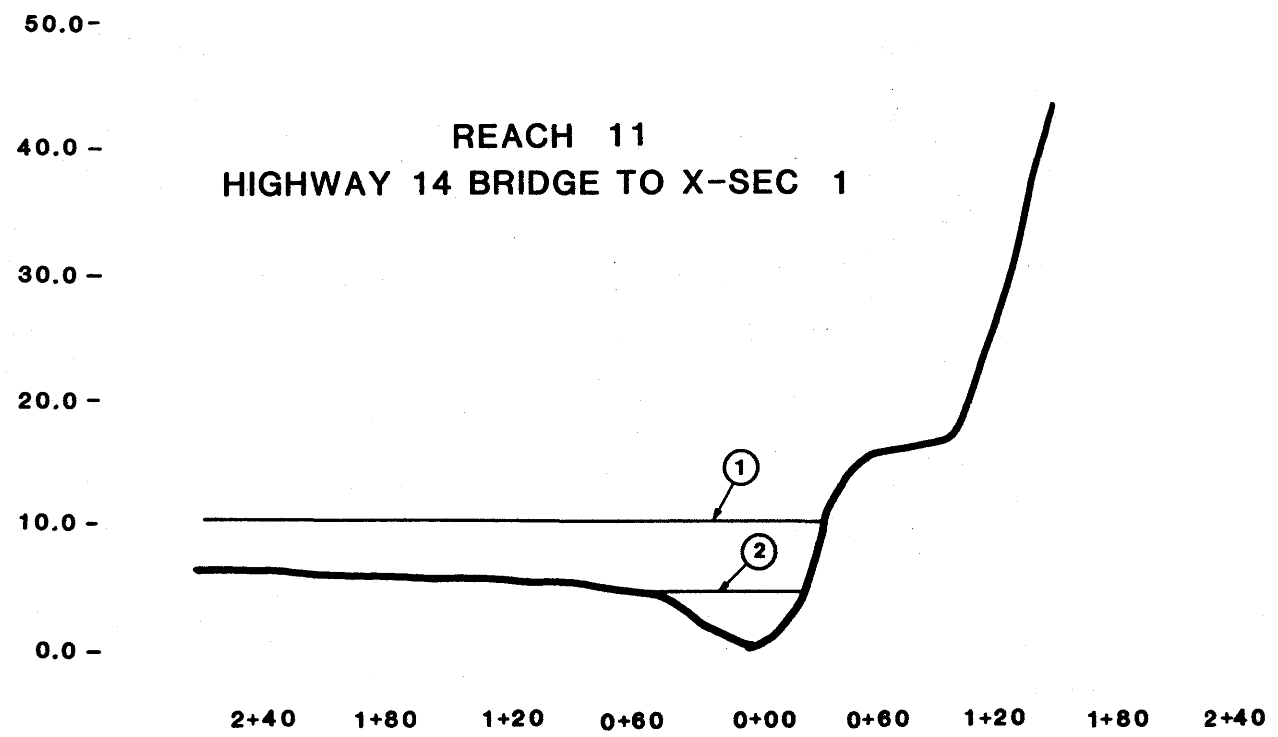

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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED-LEVEL III STUDY
FLOOD PROFILES

Checked by RJM	Date 10/22/85	Project No.	Figure No.
Approved by	Date 11/22/85	3235	IV-13

V. EMBANKMENT DESIGN

A. INTRODUCTION

The embankment design alternatives considered for the Level III studies are discussed in this chapter. Also, the engineering properties of the construction materials and the detailed analyses performed to evaluate the various designs are presented. The designs presented herein are intended to be conceptual in nature and, as such they have been carried to a level of evaluation that has enabled an assessment of their viability, safety, constructability and associated cost. The designs presented herein are not intended to represent a final design level of effort. All required construction drawings and specifications will be prepared during the Phase II portion of the Level III studies.

It should be noted that the conceptual designs presented herein are based on the results of the field and laboratory studies completed during the Level II and Level III studies. As such, the designs are based on the interpretation of limited data which requires the application of considerable engineering and geologic judgement. For dam construction in general, and earth dams in particular, the final design must be based on actual conditions encountered during construction. For example, it is only during construction when the entire foundation and abutment conditions are exposed and the true variations of borrow materials are determined. During all stages of construction, therefore, the design engineer must be prepared to evaluate new or changed conditions and be prepared to modify the final design, if necessary, in light of actual field conditions.

B. ENGINEERING PROPERTIES OF CONSTRUCTION MATERIALS

The engineering and classification characteristics of the foundation and borrow area materials were determined primarily by laboratory testing except for the field permeability tests in the foundation as discussed in Section III.E. Due to the large diameter cobbles and boulders present in both the foundation and borrow area materials, it was necessary to limit to 6-inches the maximum particle size of the samples collected for the laboratory investigation. Due to the large size, it was also not possible to obtain undisturbed samples of the foundation material.

The rationale for determining the engineering properties of the minus 6-inch fraction of the borrow materials is that this will be the maximum particle size allowed in the primary (random) zone of the embankment. Materials larger than 6-inches will be used for upstream slope protection (rip rap) and, depending on cost, may be used in a downstream rockfill zone. A complete description of the laboratory testing program is presented in Appendix B.

1. Classification of Materials

In the field of soil mechanics and earth dam design, it is advantageous to have a standard method of identifying soils and classifying them into categories or groups that have similar or distinct engineering properties. The most commonly used method at present is the Unified Soils Classification System (USCS) as described by the American Society for Testing and Materials test method ASTM D 2487. The USCS is based on recognition of the various types and significant distribution of soil constituents, considering individual grain-size magnitude, gradation characteristics, and plasticity of materials. The resulting classification of a material according to the USCS is defined by a two letter designation, the first of which defines the predominant size of the material and the second of which defines the type of gradation characteristics for coarse-grained materials or the type of plasticity for fine-grained materials. A summary of the USCS classifications of the soils tested are presented in Appendix B.

a. Foundation Materials - The results of 11 gradation tests performed on materials obtained from the foundation area are presented in Figure V-1. As can be seen on this figure, the average gradation of the near surface foundation soil indicates that about 30 percent of that material (based on dry weight) is larger than the No. 4 sieve (particle size of 4.76 mm). The No. 4 sieve size separates sand and gravel according to the USCS. The range of percentage retained on the No. 4 for the 11 samples was from 4 percent to 52 percent. Also, it can be seen that the average gradation had 11 percent of the material passing (finer than) the No. 200 sieve (particle size 0.074 mm), which is the size separating sand from silt or clay material. The average gradation characteristics of the foundation material, therefore, classify as a cobbly, gravelly, well-graded, silty sand.

b. Borrow Area Materials. In total, 18 gradation tests were performed on materials from the borrow area. For comparison purposes, the classifications have been separated for the glacial till and the underlying decomposed granite materials. The gradation results for 13 tests of the glacial till are presented in Figure V-2. These results show the till in the borrow area to be finer grained than the till within the foundation area. The average till material from the borrow area has 14 percent gravel, 61 percent sand, and 25 percent non-plastic silt to moderately plastic clay. The majority of the till is non-plastic; however, the till within the margin of the borrow area adjacent to the existing high water line tends to be more plastic. The average gradation characteristics of the glacial till within the borrow area therefore classifies as a gravelly, silty sand.

Gradation results for five tests from the decomposed granite which underlies the till in the borrow area are presented in Figure V-3. These results show that on average the material contains 32 percent gravel, 57 percent sand, and 11 percent non-plastic silty fines. In fact, the decomposed granite material within the borrow area can be generally separated into two materials with either more or less than about 5-6 percent fines. This gradational separation will be used to identify materials within the borrow area for use as a filter/drain separating foundation and embankment material from the rockfill and as a bedding material for the riprap.

2. Compaction Characteristics

The investigation of compaction characteristics in the laboratory is an attempt to define the moisture-density relationships of a given soil using a prescribed standard method of compaction. There are two basic types of compaction tests performed in the laboratory. Materials with less than about 10 percent fines densify better by vibratory energy than by impact energy. For materials with more than about 10 percent fines, however, the amount of water present within the soil at the time of compaction has a very pronounced effect on the resulting dry density that can be achieved.

For the till material present within the borrow area, impact compaction tests were performed in accordance with two compaction standards; namely, ASTM D 1557 (compactive energy equal to 56,000 ft-lb/ft^3) and ASTM D 698 (12,500 ft-lb/ft^3). The compaction test results indicate thet the majority of the gravelly, non-plastic, sands constituting the till material have a maximum dry density between 137 pcf and 141 pcf and an optimum moisture content of between 5.7 percent and 6.7 percent (based on dry weight). The clayey fraction of the till, which as stated previously is only found in a small portion of the borrow area, has a maximum dry density of 116 pcf and an optimum moisture content of 14.5 percent.

The compaction standard that will be used during construction to evaluate the adequacy of the compactive effort will be a minimum of 95 percent of the maximum dry density achieved in the laboratory using ASTM D 698. For design purposes, we have assumed that this compactive effort will result in an average total density of the random fill equal to about 140 pcf.

3. Permeability Characteristics

The permeability characteristics of the borrow area materials were evaluated on samples fabricated in the laboratory to the approximate density that similar materials will be compacted in the dam during construction. The coefficient of permeability for the non-plastic till material compacted to approximately 95 percent of the maximum dry density defined by ASTM D 698 varied between 1.1×10^{-5} cm/sec (11 ft/yr) and 4.1×10^{-7} cm/sec (0.4 ft/yr). One sample of the gravelly, clayey sand portion of the till was tested and found to have a permeabilty of 1.1×10^{-7} cm/sec (0.1 ft/yr). The cleaner fraction of the decomposed granite was also tested and found to have a permeability of about 4.9×10^{-3} cm/sec (5,020 ft/yr).

The coefficient of permeability determined in the laboratory on fabricated samples measures a value essentially in the vertical direction. For material compacted in the field, the coefficient of permeability in the horizontal direction will tend to be higher by a factor of perhaps 2 to 50 times that of the vertical permeability, depending on the percentage of fines and the degree of compaction. Based on a visual observation of the structure of samples compacted in the laboratory, we do not believe that the till material from the borrow area will exhibit a high degree of anisotropy that could affect the resulting permeability. For design purposes, we have assumed an average coefficient of permeability of 10 ft/yr and 50 ft/yr for the vertical and horizontal directions, respectively.

As discussed in Section III.E, a multiple well pumping test was performed in the maximum foundation section. The results of field pump tests and the laboratory tests were used in the seepage analyses for the three concpetual design as discussed in Section D of this chapter.

4. Shear Strength

The shear strength characteristics of the materials to be used for construction of the primary embankment zone were determined by fabricating samples in the laboratory to the approximate dry density that is expected to be achieved during construction. The samples were then saturated, consolidated to a range of pressures similar to those expected to develop within the dam, and then failed under triaxial compression. Pore pressures developed during failure were measured in order to determine both the effective and total stresses.

Shear strength parameters based on effective stresses are presented in Figure V-4. As shown thereon, the effective stress based friction angle of the compacted till was measured at 37.7 degrees and the cohesion intercept was very small. These strength results are consistent with a well compacted cohesionless gravelly silty sand. very little data scatter was observed for the six samples tested. Shear strength parameters based on total stresses are presented in Figure V-5. These results indicate that the total stress based shear strength parameters are different for the two borrow area materials tested. The total stress friction angle was measured at 22.5 degrees for samples from Trench 15 and 33.5 degrees for samples from Trench 5. The difference in total stress-based friction angles is apparently due to a slightly higher percentage of fines and compacted density for the materials from Trench 5.

The shear strength properties of the foundation materials could not be measured directly in the laboratory due to its large grain size (estimate of 40 - 50 percent greater than 6 inches). A review of the

the drill hole logs obtained during the Level II and Level III studies indicates that the minimum Standard Penetration Test result (N value) was 47. In many instances, the SPT sampler could not be advanced greater than 0.1 - 0.3 foot due to the presence of the coarse material. Based on a correlation of N value vs. Relative Density, the foundation materials classify as dense to very dense (D_r value of probably between 75 and 90 percent). Based on the inferred relative density, the foundation materials were assumed to have a minimum effective stress-based friction angle equal to 36 degrees and zero cohesion.

The shear strength parameters discussed above were used to assess the stability of the embankment under various critical loading conditions. These analyses are discussed in Section E of this chapter.

C. CONCEPTUAL DESIGN ALTERNATIVES

Three alternative designs for new dams were considered as a part of the Level III studies. The foundation and abutment preparation required for all designs is discussed in this section along with a discussion of the materials that will make up each zone within the dam. Following the discussion on zonation, each alternative design is presented.

1. Foundation Preparation

As mentioned in Section III-B, the conditions within the channel section that will form the foundation of the new dam are such that relatively limited foundation preparation will be required. Existing trees, shrubs and other deleterious material will be removed and the resulting ground surface will be leveled to remove irregular surfaces. Prior to the placement of new fill, it will be necessary to compact the foundation surface to achieve a minimum dry density equal to 95 percent of the maximum dry density defined by ASTM D 698 or a minimum relative density (D_r) of 75 percent for the minus 6-inch material fraction. The method of evaluating the resulting dry density will depend on the percentage of fines within the materials. It is intended that the principal compaction equipment to be specified for the project will be a 10-ton vibratory roller.

During foundation preparation, wet areas will be encountered in the channel section from about station 3+20 to 6+20. The extent of wet areas encountered will depend on reservoir levels during construction. However, the wet areas are concentrated in the topographic lows within the channel section. These low areas include: (1) the present discharge channel near the base of the left abutment; (2) an old channel in the right center portion of the area; and (3) the present spillway channel at the base of the right abutment. The approximate stationing along the centerline of Alternative B of these three depressions are: (1) 3+20 to 3+50; (2) 4+90 to 5+70; and (3) 6+10 to 6+25. Within these zones the foundation soils may be saturated to the surface and may have ponded or flowing water in small amounts (1-10 gpm). These areas will require drainage prior to final foundation preparation. A series of shallow open trenches 1-2 feet deep oriented in a downstream direction should be adequate to drain these areas and continue construction. It is anticipated that three trenches will suffice; one each for the three areas. The middle trench may require some laterals or fingers to cover the broader depression. A specification will be provided for backfilling these trenches prior to placing fill. A summary of the foundation preparation areas for the three design alternative is presented in Table V-1.

2. Abutment Preparation

Special consideration must be given to limiting the amount of reservoir seepage that could occur through both abutments at the embankment foundation contact with any of the design alternatives. The objective of abutment preparation during construction will be to remove loose material down to solid bedrock and then place slush grout (2 parts sand and 1 part cement), if necessary, within any voids in order to prepare a relatively smooth foundation surface prior to the placement of fill material. Since the two abutments present such different conditions, treatment procedures are discussed separately.

a. Left Abutment - Preparation and treatment within the left abutment will be limited to excavation of the till material above bedrock elevation 9250 in the abutment. This work is intended to remove the till, most likely using a dozer, working from the higher to lower elevations. The cut slope to expose the bedrock surface will be an inclination of 1.5 to 1.

TABLE V-1

Foundation Preparation Requirements

Design Alternative	Preparation Area (sq ft)
A	131,800
B	146,100
C	138,400

Notes:

1. The foundation areas include only the area within the channel section and up to Elevation 9265 on the left abutment. It does not include foundation preparation areas within the right and left abutment which will require special preparation as shown on Tables V-2 and V-3, repsectively.

2. The foundation preparation will require the removal of trees, brush and other deleterious material to a depth of up to 36 inches in isolated areas.

3. Following the clearing and grubbing, the foundation area will be leveled to remove irregular surfaces and compacted to a Relative Density (Dr) of not less than 75 percent as determined for the minus 6-inch material by ASTM Test Designation D-2048.

Once the bedrock has been exposed, slush grouting of required areas will be performed. Depending on the sequence of construction, the preparation of the foundation to receive fill may be delayed until the second construction season in order to limit the detrimental effects of frost-heave on the treated areas. This detail will be covered in the specifications. A summary of the required treatment for the left abutment area is presented in Table V-2.

b. Right Abutment - As mentioned in section III-B, the right abutment consists of a massive boulder field that will require special treatment in order to found the dam on the underlying intact granitic bedrock. Foundation preparation will require that the boulders be removed, probably by using a combination of light blasting and pre-splitting techniques.

Within the area of foundation contact, the boulders range in size up to 15 feet or more in diameter and several appear to be intact outcrops that have not yet been dislodged. Once the rock has been reduced to a maximum of 3 feet in diameter, they will be incorporated as a rock-fill at the downstream toe of the dam.

Prior to placement of the fill, it will be necessary to form a relatively smooth foundation surface by slush grouting of required areas. It is anticipated that the excavated foundation surface in the right abutment will be more irregular than in the left abutment. As a result, the specifications will require that the contractor submit a plan for how he intends to complete the right abutment treatment for approval by the engineer. A summary of the required treatment for the right abutment area is presented in Table V-3.

For both abutment areas, after the surface has been treated using slush grouting, the initial lift of fill will be selected to include the more plastic (less pervious) materials of the borrow area. This material will tend to limit the amount of seepage occurring along the embankment-foundation contact.

3. Embankment Zones

The three conceptual designs have common materials and construction requirements associated with the different embankment zones that are discussed below.

TABLE V-2

LEFT ABUTMENT TREATMENT REQUIREMENTS

Design Alternative	Treatment Area (sq. ft.)	Excavation Quantity (cu. yd.)	Surface Preparation (cu. yd.)
A	10,700	4,800	20
B	15,000	6,600	28
C	13,300	5,900	25

Notes:

1. The treatment requirements for the left abutment area consist initially of removal of trees, brush and other deleterious material. The area of required treatment is noted for each alternative under Treatment Area.

2. Following removal of organic materials, all glacial till within the left abutment above bedrock Elevation 9250 will be excavated and hauled to the borrow area. The maximum depth of cut is estimated to be about 18 feet. Required cut slopes have been assumed to be limited to an inclination of 1 1/2 to 1. The estimated quantity of required excavation for each alternative design is shown hereon.

3. Following excavation of the till, the exposed bedrock surface may require treatment prior to the placement of embankment material. It has been assumed that only 10% of the exposed surface (treatment area) will require slush grouting of an average thickness of 6 inches.

TABLE V-3

RIGHT ABUTMENT TREATMENT REQUIREMENTS

Design Alternative	Treatment Area (sq. ft.)	Excavation Quantity (cu. yd.)	Surface Preparation (cu. yd.)
A	28,700	16,000	175
B	36,900	20,500	225
C	33,400	18,600	205

Notes:

1. The primary treatment requirement in the right abutment is to remove the existing rocks down to sound granitic bedrock. The maximum depth of cut is estimated to be about 20-25 feet.

2. It is anticipated that the contractor will use a combination of light blasting and splitting to reduce the rock to a maximum allowable size of 3 feet for use in the downstream rock toe.

3. Following removal of the rock, the exposed bedrock surface may require treatment prior to placement of embankment material. It has been assumed that 1/3 of the exposed surface (treatment area) will require slush grouting equal to an average thickness of 6 inches.

a. Zone 1-Random Fill - The random fill material will be derived primarily from excavation in the auxiliary spillway area, if required, and from the borrow area. Also, material from the required excavations within the left and right abutment will be allowed in Zone 1 if it meets the following minimum requirements.

Zone 1 material will be limited to 6-inch maximum particle size. In order to limit the possibility of developing too pervious an upstream shell, the specification for the Zone 1 material to be placed upstream of the dam axis will be limited to those materials with a minimum of 12 percent fines (i.e., material finer than No. 200 sieve). This limitation is intended to preclude the use of the cleaner fraction of the decomposed granite in Zone 1.

The maximum size limitation of 6-inches for Zone 1 is to facilitate compaction of the zone. It is not intended to allow oversize to be brought to the fill and then raked off to the edges. Rather, it will be necessary for the contractor to set up a grizzly to remove the oversize material in the borrow area.

As mentioned in the previous section, the compaction specification that will be used to ensure adequate compaction of the Zone 1 material will be 95 percent of the maximum dry density obtained in accordance with the ASTM D 698 compaction standard. It is also intended that the specifications will allow the moisture content of the fill to deviate plus or minus 1.5 percent of the optimum moisture content as defined by the above standard. The moisture content of the borrow area materials during the time that samples were obtained in July and August were within plus or minus 0.5 percent of the range of optimum moisture contents determined from the previously referenced compaction tests. As the borrow area is developed, however, it is possible that it will tend to dry out. A water truck will be necessary during placement of Zone 1 material to apply additional water in the event of moisture deficiencies.

It is also intended to allow use of the existing embankment material above Elevation 9240 for the Zone 1 material. This material source is designated as Zone 5 and is discussed in more detail later in this section.

b. Zone 2-Filter - A filter material will be necessary to protect against possible migration of embankment and foundation materials into the rockfill section. The principal requirements for this material are that it be limited to 3-inch maximum particle size and not more than 8 percent fines. These specifications were designed specifically to enable the use of the cleaner fraction of the decomposed granite material in the borrow area without any special processing except the removal of oversize material. It is estimated that materials meeting the Zone 2 specifications will have a minimum coefficient of permeability equal to 3000 feet per year.

The compaction requirement of the Zone 2 material will be a minimum dry density equal to 95 percent of the maximum dry density achieved by ASTM-D 2048. The resulting dry density will be equal to a relative density of about 75 or 80 percent.

c. Zone 3 -Rockfill - The Zone 3 rockfill is intended to receive the oversize material from the required excavation in the right abutment area. The use of this material will be mandatory for all the alternative configurations. The rockfill will be limited to a maximum rock size of 36-inches. Also, although not shown on the drawings, a 2-foot thick zone will be included in the specifications that will provide a secondary transition at the base of the rockfill. This material will be limited to 12-inch maximum size and have a minimum of 20 percent finer than 1.5-inch.

Due to the large rock size allowed in this zone, it will be difficult to compact the rockfill unless some minus 6-inch material is incorporated into the top of each lift, which will be limited to 3-feet. It is anticipated that the contractor will incorporate such material from the decomposed granite source in the borrow. The need to incorporate minus 6-inch material will probably only be required if Alternative B is selected, since it is anticipated that rock material from the required excavation in the right abutment will not be lacking the gravel size fraction required for equipment access and compaction of the rockfill.

d. Zone 4-Riprap Slope Protection - The upstream slope of the dam and other elements of the project including the auxiliary spillway, the plunge pool area of the side channel spillway and the Buckley Creek Diversion structure, will need to be protected against the erosive forces of flowing water by the use of large rock (riprap) slope protection. The riprap required for the upstream face of a dam is usually designed on the basis of the wave height expected to occur within the reservoir during sustained high wind conditions.

One of the best approaches to the design of a riprap section is to evaluate comparable existing structures. For the case of Lake Adelaide Dam, the riprap section on the existing dam affords an excellent opportunity to evaluate a riprap section that has been in use for about 70 years and appears to be in excellent condition. The existing riprap facing appears to be about 12-14 inches thick (measured normal to surface), contains a maximum particle size of 12-inches, a median particle size of 6-inches, and is not underlain by a bedding layer.

The maximum wave height anticipated within the reservoir is 3-feet based on an effective fetch of less than 1 mile and sustained winds of 75 mph. Based on recent work completed at Colorado State University (Nelson, et al., 1983), the required median size of the rock is estimated to be 12 inches and the required thickness of the protective layer is 18-inches. Both of these numbers are slightly larger than the existing slope protection layer and appear reasonably conservative. It was also decided to provide a 6-inch thick bedding for the riprap since the upstream zone of the new dam will, most likely, be finer grained than the existing upstream zone. The cleaner portion of the decomposed granite will provide an excellent filter and bedding for the riprap design mentioned above.

e. Zone 5-Existing Dam - The existing dam will be used as a borrow source, assuming that the reservoir can be drawn down, and that the materials are effectively dewatered prior to their use. The materials should provide a very reasonably priced source for Zone 1 material since they are located so close to the construction site and, most likely, contain a very small percentage (if any) of oversize material. Compaction control for these materials will be the same as for Zone 1.

The potential cost savings of using the Zone 5 material is discussed in Section VII, Construction Cost Estimates.

5. Embankment Design Alternatives

The following discussion presents the design objectives associated with constructing a dam to safely and effectively retain the reservoir at some preferred new storage level. The design of a dam is evolutionary in nature as the results of field and laboratory investigations become available, and geologic and hydrologic analyses and assessments are completed. A similar process is followed for the hydraulic structures associated with each alternative design. The following discussion, however, is limited to the embankment alternatives. The design alternatives associated with the different hydraulic structures considered during this study are presented in the following chapter of the report.

A summary of the embankment design alternatives evaluated during this study is presented on Table V-4.

TABLE V-4

SUMMARY OF DESIGN ALTERNATIVES

Alternative	Crest Elevation	Design Option/Objective
A	9280	Modification of dam proposed in Level II study.
B	9287	New dam conceived to retain significantly larger reservoir than Alternative A. Also, designed to incorporate all materials to be encountered within the borrow area.
C	9287	Same as Alternative B but provide option to not require large rockfill zone.

The various embankment alternatives are discussed below.

a. Alternative A - This design represents a similar structure to that proposed during the Level II study with the following exceptions.

1. As discussed in Chapter II, after reviewing the results of the field investigation performed during the Level II studies and completing preliminary settlement analyses, it was de-

cided that the new dam must be moved far enough downstream to eliminate the need of structural support from the existing dam.

2. Preliminary seepage analyses indicated that a major cutoff trench, or some other seepage barrier, would not be required.

3. Since the boulder field in the right abutment would have to be excavated in order to properly found the dam on bedrock, it was decided to consider this volume of rock as mandatory excavation. For all alternatives, therefore, the material from this excavation will be incorporated as a downstream rock toe.

Using the concepts described above, a maximum cross section through the dam was developed and is presented in Figure V-6. The cross section shown for Alternative A was evaluated for its potential seepage losses. As a result of the seepage analyses, which are discussed in Section D of this chapter, it was discovered that the inclined drain shown on Figure V-6 was not necessary. The reason for this is directly related to the effectiveness of the pervious foundation acting as a drain. It was decided therefore to eliminate the inclined drain from the design and it was not included in the construction cost estimate.

b. <u>Alternative B</u> - A cross section depicting this design alternative is presented in Figure V-7. After the hydrologic studies were finished and it was apparent that a reservoir yield of approximately 4100 acre-feet could be reliably developed, a new dam configuration was considered that would raise the crest 7 feet, from Elevation 9280 to Elevation 9287.

As mentioned previously, the results of the field investigation indicate that a substantial quantity of plus 6-inch size material would be developed during the processing of the borrow area. Design Alternative B represents a balanced design in that a major downstream rockfill section is included to accommodate the oversize material that would be produced from the borrow area and other areas of required excavation.

c. Alternative C - The third alternative design evaluated as a part of the Level III studies considered the possibility that the cost of incorporating the rock as described for Alternative B may be too expensive. This alternative design is similar to Alternative A except that it is a larger dam, and has a steeper upstream slope. Alternative C is the same height as B and is adequate to expand the reservoir to its optimum size. A cross section for Alternative C is presented in Figure V-8.

The computed volumes for each zone associated with the three design alternatives discussed above are presented on Table V-5. These volumes, along with the associated foundation cleanup and required excavations, formed the basis for the construction cost estimates associated with embankment construction.

TABLE V-5

COMPUTED EMBANKMENT VOLUMES

Design Alternative	Embankment Volumes (cu. yd.)					Total
	Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	
A	140,600	3,500	16,000	3,300	34,000	163,400
B	128,600	6,000	58,400	4,600	34,000	197,600
C	172,600	4,000	18,600	3,300	34,000	198,500

Notes:

1. The volumes shown for each zone represents compacted in-place volumes. No shrinkage or expansion factors have been considered in representing embankment volumes.

2. The volume of Zone 2 filter required for Alternative A does not include the inclined filter depicted on Figure V-6 since seepage analyses indicated that such a zone was not required.

3. The Zone 5 material represents the volume that can be obtained from the existing dam with excavation above Elevation 9240. This volume however has not been included in the total volume since its possible use as a borrow source will be determined by the reservoir operations during construction.

D. SEEPAGE ANALYSIS

1. Introduction

One of the major concerns identified in the Level II study of Adelaide Dam was the apparent high seepage loss thought to be occurring through the foundation of the existing dam (HKM Associates, 1985). Based on results of in-situ constant head tests conducted in that investigation, permeability coefficients ranging from 7500 to 55,000 feet per year were used in the seepage analysis. The results of the Level II analysis indicate approximate seepage losses of up to 2900 acre-feet per year through the foundation materials.

The results of this investigation indicated significantly lower permeability coefficients for the dam foundation, and correspondingly reduced estimated seepage losses. In-situ pump test results, described in Section III.E, provided estimates of permeability coefficients ranging from approximately 300 to 1600 feet per year in the vicinity of the maximum section of the proposed dam. Using an average value of 1200 feet per year, seepage analyses were performed for the three alternative design configurations described in the previous section. The results of these analyses indicate a maximum seepage loss through the foundation and new embankment of less than 250 acre-feet per year. This maximum value results from using the most conservative assumptions regarding permeability coefficients, total available head under full reservoir all year, and maximum cross sectional area of seepage.

2. Layouts and Assumptions

Seepage analyses were performed for three different design configurations; Alternative A (Figure V-6), Alternative B (Figure V-7) and Alternative C (Figure V-8). Design Alternative A has a crest elevation of 9280 feet and maximum normal pool elevation equal to 9273 feet. Because the inclined drain configuration in the original Alternative A design has been abandoned for reasons discussed in the preceding section, detailed results of the seepage analyses for that configuration are not presented, although total predicted discharges for Alternative A are summarized later in this section. Each of the design Alternatives B and C has a crest elevation of 9287 feet with maximum normal reservoir elevation equal to 9280 feet. Alternative B has a 3:1 upstream slope with a large rockfill section, whereas Alternative C has a 2.5:1 upstream slope with a small rock toe. These differences affect the location and shape of the seepage line or phreatic surface through the new embankment, and the total amount of predicted annual discharges.

The following paragraphs describe the physical layouts and assumptions that were made in modelling the proposed raised dam and existing foundation conditions for seepage analysis:

1. All design alternatives have three distinct zones of material with respect to permeability characteristics, as follows: (1) an earthfill section composed of compacted silty sand, (2) a downstream rockfill toe and, (3) filter/drain zones which act as transition zones between the rockfill and embankment or foundation materials.

2. The existing dam was modelled as an upstream blanket or berm. It was assumed the existing structure was excavated down to the elevation where the upstream toe of the proposed dam would intersect the downstream face of the old dam (Elevation 9240 feet).

3. The glacial till foundation was assumed to have a constant 40 foot depth above bedrock in extension well upstream and downstream from the maximum section.

4. All materials were assumed to have isotropic permeability characteristics except for the earthfill section of the proposed dam. An anisotropy ratio of 5 to 1 for the horizontal to vertical permeability coefficients was assumed for the new earth fill section. Permeability values used in the seepage analyses for all of the embankment and foundation materials are summarized in the individual figures which show the results.

5. Two different assumptions were made regarding the permeability of the foundation till. For conservative analyses, the foundation was assumed isotropic and homogeneous with respect to permeability. The entire depth of till was

assumed to have a constant permeability equal to 1200 feet per year. However, the results of the field investigation suggested that the lower portion of the till material contained a higher percentage of large boulders compared with the upper portion. For the less conservative analysis, it was assumed that a large percentage of the total seepage area is blocked by these large boulders in the lower half of the foundation, thus reducing the effective permeability in that zone. A two layer foundation having a permeability of 1200 feet per year in the upper half and 100 feet per year in the lower half was modelled for the Alternative B and C configurations.

6. All seepage analyses were based on maximum reservoir elevation conditions, assumed to act year round at the maximum section of the embankment (maximum available head conditions).

3. <u>Methodology and Results</u>

a. <u>Flow Net Sketch</u> - A preliminary seepage analysis was performed by drawing a flow net through the foundation for the maximum section of the Alternative B design configuration as shown in Figure V-9. For purposes of drawing the flow net, it was assumed that the permeability of the new embankment was sufficiently less than the foundation and existing dam to be considered impervious by comparison. Also, the existing embankment, or upstream blanket, was assumed to have the same permeability as the foundation (1200 feet per year). Based on the flow net shown in Figure V-9, a seepage discharge of 14,400 $ft.^3/yr.$ (0.33 acre-ft./yr.) per foot of seepage area (length perpendicular to flow net section) was computed. Since the flow net was drawn for the maximum section, and therefore the maximum seepage area in two dimensions, the equivalent length of the seepage area was approximated by proportioning the length of the reservoir water line at maximum pool (730 ft.). The equivalent length was found by computing the ratio of the rectangular area equal to the maximum water line length times the maximum foundation depth (730 ft. x 40 ft. = 29,200 $ft.^2$) to the actual seepage area determined by the centerline section shown in Figure III-2 (20,567 $ft.^2$). The area porportionality ratio was thus 20,567 $ft.^2$/29,200 $ft.^2$ = 0.704. The equivalent length of seepage area was thus 0.704 x 730 ft. = 514 ft., or about 515 ft. As shown in Figure V-9, the total seepage from the flow net calculations was approximately 170 acre-feet per year.

b. Finite Element Method Modelling - Seepage analyses for each alternative design configuration were also performed using finite element method (FEM) modelling. The FEM analyses were performed on a Cyber 825 mainframe computer at Colorado State University in Fort Collins. The model used in this analysis was CFLOW3 (Edgar, 1979), which is a modified version of the CFLOW and FPM500 codes developed by the U.S. Bureau of Mines (1976). CFLOW3 provides solutions to Poisson's equation, the partial differential equation which describes steady state flow through a porous media. The model solves for the location of the phreatic surface in unconfined flow problems using an algorithm originally developed at the University of California, Berkeley, by Taylor and Brown (1967). The program output consists of pressure head and potential at each node, and "Darcy" velocity (the product of permeability and hydraulic gradient) at the center of each element of the finite element mesh.

Three meshes were drawn for the FEM analyses, one for each of the alternative design configurations. Several preliminary runs were performed to evaluate the model and to determine optimum error criteria and underrelaxation factors for locating the phreatic surfaces. Assumptions regarding input parameters such as maximum reservoir elevation, tailwater elevation and permeability coefficients for each material feature are designated in Figures V-11 through V-13. Appropriate notes summarizing the assumptions and methodology related to the FEM analysis are presented in Figure V-10.

The results for three final runs; two for Alternative B and one for Alternative C; are presented in the form of equipotential diagrams in Figures V-11 through V-13. Figures V-11 and V-12 illustrate the location of the phreatic surface and rate of head loss for the Alternative B design configuration under the two different foundation conditions discussed previously. A comparison between these results for Alternative B shows that a reduction in permeability in the lower half of the foundation has little effect on either the location of the phreatic surface or the rate of head loss. However, the total amount of predicted seepage for the two layer model (Figure V-12) is significantly less than the total discharge for the single layer model (Figure V-11). The results for total seepage are summarized in Table V-6.

Figure V-13 shows the seepage analysis results for Alternative C. With this configuration, the phreatic surface is intercepted by the inclined filter of the smaller downstream rock toe. When compared with Figure V-11, it is seen that the phreatic surface for the Alternative C design remains at a higher elevation much further downstream than in the Alternative B design with a large rock fill section. This extended and elevated phreatic surface would result in higher pore pressures developing in the downstream section of Alternative C compared with the downstream section of Alternative B. Slope stability analyses of the downstream slope, which are discussed in the following section, indicate that the factor of safety for the Alternative C design is about 17 percent lower than for Alternative B, which in part is due to the higher location of the phreatic surface.

The equipotential diagrams show that the majority of the head loss at maximum reservoir elevation occurs within the earthfill zone of the new embankment. Figures V-11 and V-12 for the Alternative B design show that more than 80 percent of the total head is lost before the phreatic surface encounters the inclined drain. Figure V-13 shows that 90 percent of head loss occurs through the embankment earthfill for the Alternative C configuration. The greater percentage of head loss in the earthfill embankment of the Alternative C may be attributed to the longer seepage path of that design compared with the Alternative B configuration.

Total seepage quantities were estimated from the CFLOW3 results using the Darcy velocity data output. The results are summarized on Table V-6 for two methods of analysis.

Method 1, which is the more conservative analysis, consists of summing the flow across each element in a vertical column of elements to determine the total discharge across any vertical plane in the section. Using the vertical column of elements at the centerline of the maximum section of the embankment, total discharge per foot width of seepage area was determined. The total width of the discharge area

TABLE V-6

Summary of Total Seepage Analysis by FEM

	METHOD 1			METHOD 2						
				FOUNDATION AT MAX. SECTION			EMBANKMENT AT MAX. SECTION			
DESIGN ALTERNATIVE	TOTAL DISCHARGE ACROSS MAX. SECTION (ac-ft/yr/ft)	LENGTH AT MAX. RESERVOIR ELEVATION (ft)	TOTAL MAXIMUM DISCHARGE (ac-ft/yr)	AVE. VELOCITY (ft/yr)	TOTAL AREA (ft^2)	FOUNDATION DISCHARGE (ac-ft/yr)	AVE. VELOCITY (ft/yr)	TOTAL AREA (ft)	EMBANKMENT DISCHARGE (ac-ft/yr)	TOTAL MAXIMUM DISCHARGE (ac-ft/yr)
A	0.29	692	200	306	20567	145	12	22063	6	151
B (homogeneous foundation)	0.32	730	230	333	20567	157	17	27182	11	168
B (two layer foundation)	0.20	730	146	380	10284	90	17	27182	11	108
				31	10284	7				
C (homogeneous foundation)	0.26	730	190	276	20567	130	12	27182	8	138
C (two layer foundation)	0.16	730	117	294	10284	70	12	27182	8	84
				24	10284	6				

was assumed equal to the length of the upstream waterline at maximum reservoir level. No adjustments of the length of seepage area were made in this analysis, as described previously in the flow net analysis. This method of analysis assumes conservative approximations for both seepage area and unit discharge.

Method 2 provides a more reasonable approximation for total seepage quantity. Instead of calculating the discharge at a vertical plane on the maximum section, the average Darcy velocity in each material layer is determined. It is assumed that the velocity is constant across the width of the seepage area. The total area of the embankment and of the foundation was then determined based on the centerline section shown in Figure III-2. The seepage area times the average velocity of each layer at centerline gives the quantity of discharge for both the embankment and foundation.

Also, it should be noted that all of the seepage quantities reported on Table V-6 were determined based on discharge rates corresponding to maximum reservoir elevation conditions. Since the reservoir will not generally be operating at maximum level year round, an approximation was made for seepage losses corresponding with a lower reservoir level. The estimated total discharge corresponding to a reservoir elevation of 9260 feet was less than 100 acre-feet per year.

E. SLOPE STABILITY ANALYSES

1. General Methodology and Assumptions

All analyses for evaluation of slope stability for the raised embankment were performed using the latest version of STABL; a computer code originally developed at Purdue University (Siegal, 1975). The version used in this investigation, STABL4, was adapted to IBM microcomputer software. The STABL4 program solves for factors of safety against slope movement along an assumed surface of sliding. The analysis involves dividing the soil mass inscribed by the assumed failure surface into vertical slices. The driving and resisting force components for each slice is computed, and force and moment equilibrium is evaluated for the entire sliding mass.

The program offers several options for the analytical method used in the analysis; all of which use the theory of limit equilibrium. In this theory, the factor of safety is defined as the ratio of available shear strength to the shear stress that is mobilized along each failure surface under the given loading conditions.

The Bishop Simplified Method (Bishop, 1955) was used for this analysis. Circular failure surfaces were assumed. The following assumptions are required to set up a determinant equilibrium equation (in terms of a factor of safety) using the Bishop Simplified Method:

1. The soil behaves as a Mohr-Coulomb material; that is, the shear strength is linearly dependent on the effective normal stress and is composed of both cohesional and frictional components.

2. The factor of safety for the cohesional and frictional components of strength are equal.

3. The factor of safety is the same for each "slice" in the soil mass inscribed by the assumed failure surface.

4. The resultant interslice forces act horizontally.

5. The normal force at the base of each slice is equal to the summation of forces in the vertical direction.

Based on this set of assumptions, the program iterates using a search routine which can be controlled by the user to evaluate factors of safety for "critical" failure surfaces. "Critical" surfaces as used herein are defined as those failure surfaces which would cause potentially significant distortion of the embankment and loss of freeboard or structural integrity. Lower factors of safety can usually be computed for non-critical surfaces, such as shallow "infinite slope" type failures, particularly when the slope material is noncohesive. The factors of safety computed for infinite slope conditions are significant only in terms of slope maintenance, or "ravelling" problems, and do not reflect the factor of safety against failure of the embankment.

The STABL4 program also solves for earthquake loading effects using a pseudo static method of analysis. In this method, the earthquake force is assumed to act as an additional horizontal force equal to some percentage of gravitational force times the weight of each slice. For example, a specified acceleration of 0.05g means that an additional horizontal force equal to 0.05 times the weight of each slice was considered to act in an unstabling direction in the stability analysis. The horizontal earthquake force is assumed to act at the mid-height of each slice. For these analyses, the pseudo-static earthquake coefficient was increased until the corresponding factor of safety was equal to 1.0. The shear strength values used for the pseudo static stability analysis were the same as those used for the static analysis. No reductions were applied.

2. Slope Stability Analysis Results

In order to evaluate the results of the stability analyses, it is necessary to compare them with some accepted standard set of values used in the design and analysis of dams. One such standard is shown on Table V-7, which presents the proposed minimum factors of safety set forth by the National Research Council (1983). As shown, the allowable minimum factor of safety varies depending on the loading condition.

TABLE V-7

MINIMUM ACCEPTABLE FACTORS OF SAFETY

Loading Condition	Minimum Factor of Safety
Long Term	1.5
Temporary	1.2 - 1.3
Pseudo-Static	1.0 - 1.1

Slope stability analyses were performed for design Alternatives B and C only (Figures V-7 and V-8). Alternative A was not analyzed, but due to the smaller dam size and similar slopes, it can be concluded that the factors of safety for that configuration will be higher than those determined for Alternative B. Several different cases were evaluated to simulate critical upstream and downstream loading conditions, as follows:

Case 1. Upstream Slope Stability

A. Steady State Seepage at Maximum Reservoir Level
B. Steady State Seepage at an Intermediate or Partial Pool Level
C. Rapid Drawdown Conditions
D. Pseudo Static Analysis at Maximum Reservoir Level

Case 2. Downstream Slope Stability

A. Steady State Seepage at Maximum Reservoir Level
B. Pseudo Static Analysis at Maximum Reservoir Level

The shear strengths and unit weight values used in the stability analyses for each embankment zone are shown in Figure V-14. Due to the cohesionless nature of the borrow materials and dam foundation soils, the cohesion intercept was assumed equal to zero for all materials. All analyses were performed using effective stress-based strength parameters.

Since material property values were assumed constant, the major differences between the Alternative B and C designs, in terms of stability, are the upstream slopes (3:1 for Alternative B and 2.5:1 for Alternative C), the downstream section geometries, and the location of the long-term phreatic surface.

The computed factors of safety for the cases listed above are summarized for both design configurations in Table V-8. In these analyses, ranges of factor of safety values are shown for the static stability analyses. Factor of safety ranges are provided rather than a single value, to indicate the variation in factor of safety corresponding to a selected band of surfaces which were considered critical.

<u>Alternative B</u>

The results for the Alternative B static analyses for the various cases evaluated are presented in Figures V-15 through V-18. The shaded regions in those figures show the range of potential critical failure surfaces, which were determined to have the minimum factor of safety range indicated. The failure surface having the minimum value of all the surfaces judged to be "critical" in terms of stability of the structure is also shown on each figure.

The lowest factor of safety for the upstream (Case 1) analyses was for the rapid drawdown loading condition, with a minimum factor of safety equal to 1.44. This value is well above the minimum acceptable factor of safety for temporary conditions shown in Table V-7, even though conservative assumptions were used for the rapid drawdown loading. These assumptions were that the reservoir is drawn down instantaneously from maximum elevation to minimum pool, with the pore pressures in the embankment defined by the maximum steady state phreatic surface. These assumptions, which are referred to as "vertical equipotentials," are very conservative considering the actual rate at which the reservoir will normally be drawn down, and the relatively free draining nature of the granular embankment materials. The long term factors of safety are also well above the acceptable values shown in Table V-8.

The pseudo-static results for Alternative B are presented in Figure V-19, for both upstream and downstream slopes. These results indicate a yield acceleration (i.e. factor of safety equal to 1.0) of 0.25g for the upstream slope and 0.30g for the downstream slope. As indicated in Section III.H, the seismicity of the project site is considered low to moderate in nature. The expected peak bedrock acceleration at the site is 0.09g to 0.13g. The peak bedrock acceleration is not to be confused with the sustained horizontal force used in the pseudo static analysis. Based on the peak bedrock accelerations, a design horizontal acceleration for evaluating the pseudo static stability analyses of 0.05 was assumed. As shown on Table V-8, the factor of safety at 0.05g, for both the upstream and downstream slope, is well above the 1.0-1.1 minimum value.

Alternative C

The static analyses for Alternative C are presented in Figures V-20 through V-22. The critical failure surface range for the upstream partial pool condition is not shown for Alternative C, but the locations for the potential failure surface range and minimum critical surface are identical with those shown in Figures V-20 and V-21 for maximum pool and rapid drawdown conditions. The minimum factor of

TABLE V-8

Summary of Factors of Safety for Slope Stability-Alternatives B and C

CASE	CONDITIONS	FACTORS OF SAFETY				
		INFINITE SLOPE[1]	RANGE FOR CRITICAL[2] FAILURES SURFACES			
			ALTERNATIVE B	(FIG. NO.)	ALTERNATIVE C	(FIG NO)
1	Upstream Slope					
	Alternative B (3:1)	2.3				
	Alternative C (2.5:1)	1.9				
(A)	Steady State Seepage at Maximum Reservoir Level (Elev. 9280 ft.)		2.62-3.14	(V-15)	2.13-2.50	(V-20)
(B)	Steady State Seepage at Partial Pool (Elev. 9260 ft.)		2.62-2.94	(V-16)	2.05-2.38	(None)
(C)	Rapid Drawdown		1.44-1.85	(V-17)	1.16-1.46	(V-21)
(D)	Pseudo Static		[3]		[3]	
	(i) 0.05g horizontal accelaration		2.02		1.76	
	(ii) 0.10g horizontal acceleration		1.63	(V-19)	1.49	(V-23)
	(iii) 0.15g horizontal acceleration		1.36		1.28	
	(iv) 0.20g horizontal acceleration		1.16		1.13	
2	Downstream Slope					
	Upper Earthfill Section (2:1)	1.5				
	Lower Rockfill Section (1.5:1)	1.5				
(A)	Steady State Seepage ot Maximum Reservoir Level (Elev. 9280 ft.)		1.81-2.03	(V-18)	1.54-1.58	(V-22)
(B)	Pseudo Static		[3]		[3]	
	(i) 0.05g horizontal acceleration		1.62		1.36	
	(ii) 0.10g horizontal acceleration		1.45	(V-19)	1.23	(V-23)
	(iii) 0.15g horizontal acceleration		1.32		1.12	
	(iv) 0.20g horizontal acceleration		1.20		1.02	

[1] Factors of safety correspnding to infinite slope conditions are significant only in terms of slope maintenance.

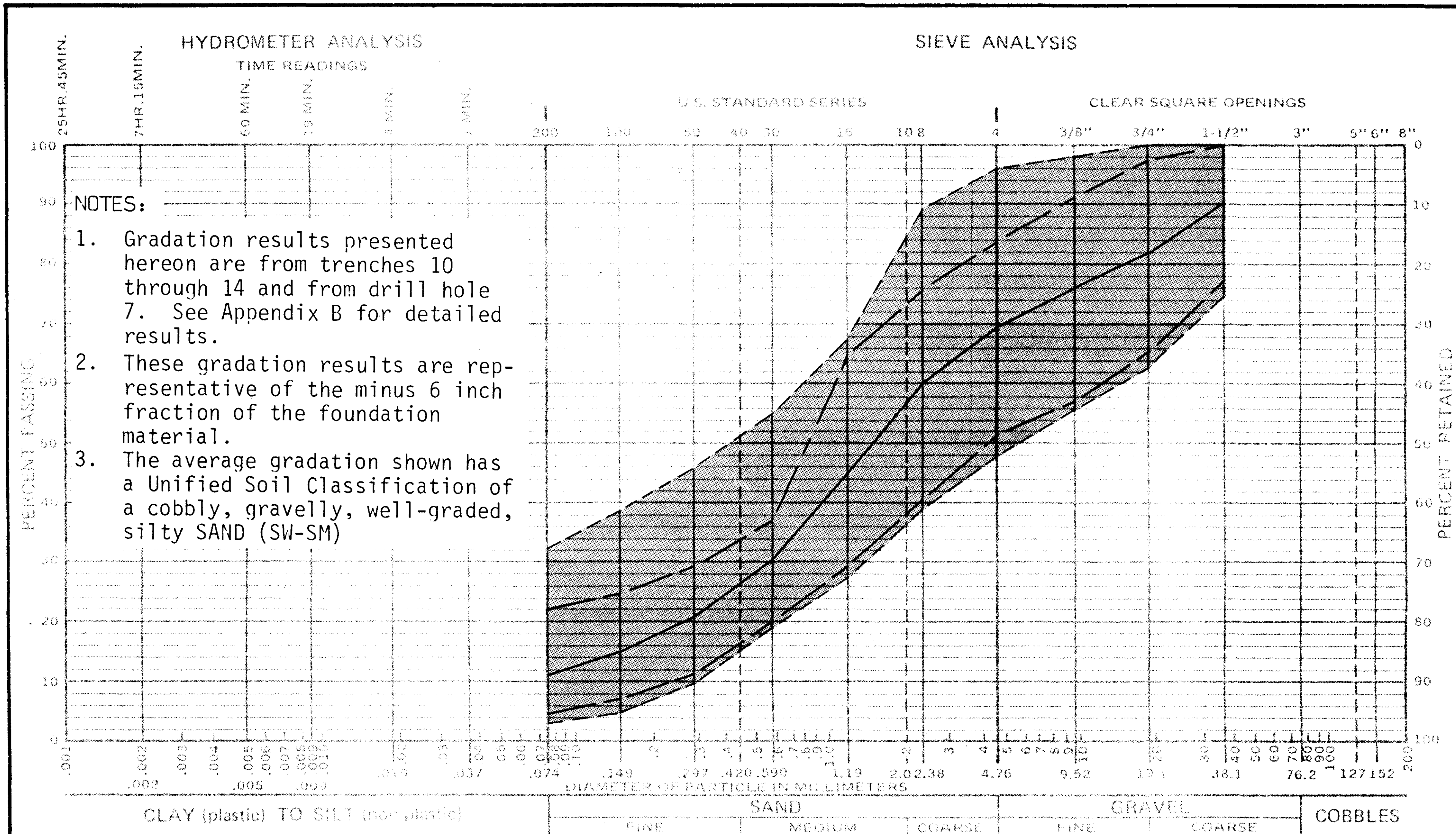
[2] "Critical" failure surfaces are herein defined as potential failure surfaces which would result in significant distortion of the embankment and loss of freeboard or structural integrity.

[3] Minimum factors of safety reported for pseudo static analyses were evaluated using the minimum critical failure surface from the static analyses.

safety for rapid drawdown for Alternative C was 1.16; slightly below the acceptable value shown on Table V-7. However, considering the grossly conservative assumptions for that loading condition, the factor of safety value is acceptable. The minimum long term factor of safety for the downstream slope is also very close to the minimum acceptable value, due to the high phreatic line at which results for this configuration.

The pseudo static results for Alternative C are shown in Figure V-23. For this configuration, the minimum yield acceleration is for the downstream slope, with a value of 0.21g.

A 2.5:1 upstream slope was evaluated for the Alternative C design because the results for the 3:1 slope of the Alternative B design were very conservative. Since the stability results for the 2.5:1 upstream slope inclination for Alternative C are questionable, the final design will consider and evaluate an intermediate slope, say 2.7:1 or 2.8:1, in order to optimize the safety and overall cost of the embankment.



KEY:

- ———— AVERAGE RESULTS FOR 11 GRADATION TESTS
- — — — UPPER AND LOWER BOUND COMPRISING 9 OF 11 TESTS (82 PERCENTILE)
- (shaded) UPPER AND LOWER BOUND FOR ALL TESTS

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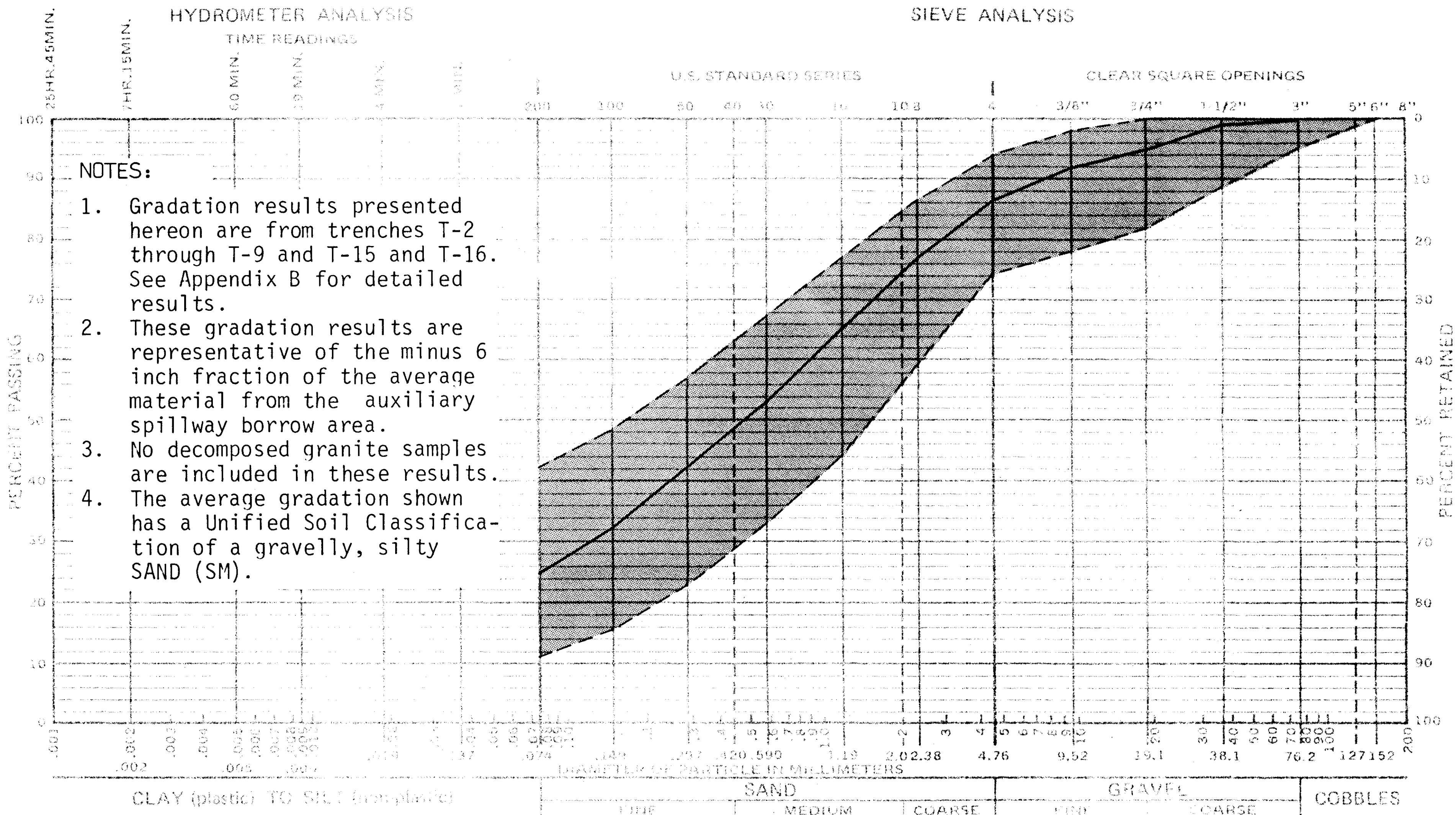
Fort Collins, Colorado

SHELL VALLEY WATERSHED – LEVEL III STUDY

GRADATION TEST RESULTS
DAM FOUNDATION AREA

Checked by D. J. Hamble	Date 10-22-85	Project No.	Figure No.
Approved by R. F. Volpe	Date 10/22/85	3235	V-1



NOTES:

1. Gradation results presented hereon are from trenches T-2 through T-9 and T-15 and T-16. See Appendix B for detailed results.
2. These gradation results are representative of the minus 6 inch fraction of the average material from the auxiliary spillway borrow area.
3. No decomposed granite samples are included in these results.
4. The average gradation shown has a Unified Soil Classification of a gravelly, silty SAND (SM).

KEY:

——— AVERAGE RESULTS FOR 13 GRADATION RESULTS

UPPER AND LOWER BOUND FOR ALL TESTS

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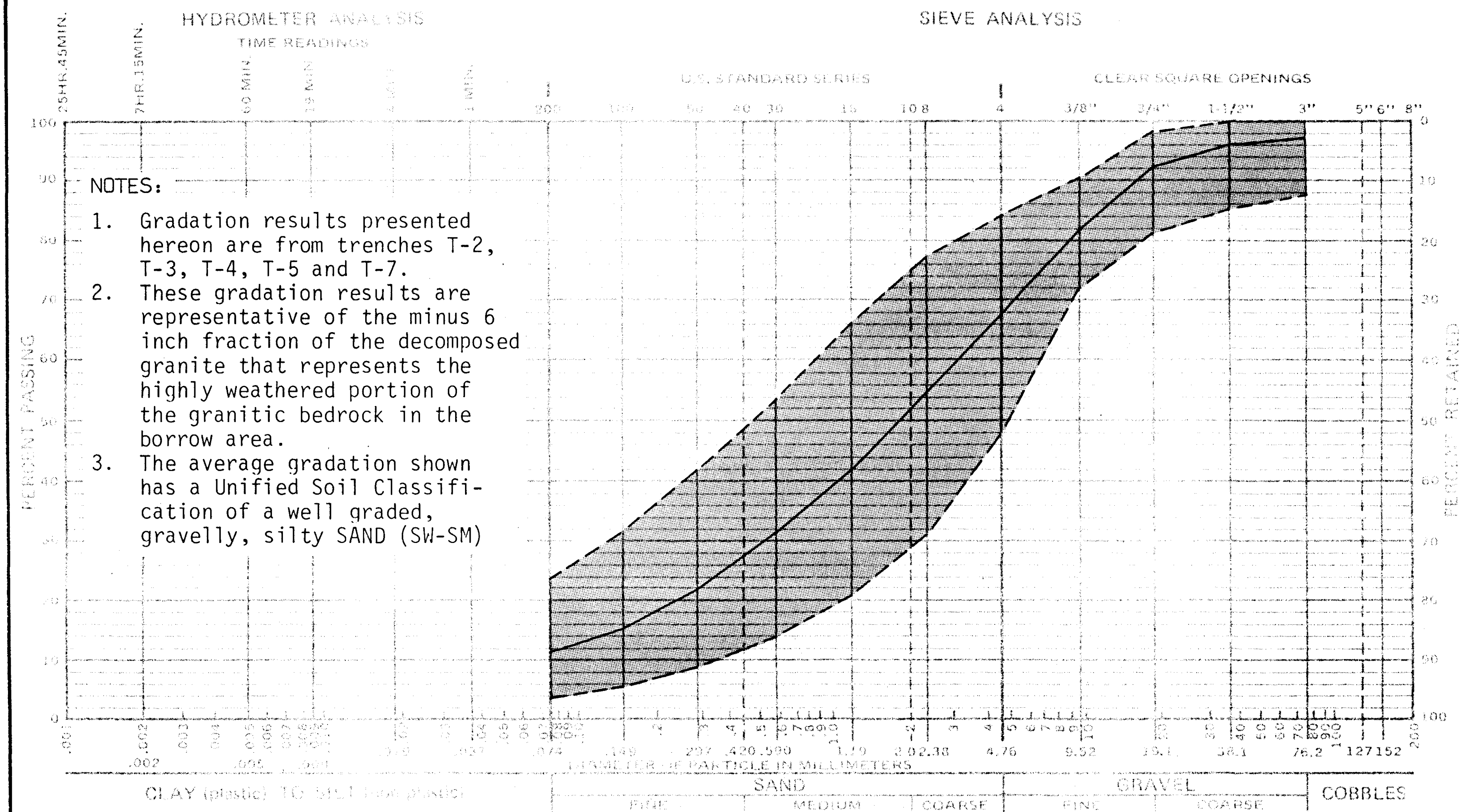
Fort Collins, Colorado

SHELL VALLEY WATERSHED - LEVEL III STUDY
GRADATION TEST RESULTS
BORROW AREA-GLACIAL TILL

Checked by D. J. Humble	Date 10-22-85	Project No. 3235	Figure No. V-2
Approved by R. P. Volpe	Date 10/22/85		



NOTES:

1. Gradation results presented hereon are from trenches T-2, T-3, T-4, T-5 and T-7.
2. These gradation results are representative of the minus 6 inch fraction of the decomposed granite that represents the highly weathered portion of the granitic bedrock in the borrow area.
3. The average gradation shown has a Unified Soil Classification of a well graded, gravelly, silty SAND (SW-SM)

KEY:

———— AVERAGE RESULTS FOR 5 GRADATION RESULTS

– – – – UPPER AND LOWER BOUND FOR ALL TESTS

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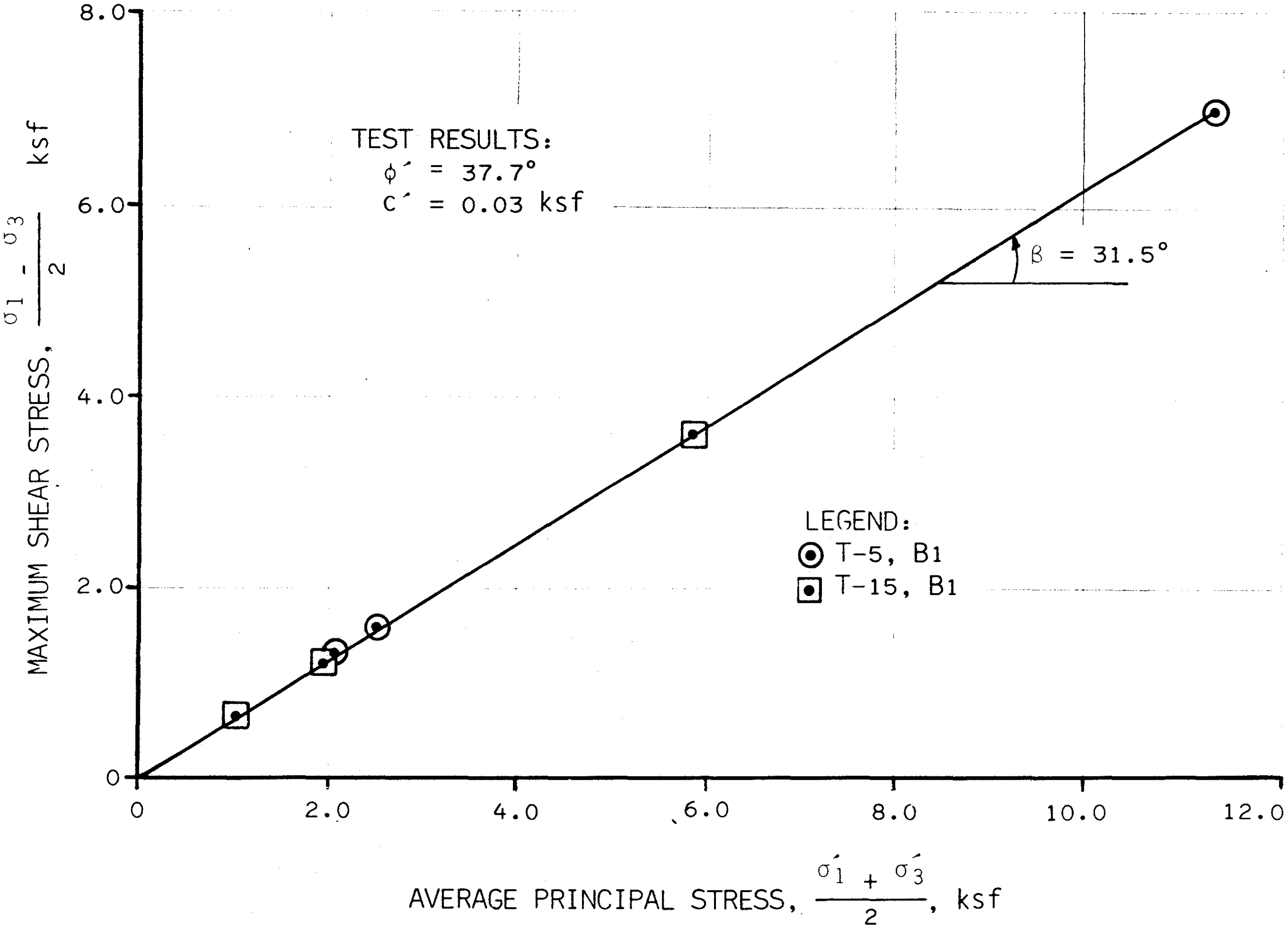
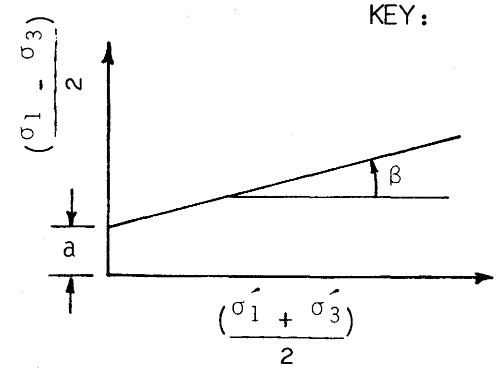
SHELL VALLEY WATERSHED - LEVEL III STUDY GRADATION TEST RESULTS BORROW AREA-DECOMPOSED GRANITE		
Checked by D. J. Hamburg Date 10-22-85 Approved by R. K. Volpe Date 10/22/85	Project No. 3235	Figure No. V-3



KEY:



$\phi' = \sin^{-1} (\tan\beta)$

$c' = \dfrac{a}{\cos\phi'}$

NOTES:

1. The test results presented hereon were determined from consolidated undrained triaxial shear tests.
2. All samples were compacted in the laboratory to 95% of the maximum dry density as defined by ASTM D 698 prior to failure.
3. These test results are considered representative of the borrow area materials.
4. See appendix B for a discussion of test procedures and results.

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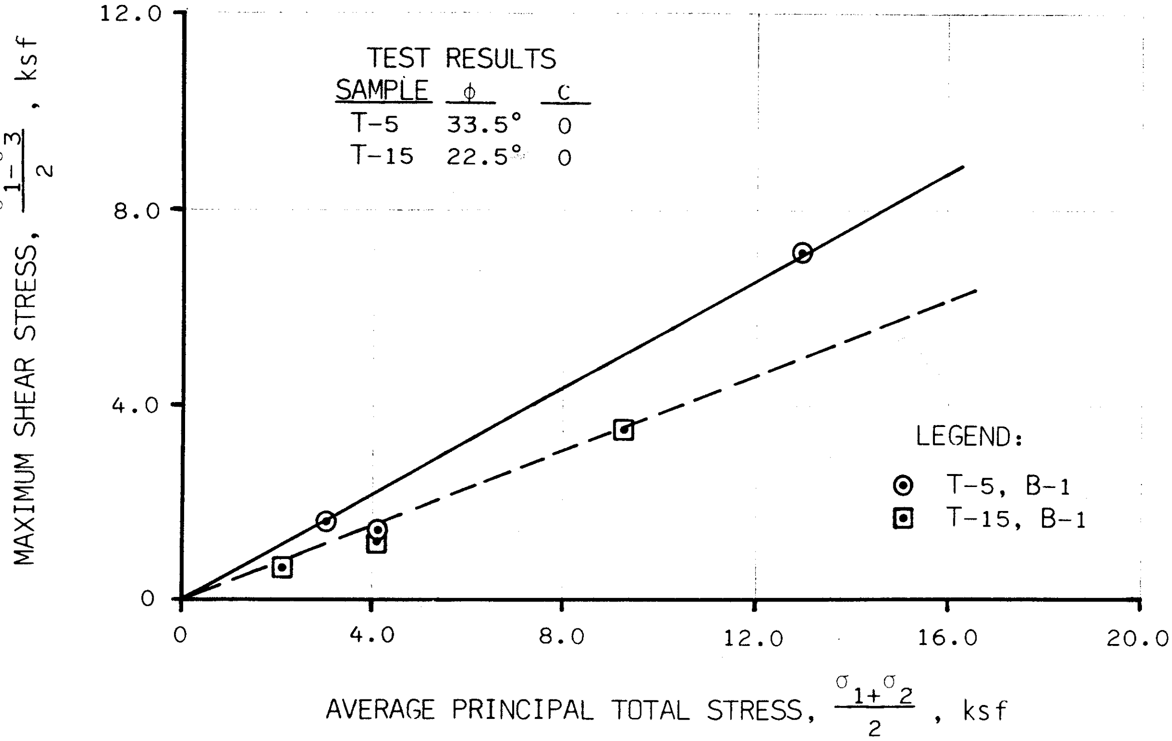
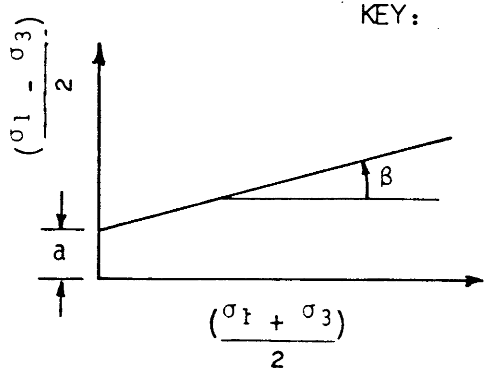
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
EFFECTIVE STRESS STRENGTH RESULTS

Checked by D. J. Hemby Date 10-22-85	Project No.	Figure No.
Approved by R. C. Volpe Date 10/22/85	3235	V-4



KEY:



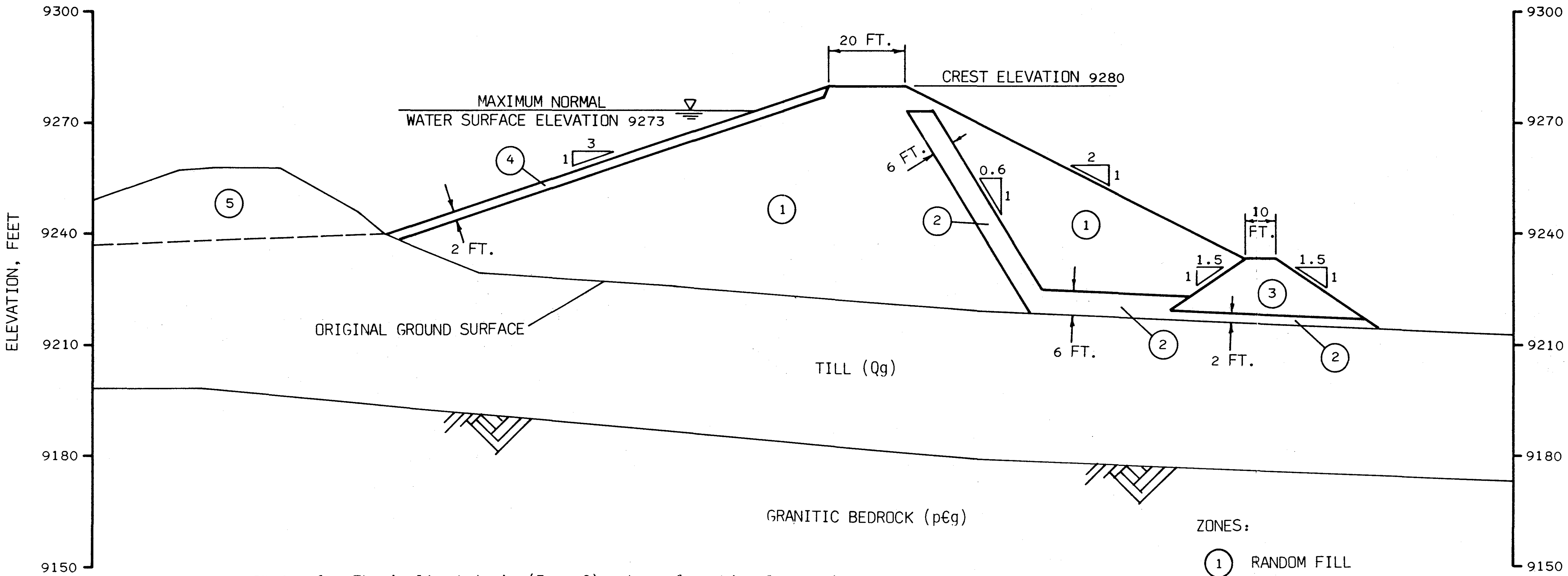
$$\phi = \sin^{-1}(\tan\beta)$$

$$c = \frac{a}{\cos\phi}$$

NOTES:
1. The test results presented hereon were determined from consolidated undrained triaxial shear tests.
2. All samples were compacted in the laboratory to 95% of the maximum dry density as defined by ASTM D 698 prior to failure.
3. These test results are considered representative of the borrow area materials.
4. See appendix B for a discussion of test procedures and results.

ESA Geotechnical Consultants Fort Collins, Colorado			
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY TOTAL STRESS STRENGTH RESULTS			
Checked by D. J. Hamber	Date 10-22-85	Project No.	Figure No.
Approved by R. E. Volpe	Date 10/22/85	3235	V-5

CONCEPTUAL DESIGN
ALTERNATIVE A



ZONES:

(1) RANDOM FILL

(2) FILTER

(3) ROCK FILL

(4) RIP RAP SLOPE PROTECTION

(5) EXISTING EMBANKMENT

NOTES:
1. The inclined drain (Zone 2) shown for this alternative was initially thought to be required. Subsequent seepage analyses indicate that the drain will not be required. Cost analysis for this alternative does not include the inclined drain, but rather the filter and rock toe configuration shown in Alternative C.
2. The rock toe (Zone 3) was sized to receive only the volume of manditory rock excavation from the right abutment area.
3. The existing dam (Zone 5) will be removed for use in construction of the new dam, assuming that the reservoir operation is sufficiently lowered to allow the safe removal of the existing dam down to approximately Elevation 9240. The cut slope will be inclined upstream at a slope of 20:1.

SCALE
0 15 30 FEET

HORIZONTAL = VERTICAL

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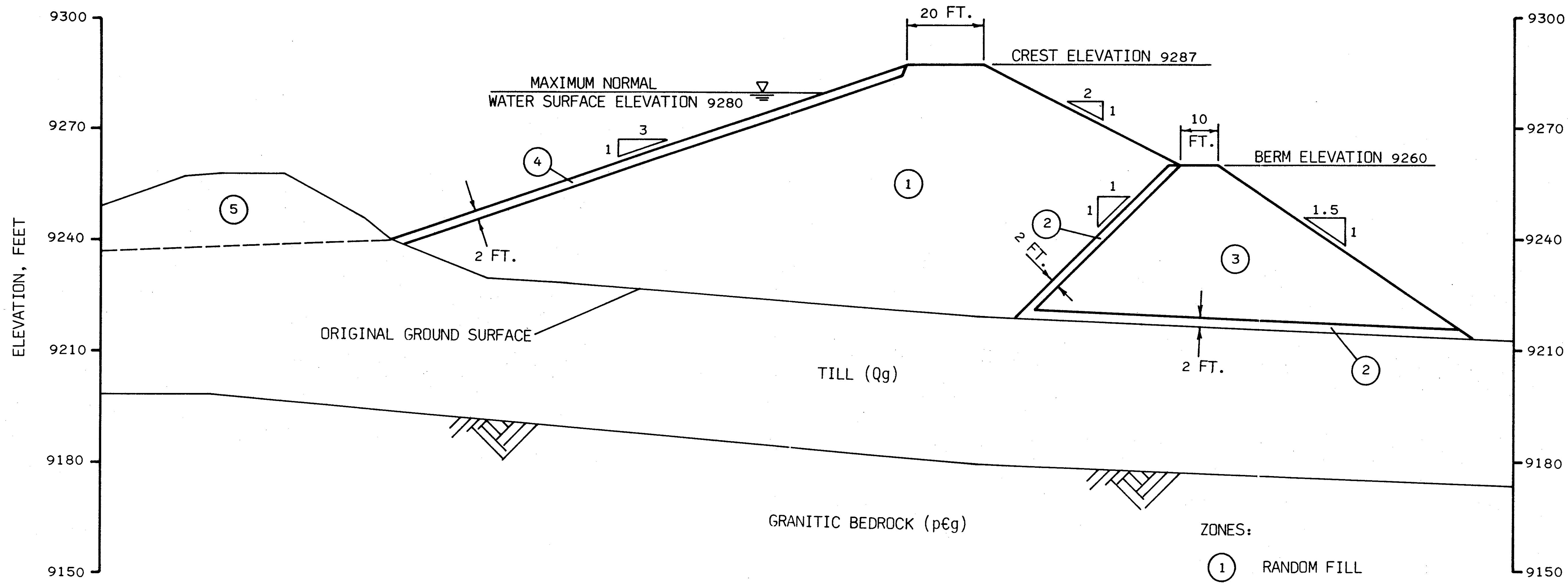
Fort Collins, Colorado

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

Checked by D.J. Hamby Date 10-22-85		Project No. 3235	Figure No. V-6
Approved by R.L. Volpe Date 10/22/85			

CONCEPTUAL DESIGN
ALTERNATIVE B



ZONES:

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

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

SCALE
0 15 30 FEET
HORIZONTAL = VERTICAL

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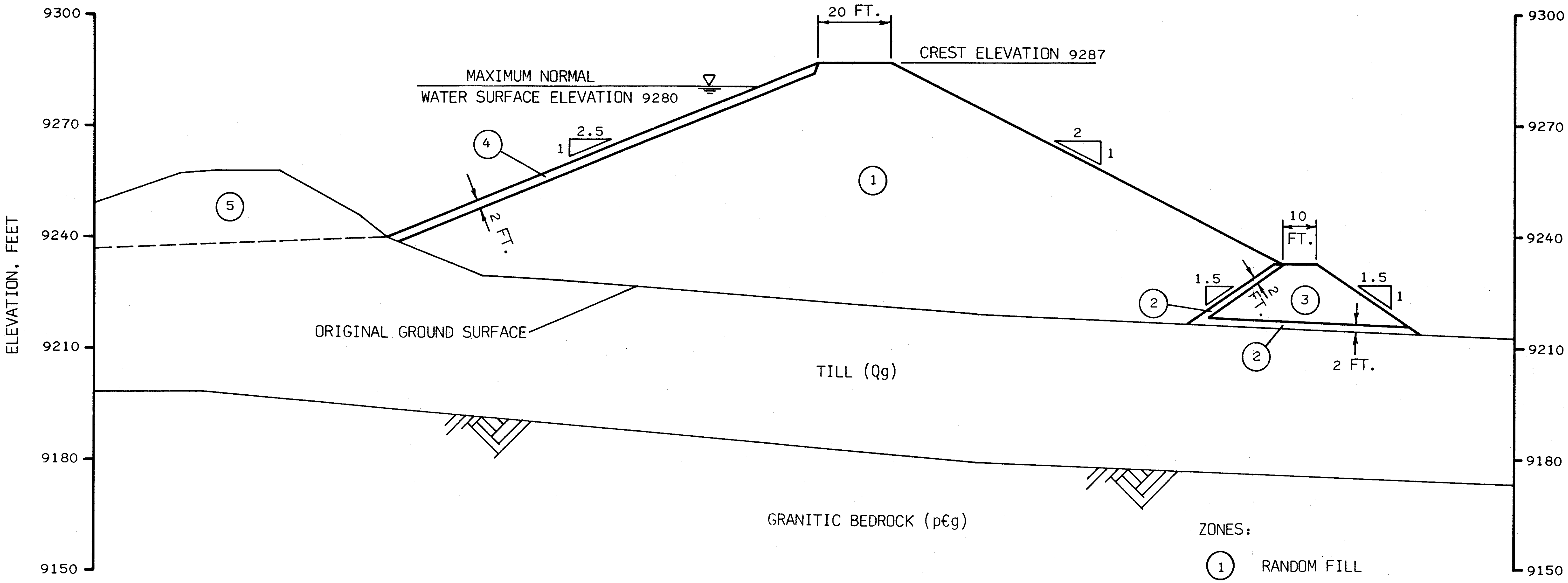
Fort Collins, Colorado

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

Checked by D. J. Hamberg Date 10-22-85	Project No.	Figure No.
Approved by R. L. Volpe Date 10/22/85	3235	V-7

CONCEPTUAL DESIGN
ALTERNATIVE C



ZONES:

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

NOTES:
1. The rock toe (Zone 3) was sized to receive only the volume of manditory rock excavation from the right abutment area.
2. The existing dam (Zone 5) will be removed for use in construction of the new dam, assuming that the reservoir operation is sufficiently lowered to allow the safe removal of the existing dam down to approximately Elevation 9240. The cut slope will be inclined upstream at a slope of 20:1.

SCALE
0 15 30 FEET
HORIZONTAL = VERTICAL

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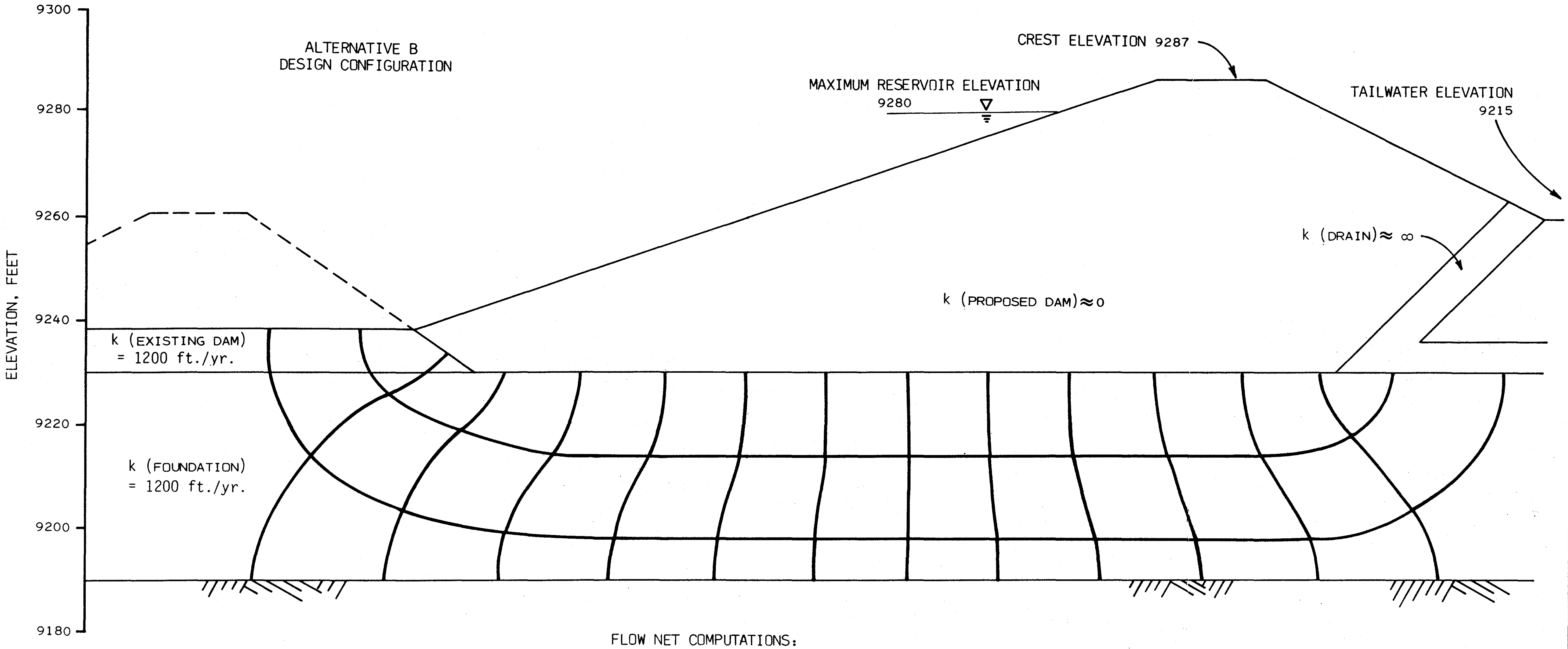
Fort Collins, Colorado

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

Checked by D.J. Hamby	Date 10-22-85	Project No.	Figure No.
Approved by R.L. Volpe	Date 10/22/85	3235	V-8



FLOW NET COMPUTATIONS:

$n_d = 13$

$n_f = 2.4$

$q = k\ h\ \frac{n_f}{n_d}$

$q = (1200\ \text{ft./yr.})\ (65\ \text{ft.})\ (\frac{2.4}{13})$

$q = 14{,}400\ \frac{\text{ft.}^3/\text{yr.}}{\text{ft.}}$

$Q = q\ l^* = (14400\ \frac{\text{ft.}^3/\text{yr.}}{\text{ft.}})\ (515\ \text{ft.})$

$Q = 7{,}416{,}000\ \text{ft.}^3/\text{yr.} = 170\ \text{ac-ft./yr.}$

* l = corrected length of seepage area (see text)

NOTE:
Ground surface shown horizontal for sketch, however total head (h) used in analysis accounts for slope of actual ground surface.

SCALE
0 10 20 FEET

ESA Geotechnical Consultants Fort Collins, Colorado			
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY FLOW NET SKETCH			
Checked by D. J. Hamburg	Date 10-22-85	Project No. 3235	Figure No. V-9
Approved by R. L. Volpe	Date 10/22/85		

THE FOLLOWING NOTES BRIEFLY DESCRIBE THE METHODOLOGY AND ASSUMPTIONS USED FOR THE FINITE ELEMENT METHOD SEEPAGE ANALYSIS RESULTS THAT ARE PRESENTED IN FIGURES V-11, V-12, AND V-13:

1. The seepage analyses were performed using the finite element method groundwater flow model CFLOW3. CFLOW3 was modified at Colorado State University (Edgar, 1979) from previous version computer codes developed by the U.S. Bureau of Mines (1976), which were based on the original model developed at the University of California, Berkeley (Taylor and Brown, 1967). The mathematical model provides solutions for steady state flow conditions and predicts flow velocity, pressure head, potential, and the location of the phreatic surface.

2. A finite element mesh was drawn for each maximum cross section for Alternative A (Figure V-6), Alternative B (Figure V-7), and Alternative C (Figure V-8) design condigurations.

3. Permeability coefficient values were selected based on laboratory test results using fabricated samples from the borrow and dam site areas, and on in-situ pump tests conducted in the dam foundation area.

4. All materials were assumed isotropic with respect to permeability except for the earthfill portion of the proposed embankment. An anisotropy ratio of 5 to 1 for the horizontal (K_V) to vertical (K_H) permeability values was assumed for the silty sand material that will be used to construct the earthfill section of the new embankment

5. The dam foundation material at maximum section consists of approximately 40 ft. of glacial till above granite or monzonite bedrock. The till was assumed isotropic and homogeneous with respect to permeability for one computer run each, for Alternative A, Alternative B, and Alternative C design sections. A second run was performed for the Alternative B and C design sections assuming a two layer foundation till; with the permeability in the lower 20 ft. assumed one order of magnitude lower than the permeability in the upper 20 ft. of till. The two layer foundation model, with a reduced permeability in the lower portion of the till as suggested from the geologic evidence, was investigated to provide a more realistic, if less conservative, estimate of seepage losses through the existing foundation soils.

6. The results from the finite element computer analysis are presented in the form of equipotential diagrams for the maximum reservoir elevation conditions as shown in Figures V-11 through V-13 for Alternatives B and C only.

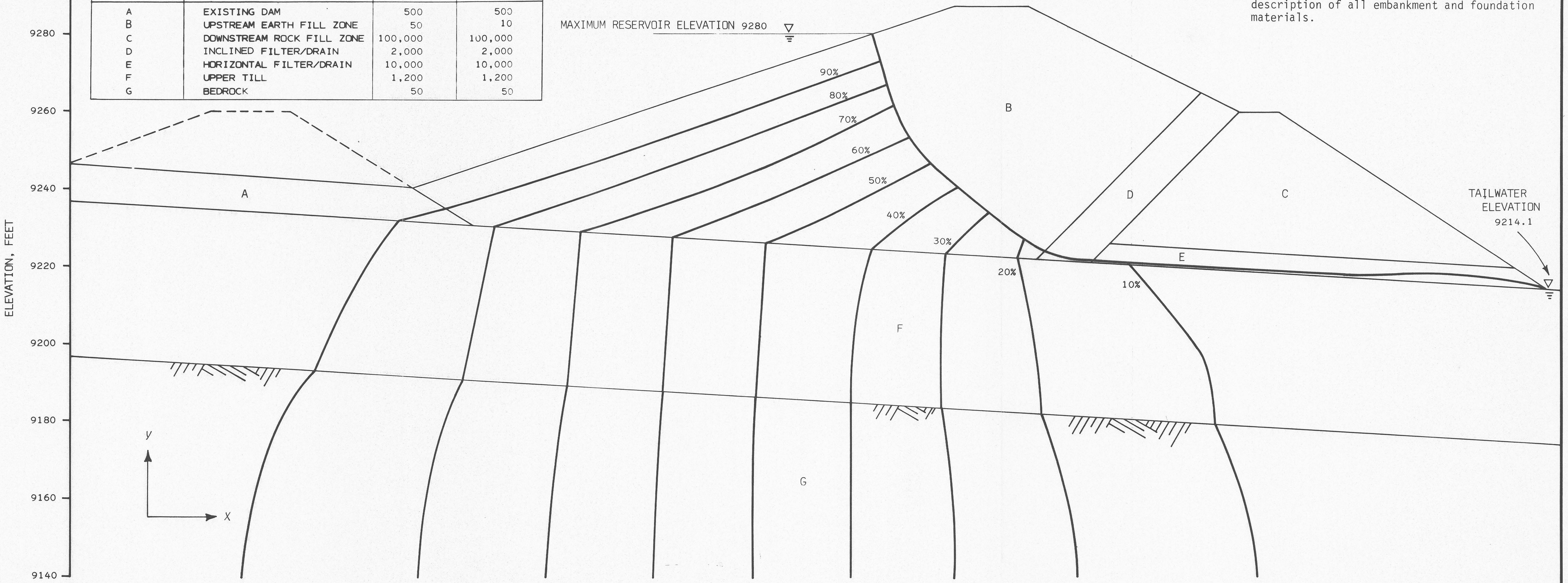
ESA Geotechnical Consultants Fort Collins Colorado		
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY .NOTES FOR EQUIPOTENTIAL DIAGRAM FIGURES		
Checked by D. J. Hamby Date 10-22-85 Approved by R. L. Volpe Date 10/22/85	Project No. 3235	Figure No. V-10

SUMMARY OF PERMEABILITY COEFFICIENTS

MATERIAL DESIGNATION	FEATURE	PRINCIPAL PERMEABILITIES (ft./yr.)	
		Kx	Ky
A	EXISTING DAM	500	500
B	UPSTREAM EARTH FILL ZONE	50	10
C	DOWNSTREAM ROCK FILL ZONE	100,000	100,000
D	INCLINED FILTER/DRAIN	2,000	2,000
E	HORIZONTAL FILTER/DRAIN	10,000	10,000
F	UPPER TILL	1,200	1,200
G	BEDROCK	50	50

NOTES:

1. See Figure V-10 for appropriate notes related to these analyses.
2. See Figure V-7 for zonation and detailed description of all embankment and foundation materials.



SCALE

0 10 20 FEET

HORIZONTAL = VERTICAL

MAXIMUM SECTION
(SINGLE LAYER FOUNDATION TILL MODEL)

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

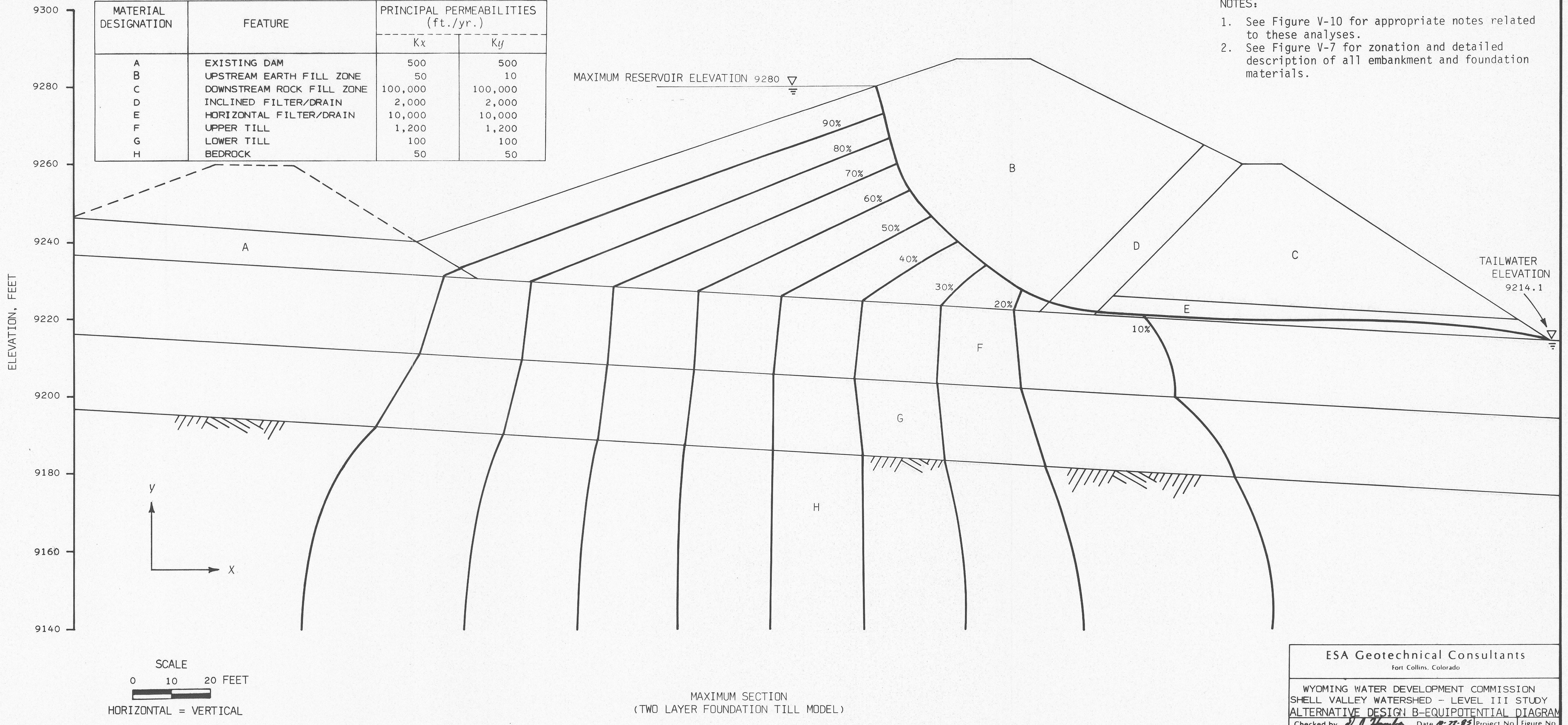
Checked by D. J. Hamburg	Date 10-22-85	Project No. 3235	Figure No. V-11
Approved by R.L. Volpe	Date 10/22/85		

SUMMARY OF PERMEABILITY COEFFICIENTS

MATERIAL DESIGNATION	FEATURE	PRINCIPAL PERMEABILITIES (ft./yr.) Kx	Ky
A	EXISTING DAM	500	500
B	UPSTREAM EARTH FILL ZONE	50	10
C	DOWNSTREAM ROCK FILL ZONE	100,000	100,000
D	INCLINED FILTER/DRAIN	2,000	2,000
E	HORIZONTAL FILTER/DRAIN	10,000	10,000
F	UPPER TILL	1,200	1,200
G	LOWER TILL	100	100
H	BEDROCK	50	50

NOTES:

1. See Figure V-10 for appropriate notes related to these analyses.
2. See Figure V-7 for zonation and detailed description of all embankment and foundation materials.



MAXIMUM SECTION
(TWO LAYER FOUNDATION TILL MODEL)

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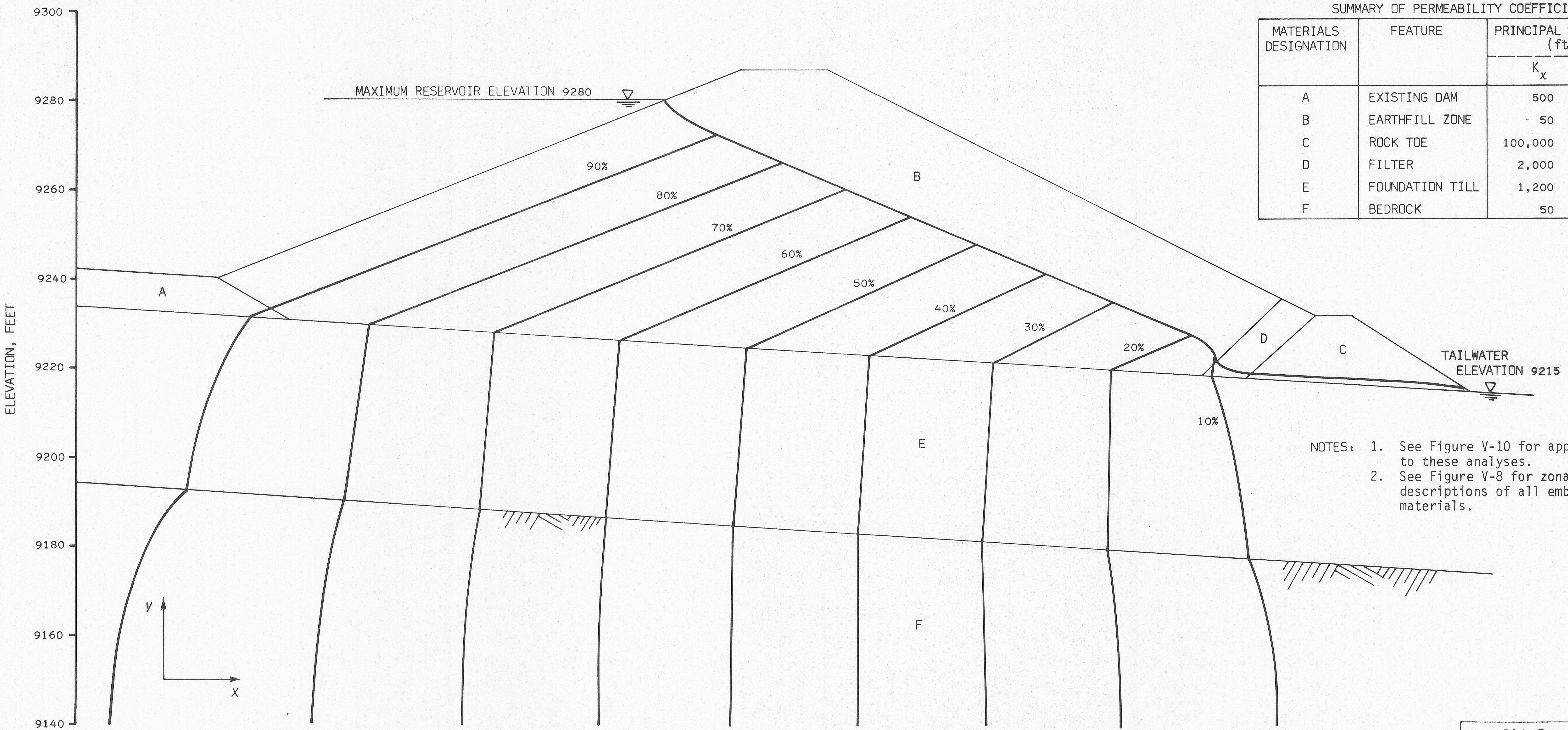
Fort Collins, Colorado

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

Checked by D. L. Hamby Date 10-22-85	Project No. 3235	Figure No. V-12
Approved by R. E. Volpe Date 10/22/85		



SCALE

0 10 20 FEET

HORIZONTAL = VERTICAL

MAXIMUM SECTION

SUMMARY OF PERMEABILITY COEFFICIENTS

MATERIALS DESIGNATION	FEATURE	PRINCIPAL PERMEABILITIES (ft./yr.)	
		K_x	K_y
A	EXISTING DAM	500	500
B	EARTHFILL ZONE	50	10
C	ROCK TOE	100,000	100,000
D	FILTER	2,000	2,000
E	FOUNDATION TILL	1,200	1,200
F	BEDROCK	50	50

NOTES:
1. See Figure V-10 for appropriate notes related to these analyses.
2. See Figure V-8 for zonation and detailed descriptions of all embankment and foundation materials.

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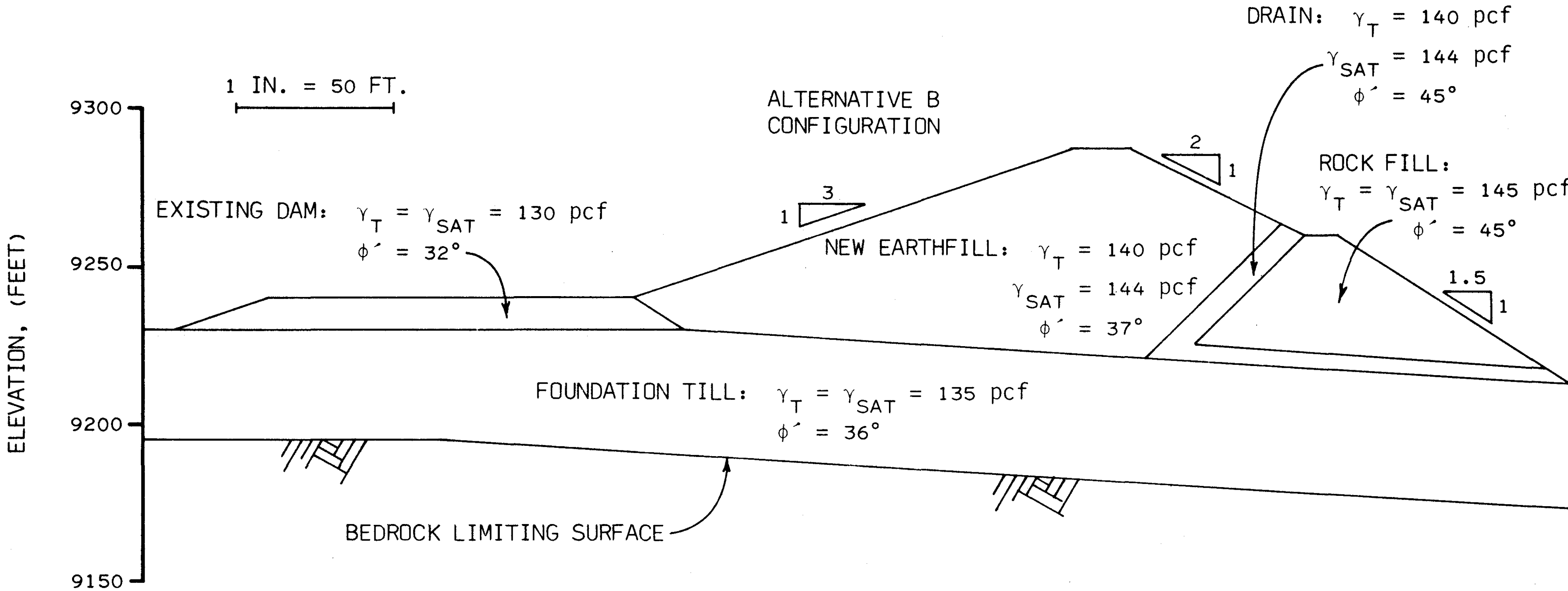
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
ALTERNATIVE DESIGN C-EQUIPOTENTIAL DIAGRAM

Checked by D. J. Hamberg	Date 10-22-85	Project No. 3235	Figure No. V-13
Approved by R. C. Volpe	Date 10/22/85		



NOTE: Material properties for the Alternative C Configuration are the same as those shown here for Alternative B.

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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
MATERIAL PROPERTIES FOR STABILITY ANALYSES

Checked by D. J. Hamby	Date 10-22-85	Project No.	Figure No.
Approved by R. L. Volpe	Date 10/22/85	3235	V-14

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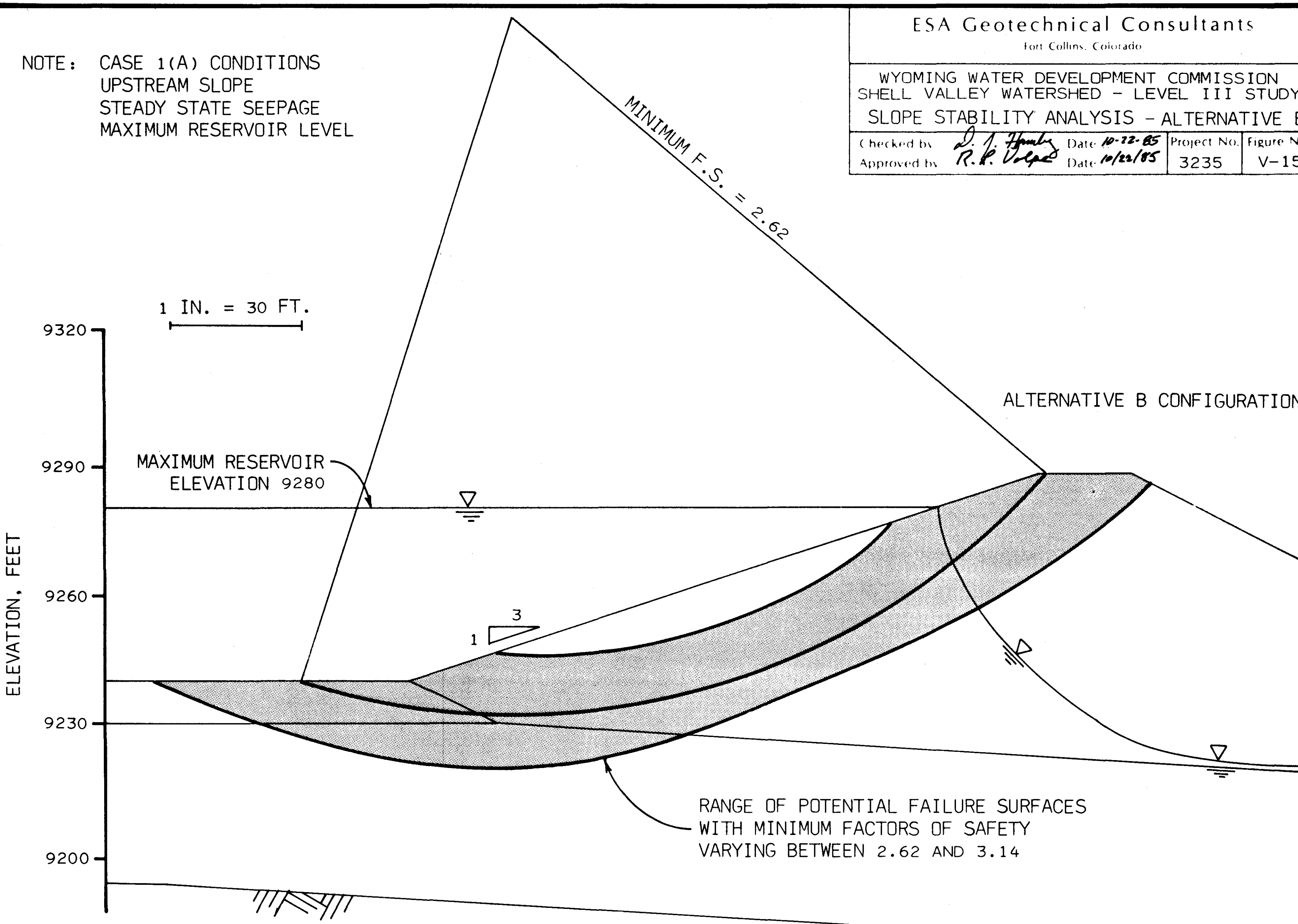
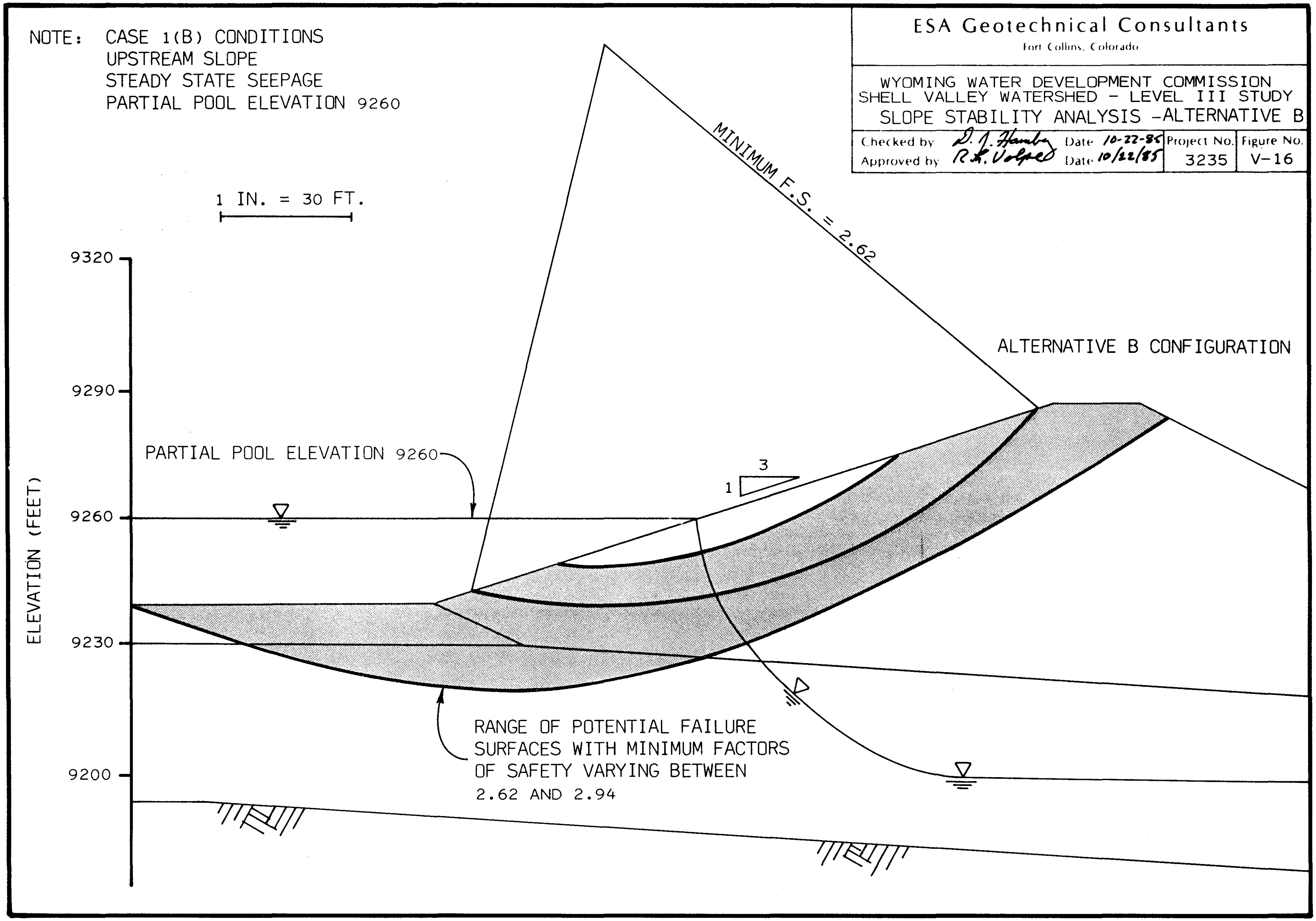
Fort Collins, Colorado

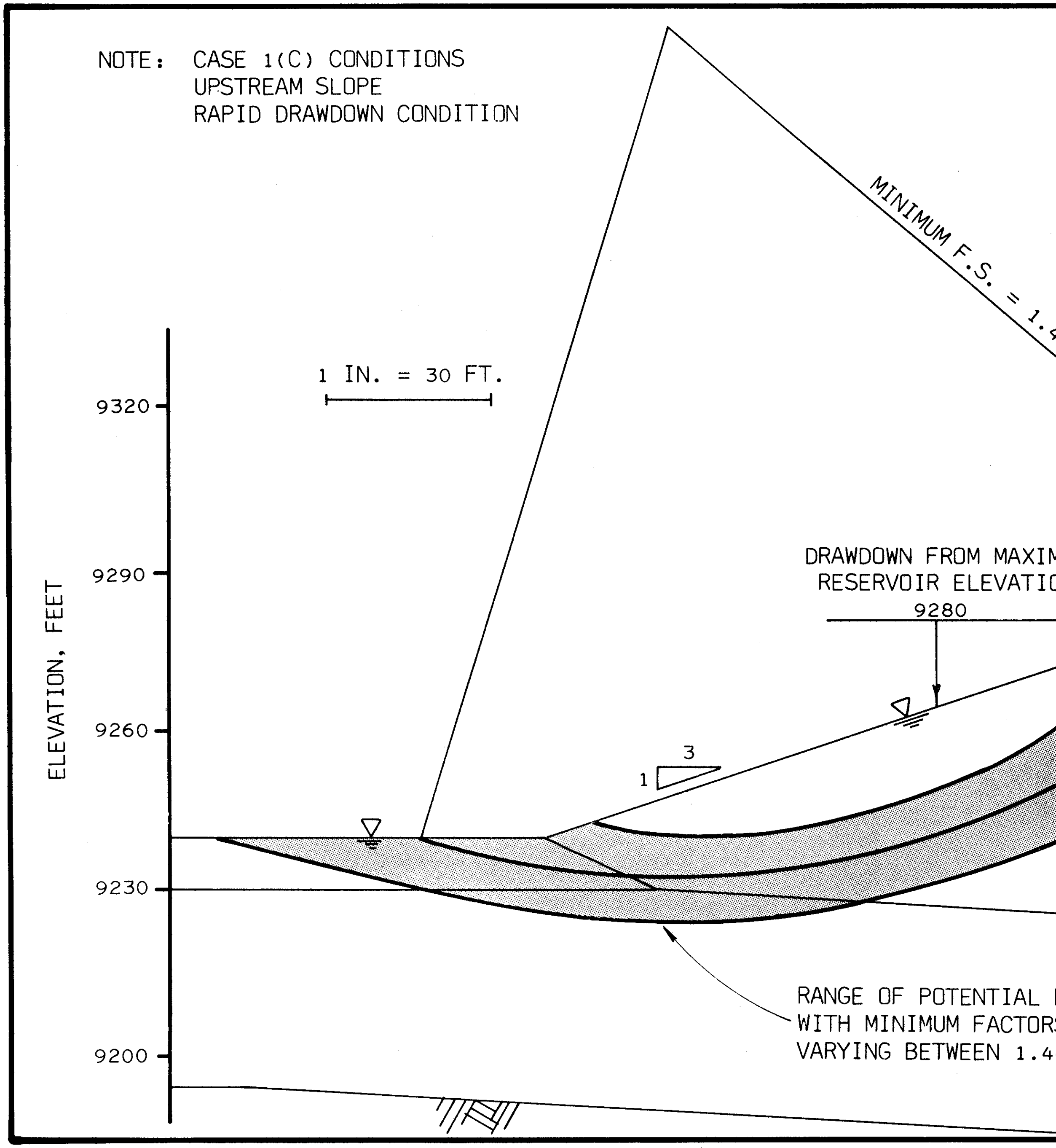
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY

SLOPE STABILITY ANALYSIS - ALTERNATIVE B

Checked by	Date 10-22-85	Project No.	Figure No.
Approved by	Date 10/22/85	3235	V-15

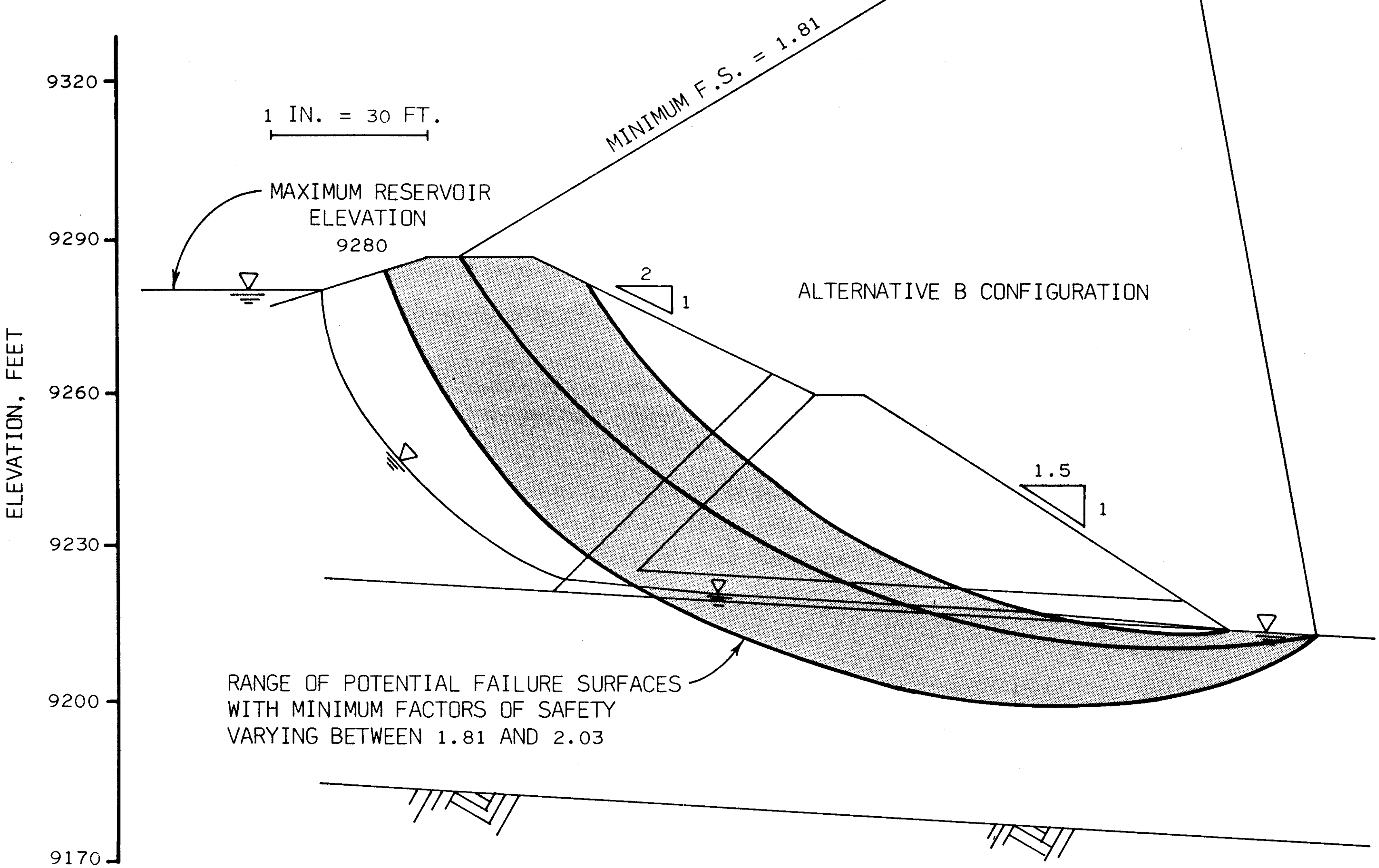



NOTE: CASE 1(B) CONDITIONS
UPSTREAM SLOPE
STEADY STATE SEEPAGE
PARTIAL POOL ELEVATION 9260
ESA Geotechnical Consultants
Fort Collins, Colorado
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
SLOPE STABILITY ANALYSIS -ALTERNATIVE B
Checked by D. J. Hamby Date 10-22-85
Approved by R. K. Volpe Date 10/22/85
Project No. 3235
Figure No. V-16
1 IN. = 30 FT.
MINIMUM F.S. = 2.62
ALTERNATIVE B CONFIGURATION
9320
9290
9260
9230
9200
ELEVATION (FEET)
PARTIAL POOL ELEVATION 9260
3
1
RANGE OF POTENTIAL FAILURE
SURFACES WITH MINIMUM FACTORS
OF SAFETY VARYING BETWEEN
2.62 AND 2.94

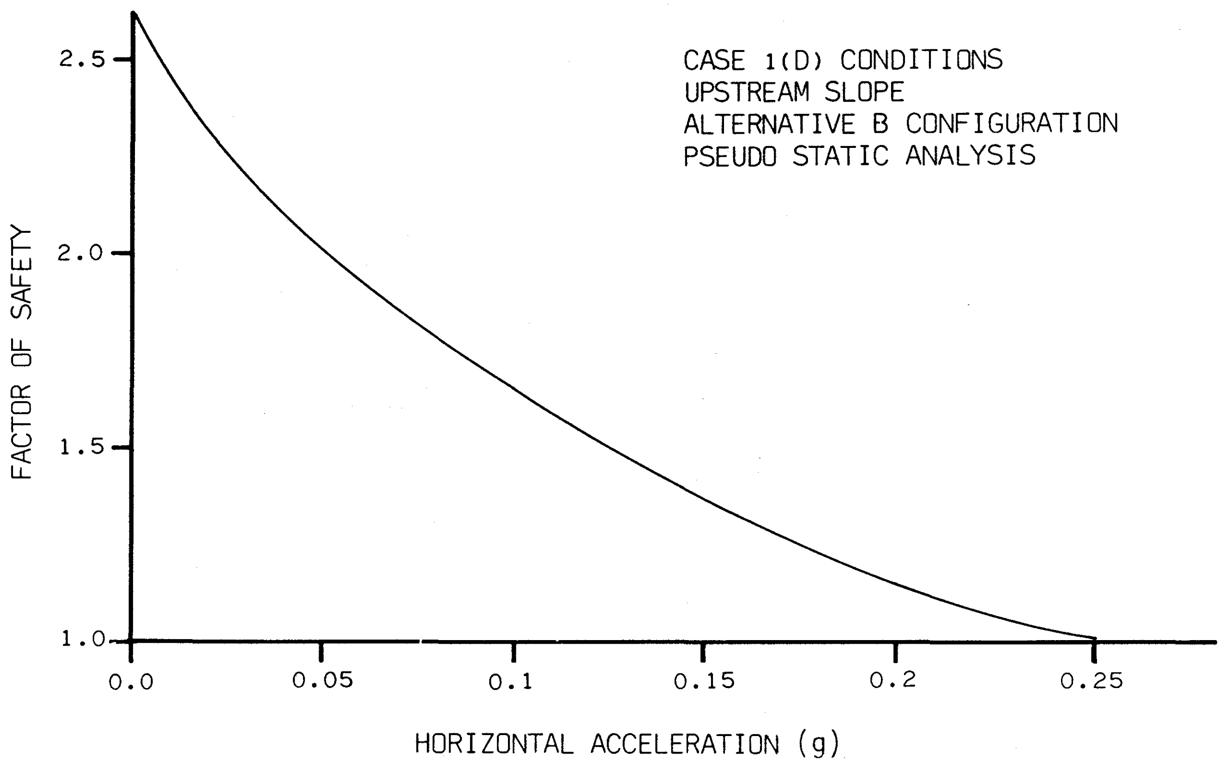


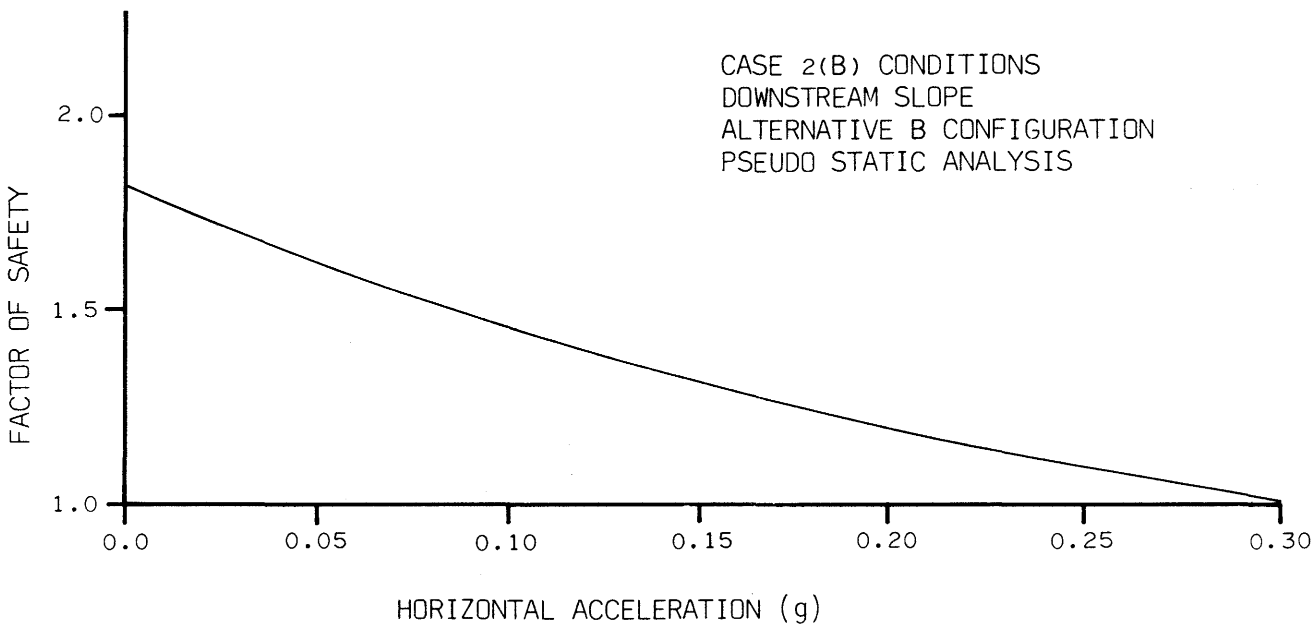
ESA Geotechnical Consultants Fort Collins, Colorado		
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY SLOPE STABILITY ANALYSIS - ALTERNATIVE B		
Checked by D. J. Humbe Date 10-22-85 Approved by R. R. Volpe Date 10/22/85	Project No. 3235	Figure No. V-17

NOTE: CASE 2(A) CONDITIONS
DOWNSTREAM SLOPE
STEADY STATE SEEPAGE AT
FULL RESERVOIR



ESA Geotechnical Consultants Fort Collins, Colorado			
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY SLOPE STABILITY ANALYSIS - ALTERNATIVE B			
Checked by D.J. Humbe Date 10-22-85	Project No.	Figure No.
Approved by R.K. Volpe Date 10/22/85	3235	V-18





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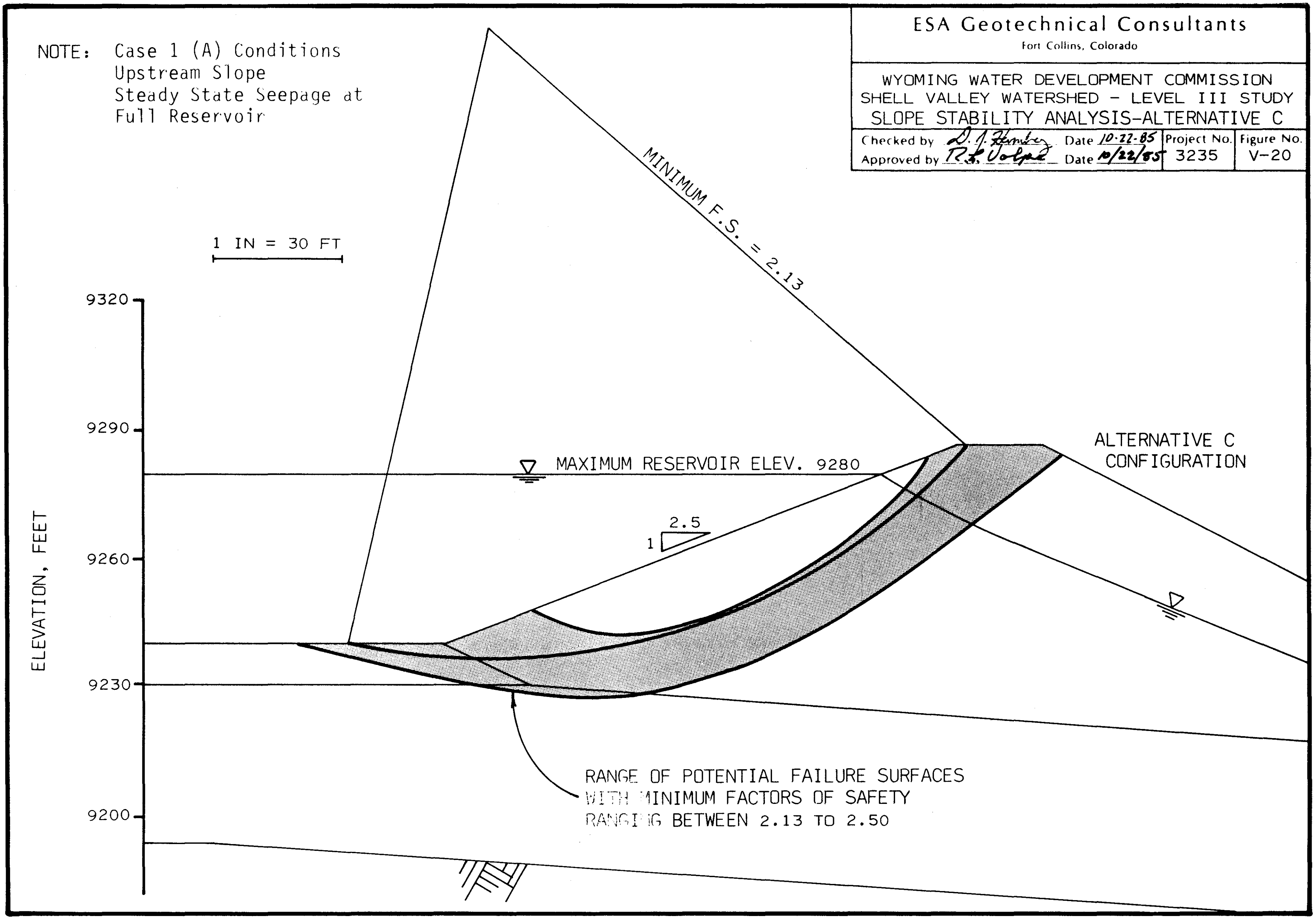
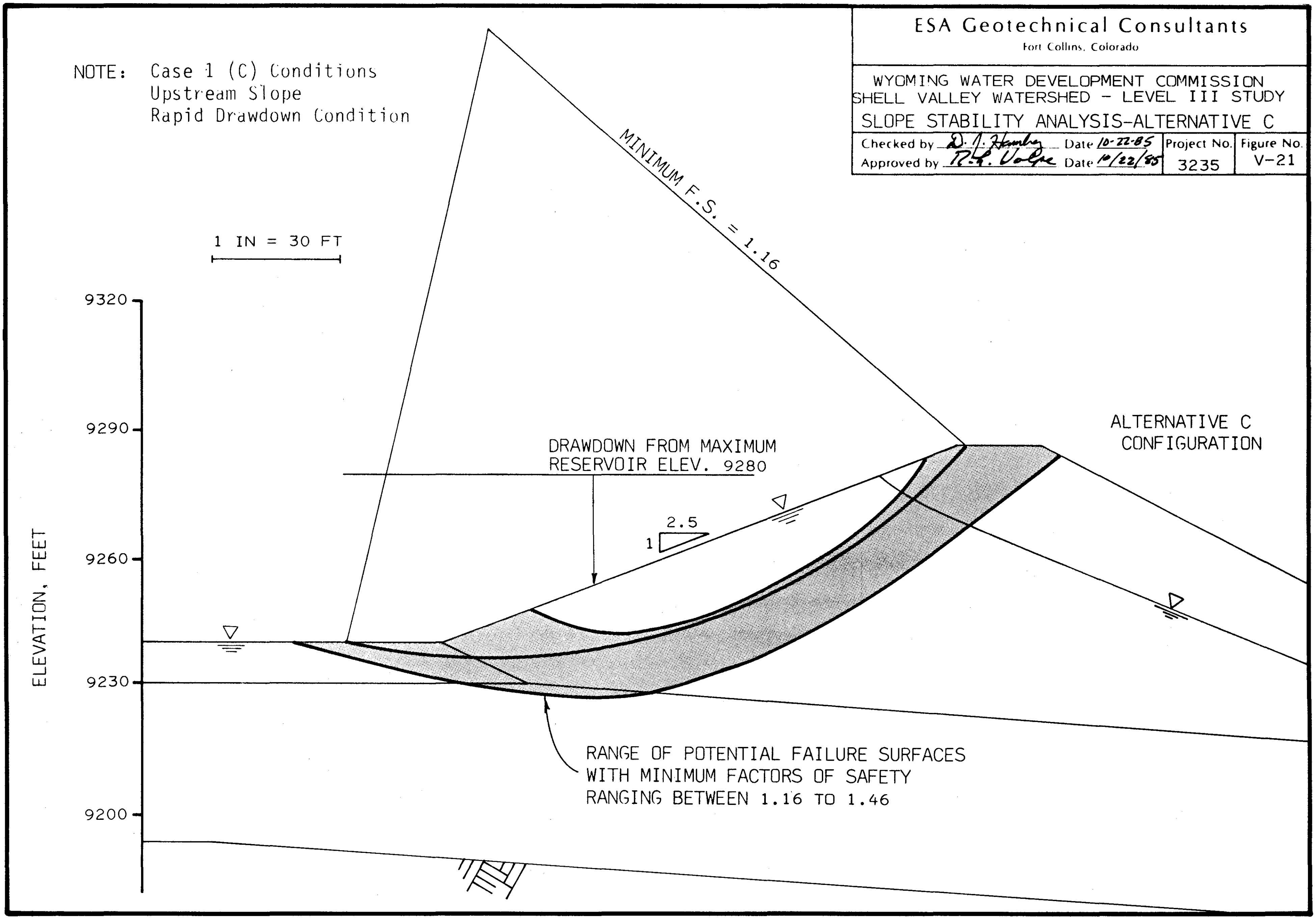
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
PSEUDO STATIC STABILITY - ALTERNATIVE B

Checked by D. J. Hamby	Date 10-22-85	Project No. 3235	Figure No. V-19
Approved by R. L. Volpe	Date 10/22/85		


ESA Geotechnical Consultants
Fort Collins, Colorado
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
SLOPE STABILITY ANALYSIS-ALTERNATIVE C
Checked by D. J. Hemby Date 10-22-85
Approved by R. L. Volpe Date 10/22/85
Project No. 3235
Figure No. V-20
NOTE: Case 1 (A) Conditions
Upstream Slope
Steady State Seepage at
Full Reservoir
1 IN = 30 FT
MINIMUM F.S. = 2.13
MAXIMUM RESERVOIR ELEV. 9280
ALTERNATIVE C
CONFIGURATION
2.5
1
ELEVATION, FEET
9320
9290
9260
9230
9200
RANGE OF POTENTIAL FAILURE SURFACES
WITH MINIMUM FACTORS OF SAFETY
RANGING BETWEEN 2.13 TO 2.50


ESA Geotechnical Consultants
Fort Collins, Colorado
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
SLOPE STABILITY ANALYSIS-ALTERNATIVE C
Checked by
Date 10-22-85
Approved by
Date 10/22/85
Project No. 3235
Figure No. V-21
NOTE: Case 1 (C) Conditions
Upstream Slope
Rapid Drawdown Condition
1 IN = 30 FT
MINIMUM F.S. = 1.16
9320
9290
9260
9230
9200
ELEVATION, FEET
DRAWDOWN FROM MAXIMUM
RESERVOIR ELEV. 9280
2.5
1
ALTERNATIVE C
CONFIGURATION
RANGE OF POTENTIAL FAILURE SURFACES
WITH MINIMUM FACTORS OF SAFETY
RANGING BETWEEN 1.16 TO 1.46

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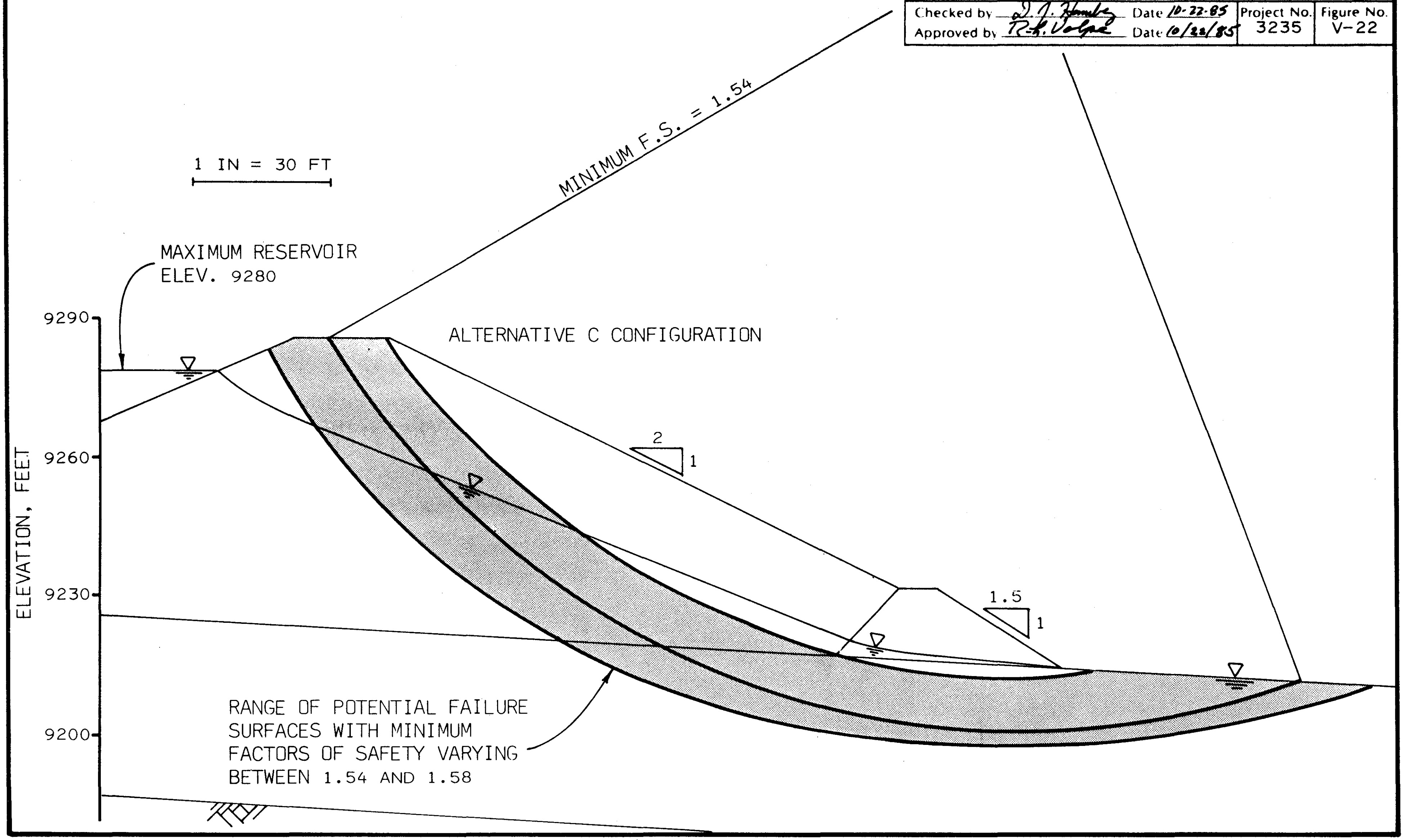
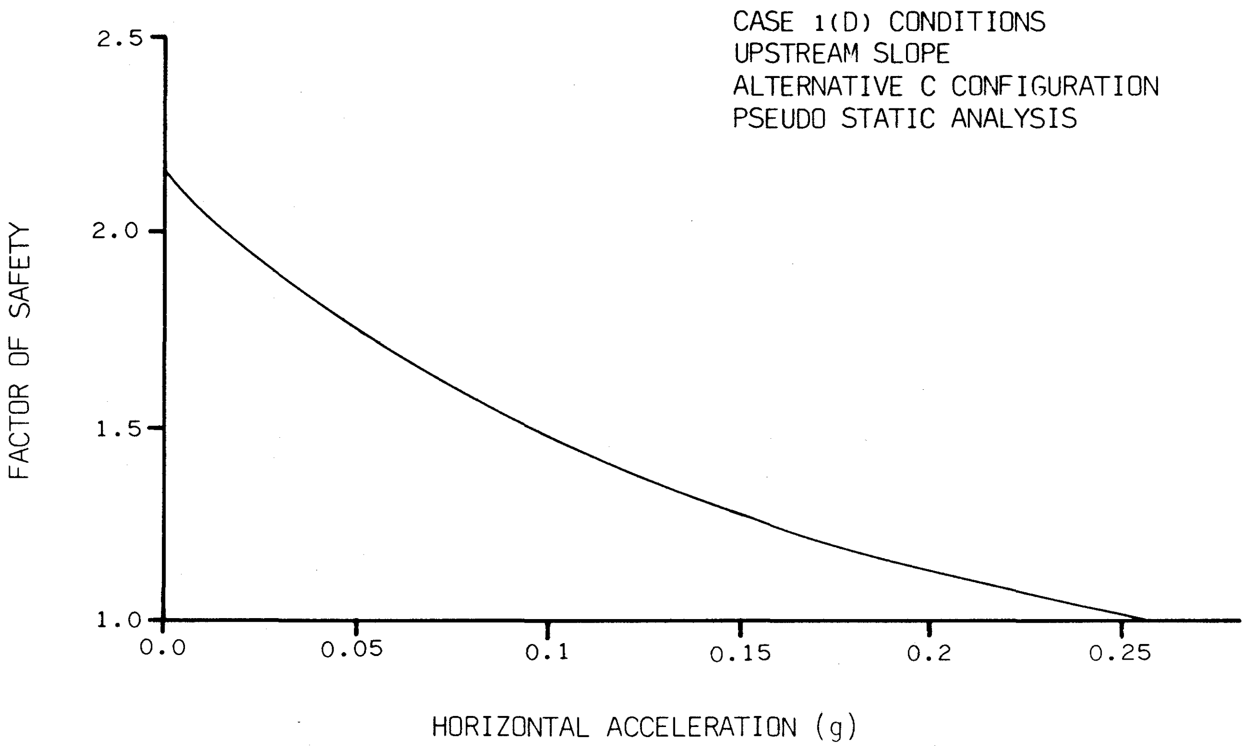
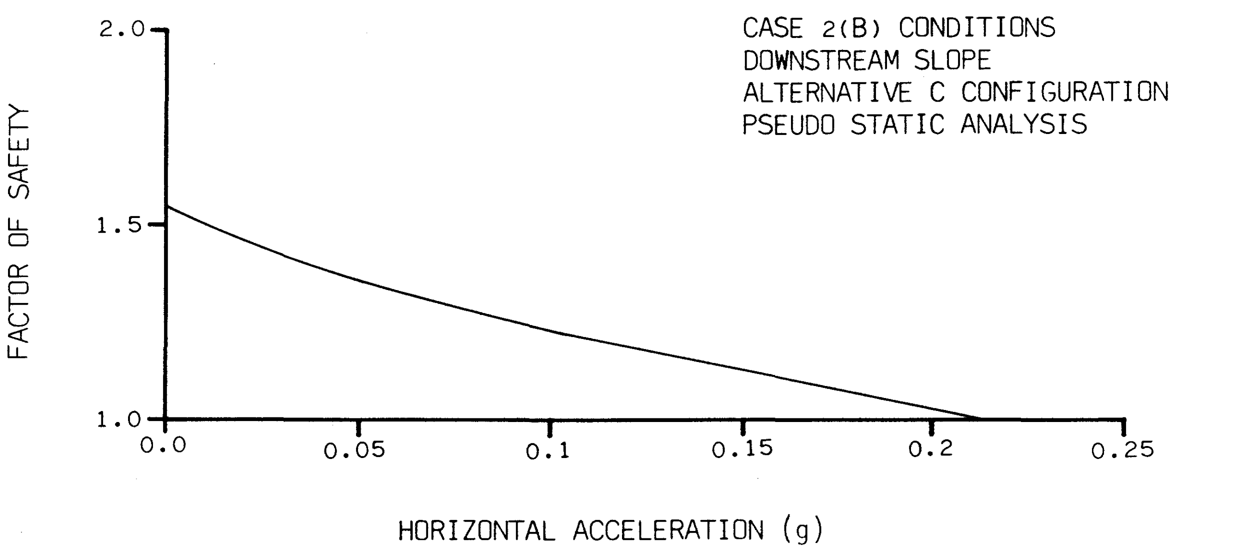
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
SLOPE STABILITY ANALYSIS-ALTERNATIVE C

Checked by D. J. Hamby Date 10-22-85	Project No.	Figure No.
Approved by R. K. Volpe Date 10/23/85	3235	V-22

NOTE: Case 2(A) Downstream Slope
Steady State Seepage
Maximum Reservoir Elevation 9280







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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
PSEUDO STATIC STABILITY-ALTERNATIVE C

Checked by D. J. Hamilton	Date 10-22-85	Project No.	Figure No.
Approved by R. E. Volpe	Date 10/22/85	3235	V-23

VI. HYDRAULIC STRUCTURES AND ACCESS ROAD

A. SPILLWAY ALTERNATIVES

The Wyoming Water Development Commission has instructed the ESA team to design a spillway configuration for Lake Adelaide which will handle a Probable Maximum Flood (PMF) event. Two different flood routing schemes have been developed to accomplish this task, with various spillway combinations associated with each scheme. Due to the need to route the full PMF event, the higher embankment design alternatives (Alternative B or C, with crest elevation 9287 feet) are more economical than Alternative A (crest elevation 9280 feet) in terms of spillway construction costs. This is because significantly more bedrock excavation would be required for the Alternative A design configuration, independent of which flood routing scheme is selected. Therefore, the spillway concepts described below were designed for use with the Alternative B embankment configuration, since the B option is recommended over Alternative C for other reasons.

1. Side Channel Spillway and Emergency Spillway

The recommended flood routing scheme utilizes a side channel service spillway, located on the left abutment, which will be capable of handling about one half of the PMF event. The side channel spillway will operate in tandem with an emergency spillway capable of passing the other one half of PMF flows. The emergency spillway will be located in the vicinity of the borrow area, on the south rim of the reservoir.

The side channel has been sized to pass 2800 cfs, which is approximately one-half of the PMF outflow. The spillway would be constructed in granitic rock on the left abutment, with a spillway crest elevation of 9280, or the maximum normal pool elevation for the Alternative B design. The right abutment is unsuitable as a spillway location due to geotechnical problems. Water would initially spill over a 50 foot long, concrete capped weir section into the side channel. The weir section would be aligned at an angle of nearly 90 degrees to the axis of the dam, paralleling existing topographic contours on the left abutment. The side channel itself is designed to maintain low velocity, subcritical flows in its upper reaches and to accelerate the flow to supercritical velocities as the water moves further downstream away from the dam. In final design, it will be necessary to further explore bedrock elevations along the side channel and discharge chute so that the exact alignment can be defined and the extend of any concrete lining can be estimated.

The spillway length (L) was determined according to the formula Q (flow in cfs) = $CLH^{3/2}$. A conservative C value of 3.1 was selected. The channel along the spillway is designed for spatially varied flow and is 15 ft. wide at the base with 1/2:1 side slopes. Downstream, the channel narrows to 12 ft. at the bottom with a 1/2:1 slope in rock and a 1:1 slope in till. The channel narrows to a 10 ft. width with 1:1 slopes where the flow is directed perpendicular to the slope and is then channeled into a plunge pool at the valley floor. A Manning "n" value of 0.035 was used in channel flow calculations. A stage discharge curve for the side channel spillway is shown in Figure VI-1. A plan and cross section of the side channel spillway are shown in Figure VI-4.

Use of this spillway for the Alternative A design will result in either increased till excavation to reach bedrock or increased concrete lining of the upper discharge chute.

It should be noted that the side channel configuration requires considerable rock excavation for construction of the access road onto the dam. In addition, a bridge over the side channel may be required. The bridge would be constructed using twin tees approximately 37 feet in length. Current plans are to eliminate the road and bridge over the spillway.

In the preliminary design effort, an uncontrolled ogee service spillway was considered, as an alternative to the side channel. This spillway concept was eliminated from further recommendation by the design team because of non-competitive cost considerations. The ogee crest would have been constructed within the left abutment ridge area at elevation 9280, and aligned parallel to the dam axis. The ogee spillway concept proved unsuitable because of the resulting undesirable amounts of rock excavation and/or required concrete lining. However, this section was used in the hydrology flood routing analysis presented in Section IV.E.

The side channel and ogee spillway concepts were designed to operate in tandem with a 36-inch outlet pipe extended through the embankment, and capable of carrying 120 cfs at low reservoir levels. The outlet facilities are discussed in more detail in Section B of this chapter.

The emergency spillway will be a relatively shallow, broad weir, located in the vicinity of the borrow area. The emergency spillway will divert extremely rare flood waters from Lake Adelaide over the ridge separating the Lake Adelaide and Shell Reservoir drainages. Flood waters will be channeled down into Shell Creek below Shell Reservoir. The emergency structure will require minimal shaping and excavation in the borrow area to form the approach apron and weir of the spillway. A detailed analysis and description of this facility will be presented in the final design. However, preliminary analyses indicate that this spillway will have a crest approximately 2.5 feet below the crest of the dam and will start spilling when flood outflows reach about 1500 cfs in the side channel.

2. Drop Inlet Spillway and Auxiliary Spillway

A flood routing scheme which was first proposed in the Level II study, would utilize a drop inlet, or morning glory spillway in combination with an auxiliary spillway. The drop inlet spillway, which would tie into the outlet works pipe extending through the dam, can be practically designed to handle only about a 100-year flood event on its own. Therefore, the drop inlet was designed to operate in tandem with an auxiliary spillway, which would accommodate inflows greater than the 100-year flood, up to the PMF event. The auxiliary structure would be constructed on the south rim of the reservoir in the vicinity of the borrow area. The auxiliary spillway crest would be set two feet higher than the drop inlet spillway crest, which is designed at the maximum minimal pool elevation of 9280 feet. This arrangement takes advantage of reservoir storage to dampen the effects of normal flooding.

The auxiliary spillway will be designed as a broad-crested weir that converges into a discharge channel that will be 30 feet wide with 1:1 side slopes. The approach apron and convergence transition section will be lined with rock riprap to minimize maintenance requirements at low flow usages. Although the facility is not being armored to prevent erosion damage at full design capacity, dam integrity will not be affected. The riprapped portion of the facility is designed using a Manning's "n" of 0.035 while the unlined chute is sized with a "n" value of 0.025. A "C" factor of 3.087 for a broad crested weir was selected for spillway length determination. A stage discharge curve for this spillway is shown as Figure No. VI-2.

The upper portion of the auxiliary spillway will be located on the western side of the planned borrow area. This location will allow for a straight alignment of the spillway and chute centerline. Flows from the auxiliary spillway will enter Shell Creek below Shell Reservoir. It should be noted that the access road to Lake Adelaide will unavoidably pass through the auxiliary spillway. A plan and section of the auxiliary spillway are shown on Figure No. VI-5.

The drop inlet spillway will consist of a 6-foot diameter inlet which converges into a 36-inch reinforced concrete drop pipe. The 36-inch drop pipe will be increased at the base to a 54-inch reinforced concrete pipe outlet tube which will extend outward well beyond the toe of the dam. The transition between the 6-foot inlet to the 36-inch drop pipe will be field formed and poured in-place. The base elbow will be encased entirely in concrete. The drop inlet spillway will be capable of passing 155 cfs with two feet of surcharge and 307 cfs under PMF flood conditions, with 7 feet of surcharge.

A stage discharge curve for the drop inlet spillway is shown as Figure No. VI-3. The spillway will function under submerged conditions above a 100-year release. The 54 inch discharge tube is designed to remain at less than full flow under all flow conditions enabling air passage up the discharge tube to prevent formation of subatmospheric pressures. A Manning "n" value of 0.014 was used in pipe friction calculations. The 6-foot inlet will be fitted with an anti-vortex baffle and trash rack as shown in Figure VI-6.

B. OUTLET WORKS

1. Existing Outlet

The existing outlet conduit is made up of 30-inch reinforced concrete pipe which is in poor condition. Numerous cracks and spalled concrete have been noted at several locations along the conduit interior. Water has been noted seeping into the pipe through cracks and holes at several locations. Flow control within the pipe is provided by a vertical slide gate which is stem operated from above the dam. The existing outlet is in marginal condition at present and could develop serious operating problems with further deterioration of the pipe. The entire conduit is in need of replacement. The existing gate appears to be in fair condition, but badly in need of a replacement seal. Also, the inlet wing walls have collapsed inward, partially blocking the inlet.

2. New Facilities

A new outlet works is required consisting of a cut and cover conduit placed in the till near the base of the left abutment. Depending upon which spillway alternative is selected, the outlet pipe would consist of 36-inch reinforced concrete pipe extending beneath the entire embankment or connecting into the 54-inch pipe of the drop inlet spillway outlet. Flow control through the 36-inch outlet will be provided by a sluice gate mounted on the upstream end of the pipe. The gate will be stem operated from the top of the dam embankment. A trash rack will be placed immediately upstream of the sluice gate. The trash rack, sluice gate and outlet conduit are shown on Figure No. VI-4 and VI-6.

The 36-inch outlet pipe is sized to carry 120 cfs with 12 feet of water depth above the pipe invert. Maximum flow through the outlet pipe is set at 190 cfs to coincide with a maximum reservoir water level of 9,280 feet.

Selected bedding for the outlet pipe, to be derived from the minus 3-inch fraction of decomposed granite material in the borrow area, will be placed 2 feet around and over the pipe to prevent undesirable contact of the pipe with boulders. The outlet pipe will be extended approximately 40 feet beyond the toe of the embankment slope and will terminate in a headwall structure that is riprapped to prevent erosion of the structure. Due to the erosion resistant grading characteristics of the bedding material, in addition to the relatively low hydraulic head and imposed load conditions on the outlet pipe, no cutoff collars or concrete cradles are considered necessary.

As shown in the plan views on both Figure No. VI-4 and Figure No. VI-6, the invert elevation of the outlet pipe has been set at 9233.0, the same as the existing outlet pipe. The new alignment for the outlet pipe will be located between 80 feet and 100 feet west (left abutment direction) of the existing outlet. In order to maintain existing discharge requirements during the two year construction period, a temporary pipe will connect the existing outlet pipe to the planned outlet pipe. The alignment of the connector pipe will be decided during final design. Most likely, the connector pipe will include an elbow and short section of pipe approximately parallel to the crest of the raised dam. Once construction is complete, the entrance to the new outlet will be formed by cutting through the existing dam to establish a minimum 15-feet wide entrance channel. As discussed previously, all design alternatives have considered using the existing dam as a borrow source during the second year of construction. As shown on Figure No. V-7, for example, it is planned to remove the existing dam down to Elevation 9240 at the upstream toe contact and then slope the surface at 20:1 (horizontal to vertical). The resulting cut required for the entrance channel to the entrance to the outlet conduit will vary from a maximum of 7 feet near the entrance to daylight at the upstream side, a distance of approximately 120 feet.

C. BUCKLEY CREEK DIVERSION

1. Existing Diversion Structure

The existing diversion structure consists of a steel fabricated "L" shaped diversion dam placed across Buckley Creek. The long portion of the "L" serves to dam the flow in the creek and to force flow toward the short portion of the "L". The short portion of the "L" supports a steel irrigation gate which directs water into an estimated 27-inch diameter pipe leading to the diversion ditch. The dimensions of the "L" shaped diversion structure are approximately 9 feet x 5 feet x 4 feet deep.

The "L" shaped structure appears capable of withstanding only minimal flood flows and would probably be washed out with flows in excess of 100 cfs. Adjacent to the diversion structure the diversion pipe discharges into a small pool which contains an 8-inch pipe for routing minimum flows back to Buckley Creek. From the small earth-lined pool water flows into an unlined canal which carries the flow to Mud Lake and then into Adelaide Lake.

2. Proposed New Structure

A gated reinforced concrete structure is proposed to divert water from Buckley Creek into a diversion ditch which will carry a 50 cfs flow into Mud Lake upstream from Adelaide Reservoir. The diversion structure will be located at the existing diversion site. The new structure has been designed to allow flows greater than 52 cfs to flow over a weir and to continue down Buckley Creek.

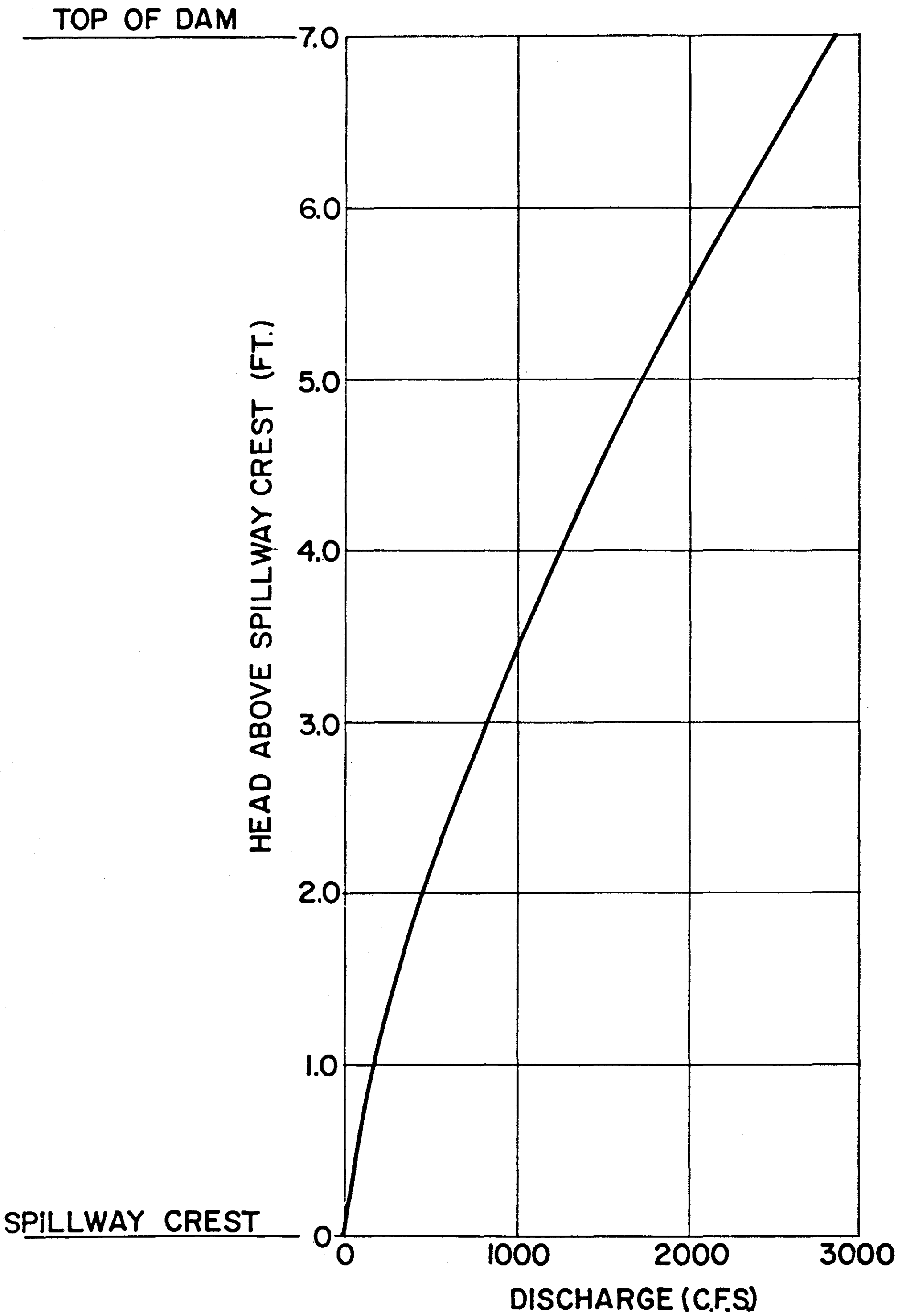
The structure is sized to accommodate the estimated 100-year flow on Buckley Creek of 600 cfs. Minimum flows up to 2 cfs will be diverted similar to the existing method and will be routed back to Buckley Creek through an 8-inch pipe. The culvert is designed to return all flows up to 1.3 cfs before any diversion occurs.

A drawing showing the proposed new Buckley Creek Diversion, stilling pool, minimum flow bypass pipe and diversion ditch entrance is shown on Figure No. VI-7.

D. ACCESS ROAD

The access road proposed makes maximum reuse of the existing access road. The roadway into Shell Reservoir will be improved in its' existing location as required for contractor access during construction. The proposed alignment between Shell Reservoir and Lake Adelaide was selected based upon discussion with Forest Service officials and is designed to avoid the designated wilderness area to the northwest.

The new roadway between Shell and Adelaide Reservoirs will be designed for maximum slopes of 10% except in one short stretch where a 12% grade will be used to avoid some large boulders. One new switch-back is being added to improve grades along the route. The roadway will have a 12-foot driving surface with 1 1/2:1 side slopes. Two large trees, six feet in diameter, will require removal along the route as well as several smaller ones. Improvements between Shell Reservoir and Lake Adelaide are shown on Figure No. VI-8.



50 FT. WIDE SIDE CHANNEL SPILLWAY

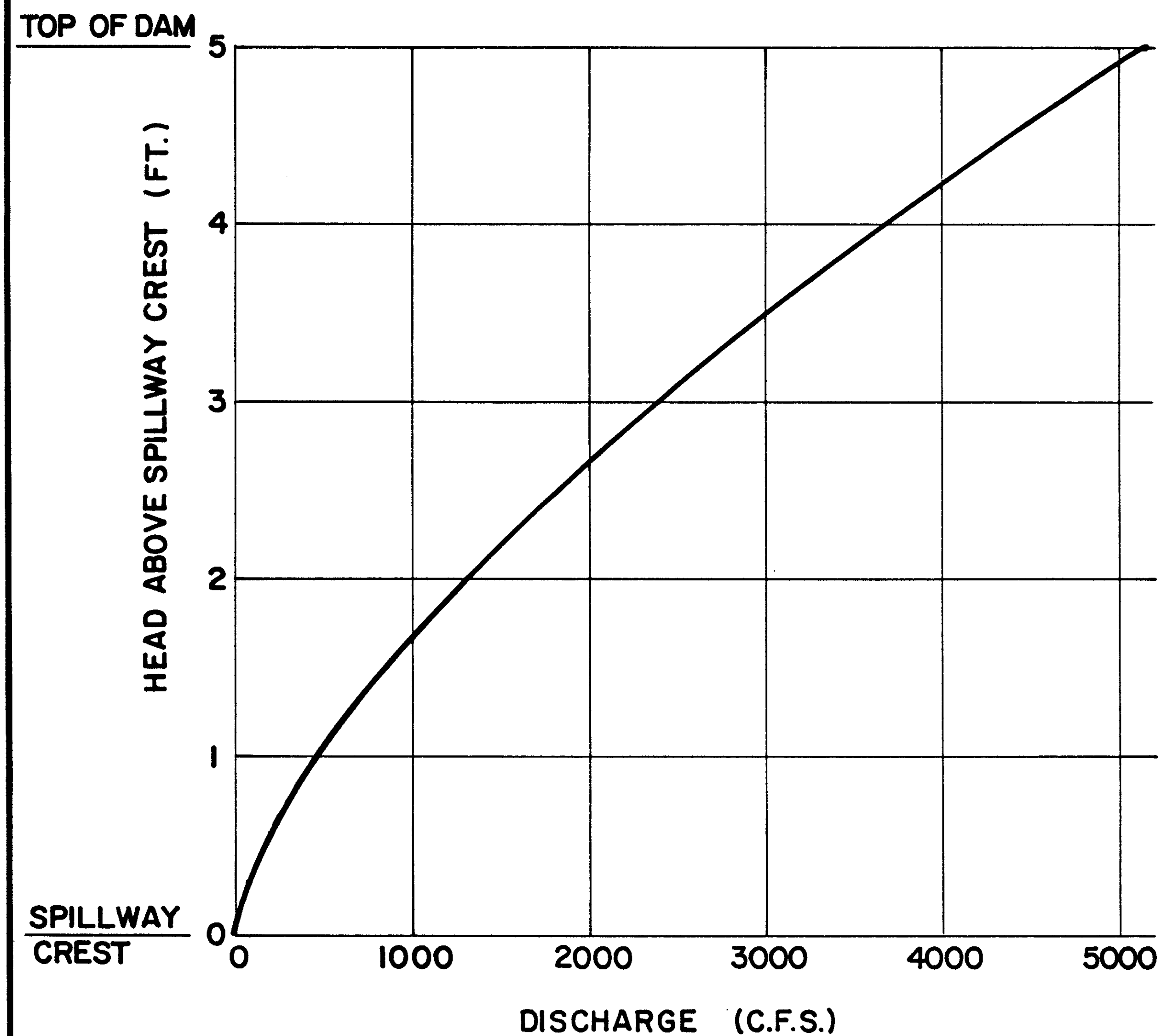
PREPARED BY ARIX

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY

SIDE CHANNEL SPILLWAY STAGE DISCHARGE CURVE

Checked by WRK	Date 10-23-85	Project No.	Figure No.
Approved by DDA	Date 10-23-85	3235	VI-1



150 FT. WIDE AUXILIARY SPILLWAY

PREPARED BY ARIX

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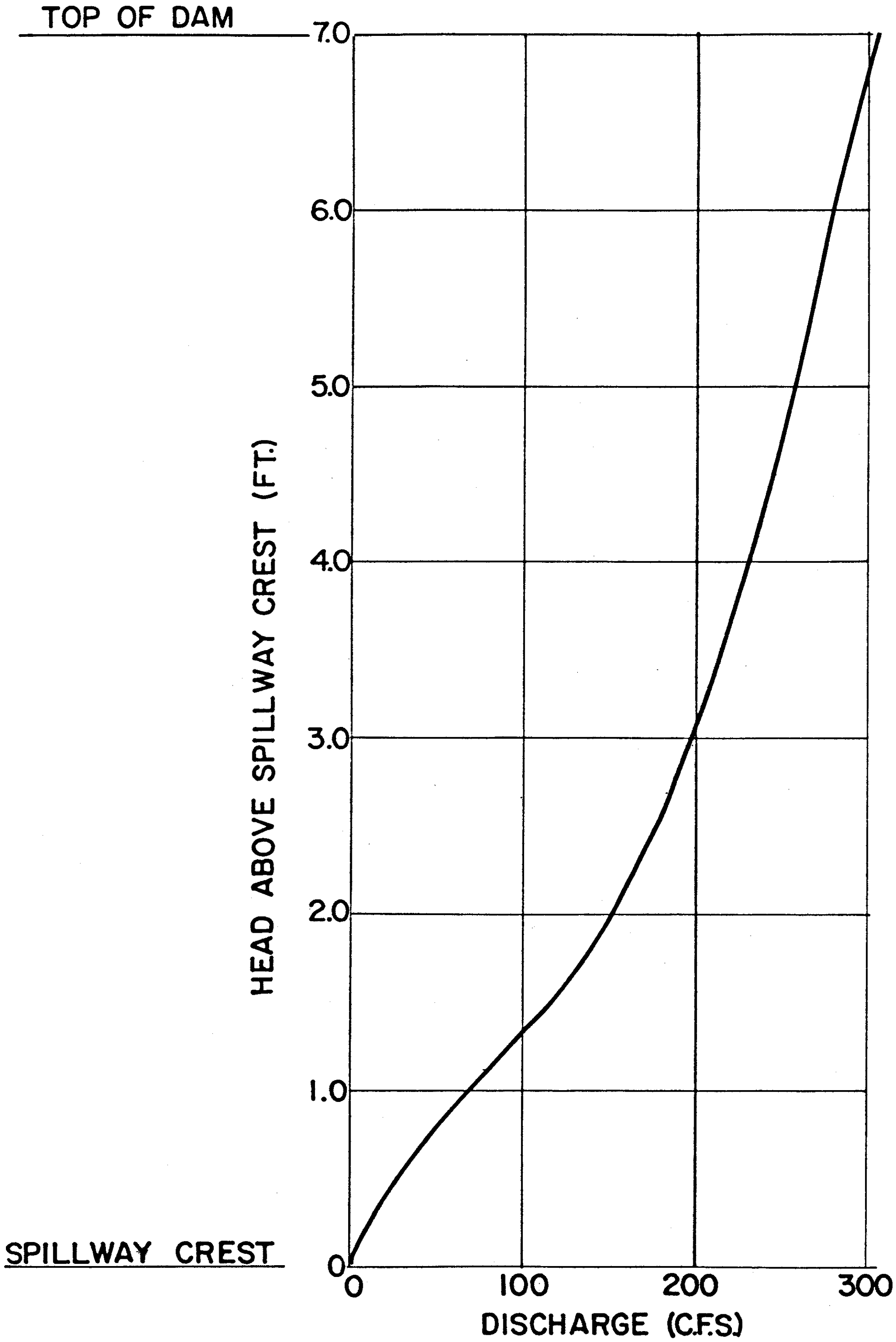
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED – LEVEL III STUDY
AUXILIARY SPILLWAY STAGE DISCHARGE CURVE

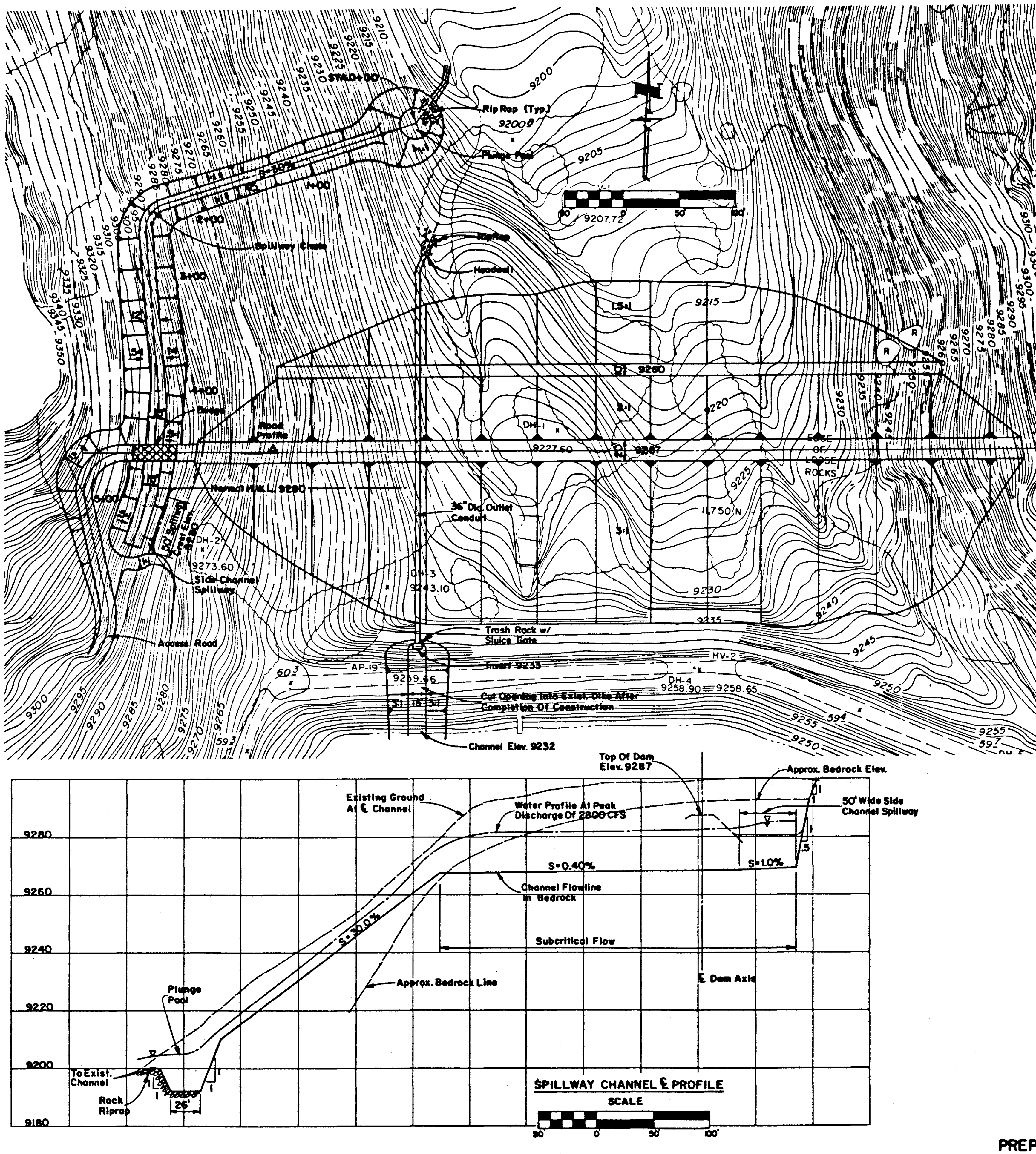
Checked by WRK	Date 10-23-85	Project No.	Figure No.
Approved by DDA	Date 10-23-85	3235	VI-2

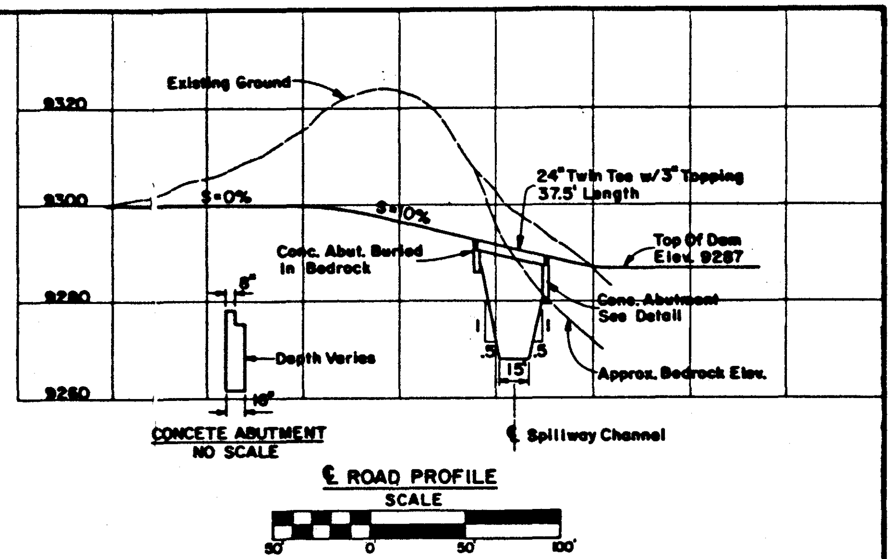


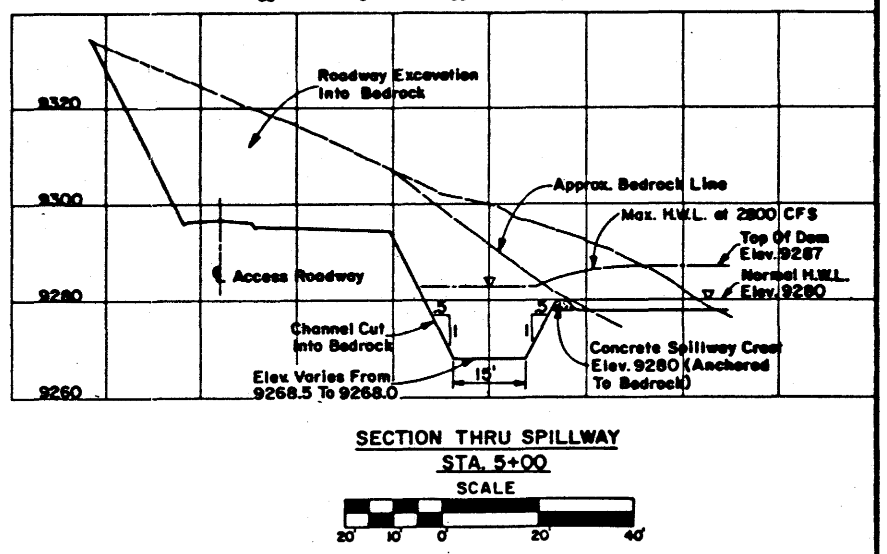
6 FT. DIAMETER DROP INLET SPILLWAY

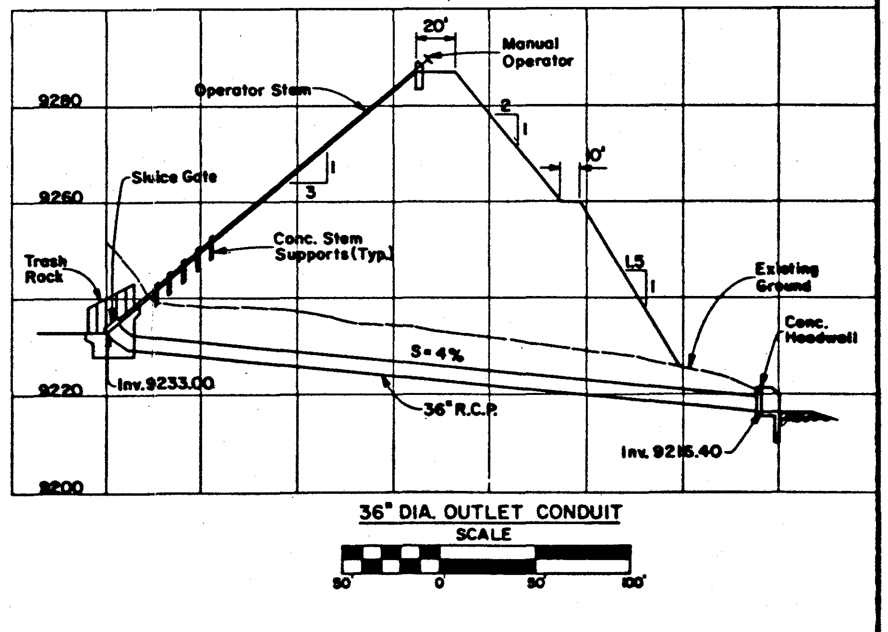
PREPARED BY ARIX

ESA Geotechnical Consultants Fort Collins, Colorado			
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY DROP INLET SPILLWAY STAGE DISCHARGE CURVE			
Checked by WRK Approved by DDA	Date 10-23-85 Date 10-23-85	Project No. 3235	Figure No. VI-3









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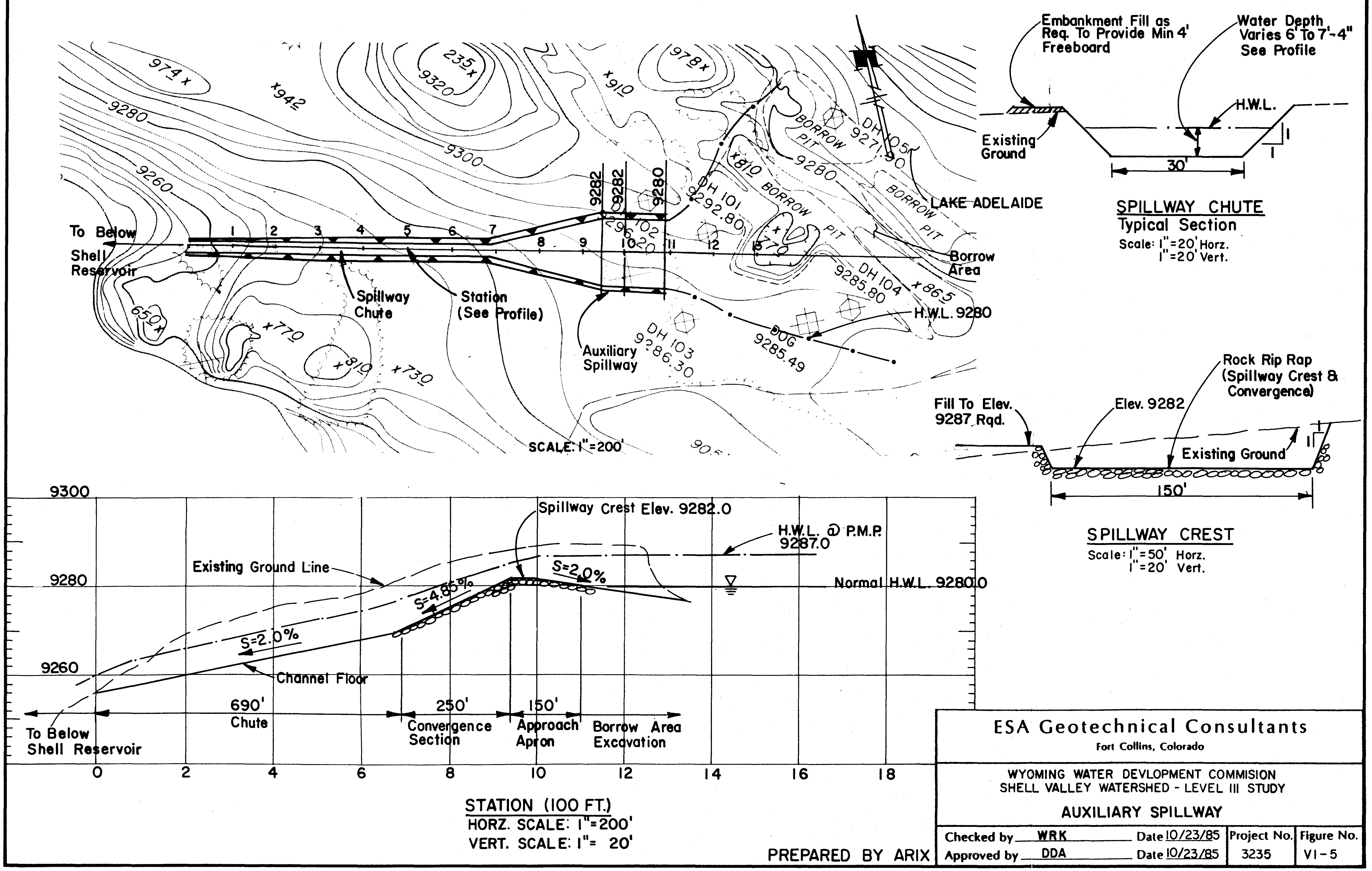
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY

SIDE CHANNEL SPILLWAY AND RESERVOIR OUTLET

Checked by WRK	Date 10/23/85	Project No. 3235	Figure No. VI-4
Approved by DDA	Date 10/23/85		

PREPARED BY ARIX



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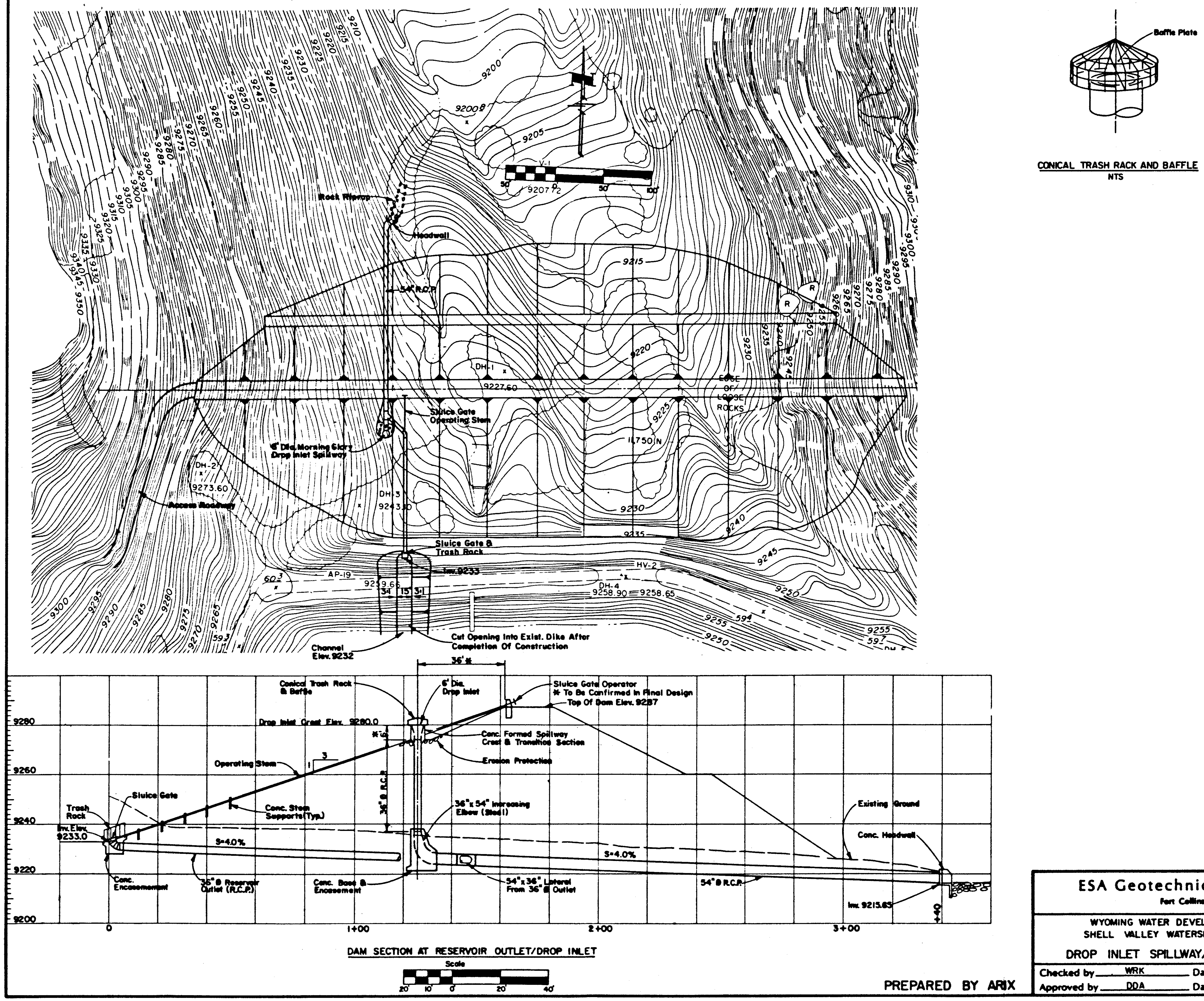
WYOMING WATER DEVLOPMENT COMMISION
SHELL VALLEY WATERSHED - LEVEL III STUDY

AUXILIARY SPILLWAY

Checked by WRK Date 10/23/85	Project No.	Figure No.
Approved by DDA Date 10/23/85	3235	VI-5



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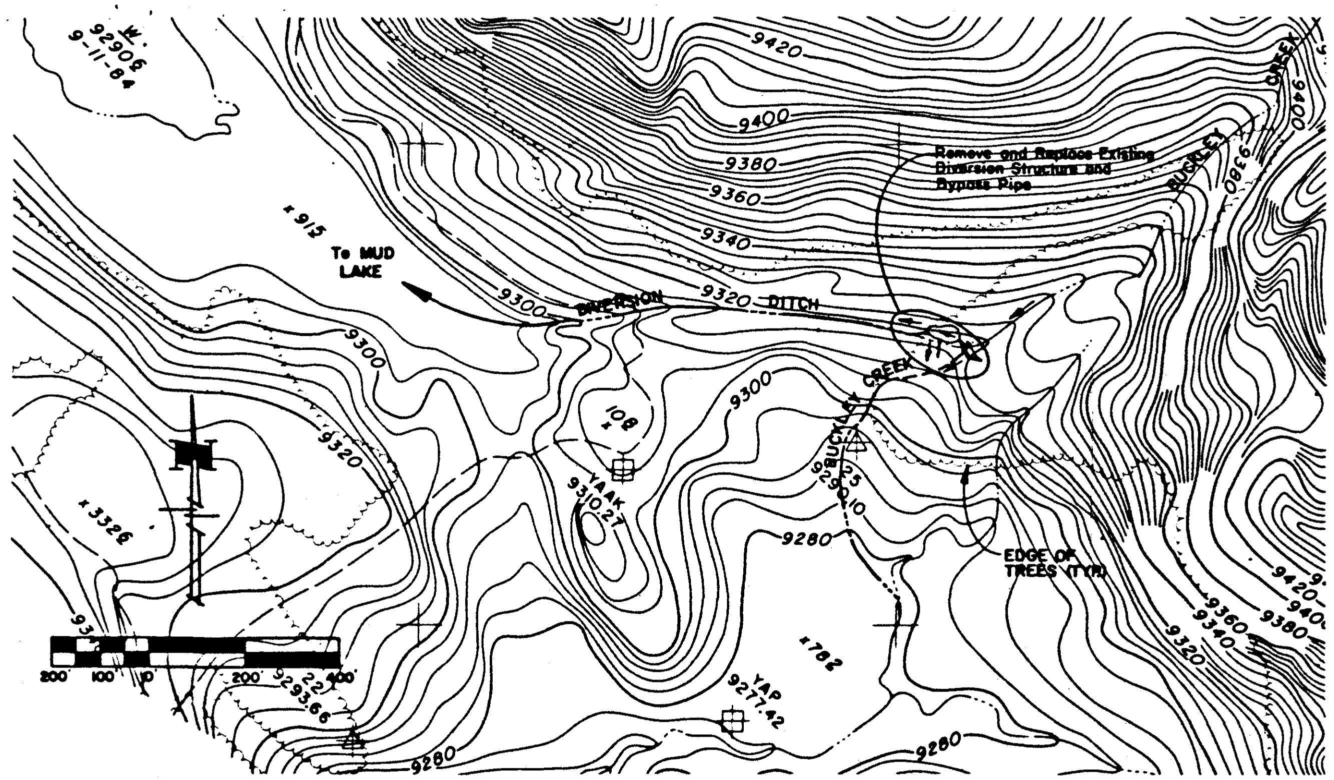
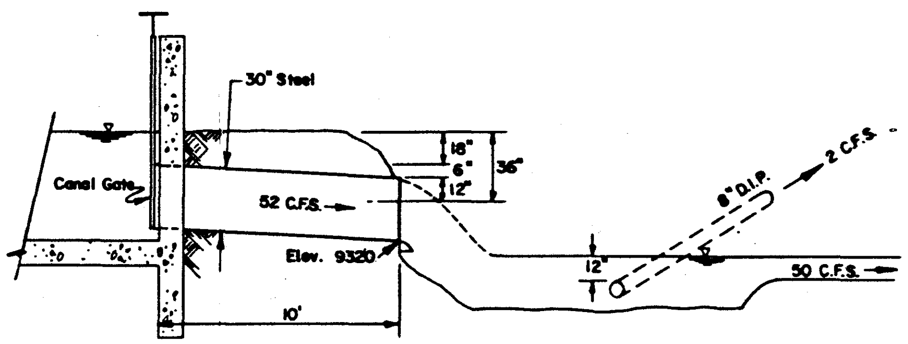
SHELL VALLEY WATERSHED - LEVEL III STUDY

DROP INLET SPILLWAY/RESERVOIR OUTLET

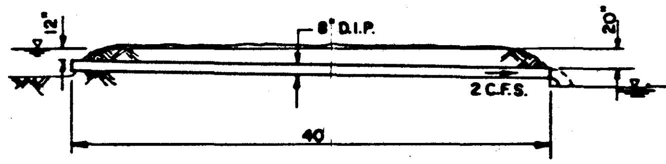
Checked by WRK	Date 10/23/85	Project No.	Figure No.
Approved by DDA	Date 10/23/85	3235	VI-6



BUCKLEY CREEK DIVERSION
LOCATION MAP



CULVERT DETAIL
Scale: 1/4" = 1'-0"



CULVERT DETAIL
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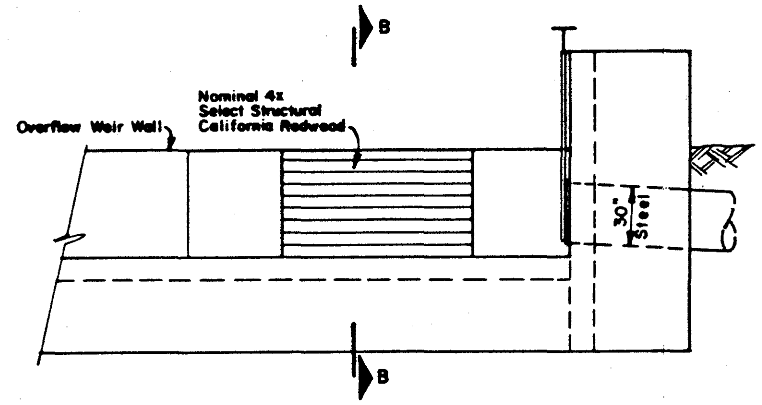

SECTION A-A
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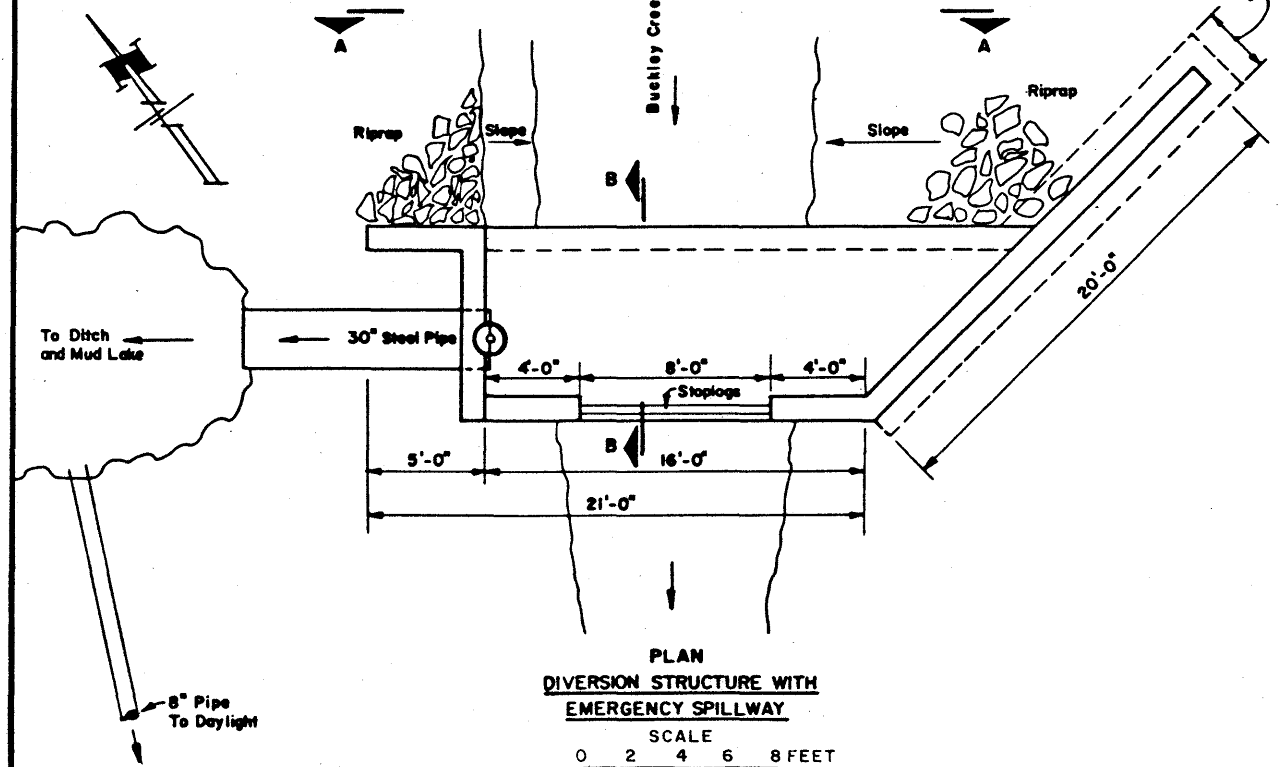
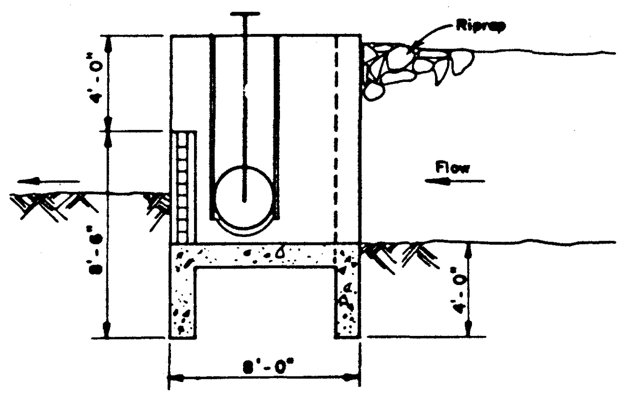


PLAN
DIVERSION STRUCTURE WITH EMERGENCY SPILLWAY

SCALE
0 2 4 6 8 FEET



SECTION B-B
Scale: 1/4" = 1'-0"

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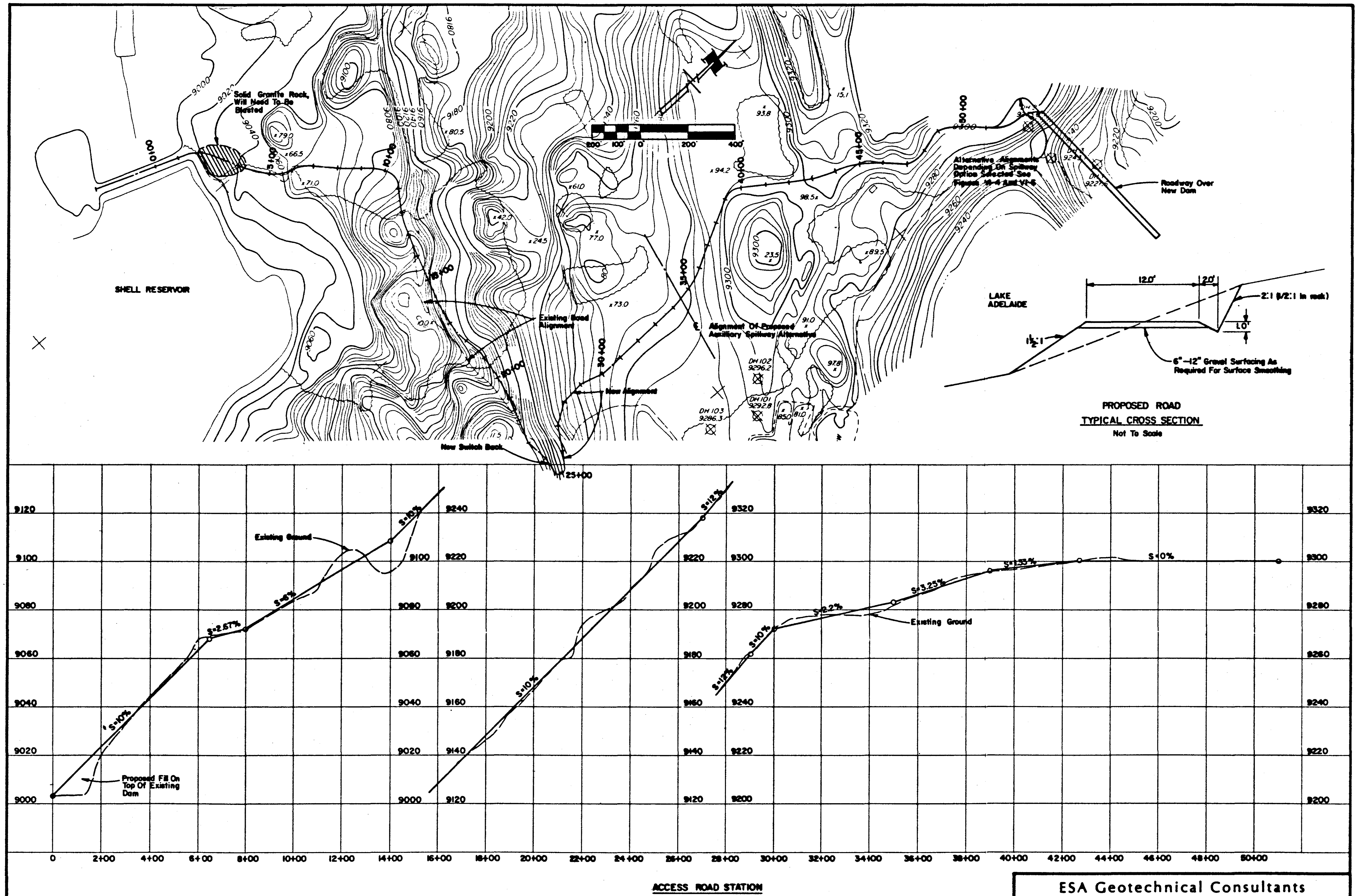
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY

BUCKLEY CREEK DIVERSION

Checked by WRK	Date 10-23-85	Project No. 3235	Figure No. VI-7
Approved by DDA	Date 10-23-85		



PREPARED BY ARIX

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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY

LAKE ADELAIDE ACCESS ROAD

Checked by WRK/CDH	Date 10-23-85	Project No. 3235	Figure No. VI-8
Approved by DDA	Date 10-23-85		

VII. CONSTRUCTION COST ESTIMATES

A. INTRODUCTION

The construction cost estimates presented in this section were completed based on the various design options for the overall project as discussed in Chapter V, Embankment Design and Chapter VI, Hydraulic Structures and Access Road. Estimated construction costs for each element of required construction were developed and then combined to provide total project costs.

B. BASIS OF THE CONSTRUCTION COST ESTIMATE

The construction cost estimates for this project were developed in the same manner that a contractor would to prepare a bid estimate. The proposed site was visited with attention given to the overall features of the left and right abutment, dam location, borrow sources, outlet works, and access road. Of particular concern is the access road and its upgrading to a point that heavy equipment can be brought into the site.

After the various bids items and quantities were derived, the unit costs were computed on expected production rates commensurate with the appropriate labor, equipment, material and overhead costs. These costs also take into account the elevation, short work season, and the remoteness of the project site. All costs are based on 1985 mid-summer rates. The following is a recap of what the aforementioned items consist of:

Labor Costs: Labor rates are based on the labor classifiications as published in the Wage Determination Decision rendered on August 1, 1985, for the Fourth Judicial District, State of Wyoming. The labor rates used include Unemployment Compensation, Workman's Compensation, FICA, and employee benefits.

Equipment Costs: An equipment rental rate was established for each major item of equipment which would be utilized. The rental rate includes fuel, maintenance and repairs, and operating costs.

Material Costs: Material suppliers were contacted to obtain current prices for the various materials required.

Overhead Costs: The overhead rate used for this project is slightly higher than a contractor would normally use due to the location of the project. An unknown in this area is predicting future insurance costs.

Profit: A higher rate of profit was applied than normal because of the risk factor and remoteness of the project.

Production rates were established for each item of work and the estimated number and types of equipment were assigned to these items based on cycle times, load capabilities, etc. Based on these factors the amount of labor hours and equipment usage was determined and was factored together to arrive at the estimated cost for the project.

C. ESTIMATED COST OF INDIVIDUAL PROJECT ELEMENTS

1. Dam Construction

The estimated direct cost of construction for Alternatives A, B and C are presented on Tables VII-1 through VII-3. Alternative A, which would provide a dam to retain a 3,500 acre-feet reservoir, is shown to have a total direct cost of $1,010,555. A potential savings of about $68,000 can be realized if the reservoir level during the second year of construction can be regulated to enable using the existing dam as a borrow source. This possibility must be evaluated by the SVWID and, if selected, would be incorporated in the project specifications.

Alternative B and Alternative C cost estimates indicate that either option can be built for about $1,200,000. Both of these alternates will provide the maximum possible reservoir storage of about 4,550 acre-feet. The same potential savings of about $68,000 for utilizing the existing dam as a borrow source also occurs for the B and C design alternative. It will be noted that a $100,000 mobilization cost has been applied to the cost of dam construction. A separate mobilization cost was considered for the Buckley Creek Diversion. No mobilization cost has been considered for the hydraulic structures since an appropriate figure is already included for dam construction.

2. Side Channel Spillway and Outlet Facilities Combined

The estimated cost of providing outlet facilities and a left abutment side channel spillway is presented on Table VII-4. As shown, the estimated direct cost for these facilities is $216,000. It will be noted that $60,560 of this total is associated with providing vehicle access across the spillway to the crest of the dam. A far less expensive option may be desired by the sponsors, if such direct access is not required.

3. Morning Glory Spillway and Outlet Facilities Combined

Table VII-5 presents the estimated direct cost of providing a Morning Glory Spillway and outlet facility combined as a continous structure. As shown, the estimated direct cost will be $216,600.

4. Auxiliary Spillway

The estimated cost of constructing an auxiliary spillway is presented on Table V-6 and is shown to be $111,500. It should be noted that the cost of the auxiliary spillway, which has been designed to pass the full PMF should be added to the cost of the Morning Glory Spillway option since both of these spillway structures would be required in tandem.

5. Buckley Creek Diversion

The estimated cost of the Buckley Creek Diversion is $24,850 as shown on Table VH-7. Due to the remoteness of the area, an additional mobilization cost of $3,000 has been included for this work.

6. Access Road

The cost of improving the 4 miles of access road to the project site has been estimated at $60,000, which correlates to a cost of $15,000 per mile. Details regarding the exact improvement scheme can not be developed until the U.S. Forrest Service has completed an environmental assessment of the proposed project. This cost assumes that road improvements will be made under a separate contract and during the summer prior to the start of dam construction.

D. ESTIMATED COST OF TOTAL PROJECT

The estimated cost of the total project in 1985 dollars is presented below. As a result of the direction that the design process followed, estimated total project costs are presented for two alternatives. Dam Alternative A is the minimum cost to provide a reservoir storage of 3,500 acre-feet, which is essentially the project envisioned from the Level II study. Dam Alternative B or C reflects the minimum estimated cost of providing a reservoir storage of 4,500 acre-feet. The latter reflects the largest practical reservoir capacity at the Adelaide Creek site.

1. Dam Alternative A

The minimum direct cost of the required project facilities for Dam Alternative A, is $1,332,000 as shown on Table VII-8. A potential savings of $128,600 can be realized if the road and bridge across the spillway are deleted and the existing embankment is used as a borrow source. This option will provide a new reservoir capacity of 3,500 acre-feet, an increase of 1,800 acre-feet over the existing capacity. The approximate cost for the project expansion, therefore, is about $740 per acre-foot of storage increase, neglecting potential savings.

It should be noted that the costs presented for Alternative A do not reflect the additional cost which would be associated with routing a full PMF event with this configuration. The costs developed for Alternative A were based on the assumption that routing only up to approximately one-half of the PMF through the side channel spillway alone would be acceptable.

2. Dam Alternative B or C

The minimum estimated total direct cost for the required project facilities to provide the maximum practical storage at the site is on Table VII-9. The two options costed differ in their spillway configurations. Option 1 is recommended and includes a side channel spillway coupled with an emergency spillway. The cost of the emergency spillway is roughly equivalent to the final grading and revegetation costs required in the borrow area for all the alternatives. Option 2, which incorporates a drop inlet and auxiliary spillway, is more costly and is more difficult to construct. The approximate minimum cost for Option 1 is $1,400,000 or about $535 per acre-foot of storage increase.

E. SUMMARY OF ESCALATED PROJECT COSTS

A summary of the escalated project costs for construction commencing in 1987 for Dam Alternative A are shown in Table VII-10 and for Dam Alternative B or C in Table VII-11. As shown in the referenced tables, a contingency of 15 percent was applied to the estimated total direct costs. These total costs were then escalated by 7 percent per year compounded (15%) for the two year delay before construction could begin in 1987.

In addition to the escalated total direct cost, we have also included other costs related to construction. The cost of field engineering, surveying, construction control and office engineering support during construction has been estimated at 10 percent of the toal construction costs. Also, a cost for contract administration equal to 5 percent of total construction cost has also been included.

As shown on Table VII-10, the total construction cost for a facility to provide 3,500 acre-feet of storage is $1,800,000. A larger facility to provide a maximum of 4,500 acre-feet of storage has a total construction cost estimated at $2,100,000.

TABLE VII-1

ESTIMATED DIRECT COST OF DAM CONSTRUCTION
ALTERNATIVE A

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
Mobilization	LUMP SUM	--	--	$100,000
Foundation Clearing and Preparation	131,800	SF	$ .15	19,770
Left Abutment				
Clearing	10,700	SF	.05	535
Excavation	4,800	CY	3.00	14,400
Surface preparation	20	CY	300.00	6,000
Right Abutment				
Surface preparation	175	CY	300.00	52,500
Embankment				
Zone 1	140,600	CY	4.75	667,850
Zone 2	3,500	CY	7.00	24,500
Zone 3	16,000	CY	5.75	92,000
Zone 4	3,300	CY	10.00	33,000
Zone 5	34,000	CY	2.75	--
	TOTAL DIRECT COST			$1,010,555

NOTE: The cost savings for this alternative if the Zone 5 material can be used in Zone 1 is $68,000.

TABLE VII-2

ESTIMATED DIRECT COST OF DAM CONSTRUCTION
ALTERNATIVE B

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
Mobilization	LUMP SUM	--	--	$100,000
Foundation clearing	146,100	SF	$ .15	21,915
Left Abutment				
Clearing	15,000	SF	.05	750
Excavation	6,600	CY	3.00	19,800
Surface preparation	28	CY	300.00	8,400
Right Abutment				
Surface preparation	225	CY	300.00	67,500
Embankmkent				
Zone 1	128,600	CY	4.75	610,850
Zone 2	6,000	CY	6.10	36,600
Zone 3	58,400	CY	5.20	303,680
Zone 4	4,600	CY	10.00	46,000
Zone 5	34,000	CY	2.75	--
	TOTAL DIRECT COST			$1,215,495

NOTE: The cost savings for this alternative if the Zone 5 material can be used in Zone 1 is $68,000.

TABLE VII-3

ESTIMATED DIRECT COST OF DAM CONSTRUCTION
ALTERNATIVE C

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
Mobilization	LUMP SUM	--	--	$100,000
Foundation clearing	138,400	SF	$ .15	20,760
Left Abutment				
Clearing	13,300	SF	.05	665
Excavation	5,900	CY	3.00	17,700
Surface preparation	25	CY	300.00	7,500
Right Abutment				
Surface Preparation	205	CY	300.00	61,500
Embankment				
Zone 1	172,600	CY	4.75	819.850
Zone 2	4,000	CY	7.00	28,000
Zone 3	18,600	CY	5.75	106,950
Zone 4	3,300	CY	10.00	33,000
Zone 5	34,000	CY	2.75	--
	TOTAL DIRECT COST			$1,195,925

NOTE: The cost savings for this alternative if the Zone 5 material can be used in Zone 1 is $68,000.

TABLE VII-4

ESTIMATED DIRECT COST OF SIDE CHANNEL
SPILLWAY AND OUTLET FACILITIES COMBINED

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
A. Side Channel Spillway				
Common excavation				
1) Till material	6,700	CY	$ 2.50	$16,750
2) Bedrock	6,200	CY	10.00	62,000
3) Road across spillway	2,600	CY	10.00	26,000
Bridge	576	SF	60.00	34,560
Concrete wier cap	12	CY	500.00	6,000
Rock anchors	6	EA	200.00	1,200
Riprap	400	CY	10.00	4,000
B. Outlet Facilities				
36-in RCP w/bedding	340	LF	90.00	30,600
36-in RCP elbow	2	EA	1200.00	2,400
Concrete headwall	6	CY	500.00	3,000
Concrete support at sluice gate and trash rack	14	CY	500.00	7,000
36-in sluice gate with with appurtenances	1	EA	16000.00	16,000
Trash rack	1	EA	2000.00	2,000
Concrete stem supports	17	EA	200.00	3,400
Riprap	175	CY	10.00	1,750
			TOTAL DIRECT COST	$216,660

TABLE VII-5

ESTIMATED DIRECT COST OF MORNING GLORY SPILLWAY AND OUTLET FACILITIES COMBINED

ITEM		QUANTITY	UNIT	UNIT COST	TOTAL
A.	Morning Glory Spillway				
	6-ft diameter concrete inlet	5	CY	$ 700	$ 3,500
	Conical Trash Rack	1	EA	2,500	2,500
	36-in. RCP	40	LF	80	3,200
	36-in. x 54-in. Steel Elbow	1	EA	7,500	7,500
	Concrete Elbow Support	40	CY	500	20,000
B.	Reservoir Outlet				
	36-in. RCP	150	LF	90	13,500
	36-in. Elbow	2	EA	1,200	2,400
	Concrete Support at Sluice Gate and Trash Rack	14	CY	500	7,000
	36-in. Sluice Gate with Appurtenances	1	EA	16,000	16,000
	Trash Rack	1	EA	2,000	2,000
	54-in. x 36-in. Lateral	1	EA	2,100	2,100
C.	Discharge Facility				
	54-in. RCP	208	LF	150	31,200
	Concrete Headwall	10	CY	500	5,000
	Riprap	25	CY	10	2,500
			TOTAL DIRECT COST		$118,400

TABLE VII-6

ESTIMATED DIRECT COST OF AUXILIARY SPILLWAY

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
Excavation and Shaping	28,000	CY	$ 3.00	$ 84,000
Riprap	2,750	CY	10.00	27,500
		TOTAL DIRECT COST		$ 111,500

TABLE VII-7

ESTIMATED DIRECT COST OF BUCKLEY CREEK DIVERSION

ITEM	QUANTITY	UNIT	UNIT COST	TOTAL
Mobilization	Lump Sum	--	--	$ 3,000
Cleanup and Creek Realignment	Lump Sum	--	--	2,000
Excavation & Backfill	Lump Sum	--	--	750
Concrete	25	CY	$ 600	15,000
Stoplogs	Lump Sum	--	--	1,000
Riprap	30	CY	10	300
Canal Gate	Lump Sum	--	--	1,500
30-in diameter pipe	10	FT	50	500
8-in diameter pipe	40	FT	20	800
		TOTAL DIRECT COST		$ 24,850

TABLE VII-8

MINIMUM DIRECT COST OF PROJECT FACILITIES
TO PROVIDE RESERVOIR STORAGE OF 3500 ACRE FEET
(DAM ALTERNATIVE A)

PROPOSED FACILITY	ESTIMATED DIRECT COST
Side Channel Spillway and Outlet Facilities	$ 216,600
Dam Enlargement (Alternative A)	1,010,555
Buckley Creek Diversion	24,850
Access Road	60,000
Final Grading and Revegetation	20,000
Total Direct Cost	$ 1,322,005
Potential Savings	
Delete road and bridge across spillway	$ 60,560
Use of existing dam as Zone 1	68,000
Total Potential Savings	$ 128,560
MINIMUM DIRECT COST	$ 1,203,445

TABLE VII-9

MINIMUM DIRECT COST OF PROJECT FACILITIES
TO PROVIDE RESERVOIR STORAGE OF 4500 ACRE FEET
(DAM ALTERNATIVGE B OR C)

PROPOSED FACILITY	ESTIMATED DIRECT COST
Option 1	
Side Channel Spillway & Outlet Facilities	$ 216,600
Dam Enlargement (Alternative B)	1,215,495
Buckley Creek Diversion	24,850
Access Road	60,000
Final Grading and Revegetation (Emergency Spillway Area)	20,000
Total Direct Cost	$ 1,536,945
Potential Savings	
Delete Road and Bridge Across Spillway	60,560
Use of existing dam as Zone 1	68,000
Potential Savings	$ 128,560
MINIMUM DIRECT COST	$ 1,408,385
Option 2	
Auxiliary Spillway	$ 111,500
Morning Glory Spillway and Outlet Facilities Combined	118,400
Dam Enlargement (Alternative B)	1,215,495
Buckley Creek Diversion	24,850
Access Road	60,000
Final Grading and Revegetation	20,000
Total Direct Cost	$ 1,550,245
Potential Savings	
Use of Existing Dam as Zone 1	$ 68,000
MINIMUM DIRECT COST	$ 1,482,245

TABLE VII-10

SUMMARY OF ESTIMATED PROJECT COSTS
TO PROVIDE RESERVOIR STORAGE OF 3500 ACRE-FEET
(DAM ALTERNATIVE A)

COST ELEMENT	ESTIMATED COST
Minimum Direct Cost (1985)	$ 1,203,500
Contingency (15%)	180,500
Subtotal	$ 1,384,000
Escalation to 1987 (15%)	$ 207,600
Total 1987 Construction Costs	$ 1,591,600
Other Construction Related Cost	
Engineering During Construction (10%)	$ 159,200
Contract Administration (5%)	80,000
TOTAL CONSTRUCTION COST	$ 1,830,800

TABLE VII-11

SUMMARY OF ESTIMTED PROJECT COSTS
TO PROVIDE RESERVOIR STORAGE OF 4500 ACRE-FEET
(DAM ALTERNATIVE B OR C)

COST ELEMENT	ESTIMATED COST
Minimum Direct Cost (1985)	$ 1,408,500
Contingency (15%)	211,500
Subtotal	$ 1,620,000
Escalation to 1987 (15%)	$ 243,000
Total 1987 Construction Cost	$ 1,863,000
Other Construction Related Cost	
Engineering During Construction (10%)	$ 186,300
Contract Administration (5%)	93,000
TOTAL CONSTRUCTION COST	$ 2,142,300

VIII. CONCLUSIONS AND RECOMMENDATIONS

A. CONCLUSIONS

The following conclusions have been reached regarding the conceptual design studies described in this report.

Geotechnical Considerations

1. Interpretations of existing and new field data revealed no geologic conditions that would constitute a "fatal flaw" in the planned enlargement of Lake Adelaide reservoir. No evidence was found that the boulder field on the right abutment is the result of a landslide, as thought possible during the Level II studies. Instead, detailed field mapping, aerial photo interpretation and drilling indicates the boulder field is the result of stress relief (glacial unloading) along widely spaced fractures and freeze-thaw cycles. Preserved joint structures indicate the boulders are essentially in place with little migration downslope. The boulders are 20 to 25 feet thick along the dam axis, underlain by competent granitic bedrock that exhibits little weathering.

2. New field data based on multiple well pumping tests, indicates that the till deposits forming the foundation of the dam in the channel section and left abutment areas have a relatively low permeability. As a result, expensive seepage cutoff measures are not needed to control reservoir seepage losses. However, there is a possibility that small undetected zones of high permeability may exist within the foundation. These zones, if present, would not threaten the safety of the dam. If higher than anticipated reservoir losses are sustained, they can be controlled by later upstream blanketing or chemical grouting. Computed reservoir losses of up to 250 acre-feet per year can be easily accomodated. Most of this seepage would occur during normal releases from the reservoir after the spring runoff. Reservoir rim seepage losses should not be significant even under worst case scenerios.

3. The right abutment boulder field is about 20-25 feet in maximum thickness at the proposed contact area of the dam and is underlain by competent granitic bedrock. It will be necessary to remove this rock by blasting in order to expose a relatively smooth

contact surface to form the right abutment. In order to limit the possibility of excessive seepage occurring along the embankment abutment contact, provisions will be incorporated in the plans and specifications to provide a compacted layer of relatively plastic clayey sand to blanket the abutment before placement of the less plastic embankment material.

4. The area in and around the original borrow area was explored and found to have an ample supply of materials to adequately construct a new dam. The materials contain about 25 percent by volume of cobbles and boulders greater than 6 inches in maximum size. It will be necessary to segregate this oversize material for use in the rockfill zone. The construction materials exhibit an excellent shear strength when compacted.

5. The seisimicity of the region is considered to be low to moderate, although the seismicity could be significantly higher than generally thought as new evidence is accumulated for this relatively unknown area. Pseudo static stability analysis indicate that all of the alternative embankment designs will easily withstand any major earthquakes that can be reasonably expected to occur in the site area.

Hydrologic Considerations

6. A hydrologic analysis of the Adelaide Creek and Buckley Creek drainage areas indicates that the optimal reservoir storage capacity is 4500 acre-feet.

7. By increasing the Lake Adelaide storage capacity to 4500 acre-feet, Shell Canal will be able to meet more than 80 percent of the diversion requirements 98 percent of the time. One hundred percent of the diversion water demands will be satisfied 68 to 73 percent of the time, depending upon environmental operating constraints such as minimum pool and instream flow requirements.

8. Analyses indicated the design floods will be a 100-year storm event resulting in a peak inflow to the reservoir of 665 cfs, and the probable maximum flood (PMF) event resulting in a peak inflow of 6765 cfs.

Dam Design Alternatives

9. Three embankment alternatives are proposed, all of which are constructable and will result in safe economical structures under all critical loading conditions. A special series of seepage analyses were performed and these results indicate that seepage cutoff measures in the foundation are not required. Embankment and foundation seepage will be collected and controlled by a filter and drain zone.

10. Due to the relatively short construction season at the project site, a two year construction period will be required. The design will permit continued operation of the reservoir during construction. However, it will be necessary during the second construction season to operate the existing reservoir so that it will be completely drawn down during the last 4 to 8 weeks of the construction season. This will permit the use of the existing embankment as a borrow source and to finish the outlet works.

Hydraulic Structures

11. Two different spillway combination schemes have been developed to route the PMF event through Lake Adelaide. One scheme utilizes a side channel spillway on the left abutment which is designed to handle up to approximately 50 percent of the PMF. The remaining 50 percent of the PMF will be routed using the side channel in tandem with a low cost emergency spillway located in the borrow area.

The second scheme utilizes a drop inlet (Morning Glory) spillway, which ties directly into the outlet works. The drop inlet spillway is designed to pass flood flows up to the 100 year storm on its own. Flows in excess of the 100-year storm will be routed through an auxiliary spillway constructed in the borrow area. The auxiliary spillway is designed to handle nearly the full PMF outflow and is therefore more difficult to construct and more costly than the emergency spillway described above.

12. It is concluded that an entirely new outlet works is needed to provide both reliable performance and adequate capacity. The new outlet will be located about 80 to 100 feet west of the existing outlet in the left abutment area.

13. Because of potentially unstable stream channel conditions during a large flood event, a relatively simple diversion structure on Buckley Creek was judged to be appropriate at this site. However, considerable repairs may be necessary after a major flood event. The structure as designed, will divert the required flow for optimum reservoir yield.

Construction Costs

14. The optimum reservoir capacity of 4,500 acre-feet can be developed for an estimated construction cost of about 2.2 million dollars.

15. A reservoir capacity of 3,500 acre-feet will have an estimated construction cost of about 1.8 million dollars.

B. RECOMMENDATIONS

Based on the Level III studies presented in this report, and discussions with both the Shell Valley Watershed Improvement District and the Wyoming Water Development Commission, the following recommendations are provided with regard to proceeding with Phase 2 of the Level III studies:

1. It is recommended that the optimum reservoir storage capacity of 4548 acre-feet be developed because it provides the most cost effective and reliable water supply.

2. The Alternative B design configuration is recommended. This design allows for the optimum use of construction materials and provides the safest embankment in terms of structural stability.

3. The side channel and emergency spillway combination is recommended. The side channel spillway is designed to route up to one-half of a PMF event on its own. Therefore, the emergency spillway, which will probably never go into operation, can be adequately constructed by a minimal amount of earthwork in the borrow area.

4. We recommend that the existing outlet pipe be abandoned and the new 36-inch diameter outlet pipe be installed in the left abutment area. The new outlet pipe should be installed early during the first construction season and then connected to the existing outlet pipe by

a temporary pipe. This will enable controlled reservoir releases during construction. After the new dam is completed, the existing outlet pipe will be abandoned.

5. It is recommended that the access road be improved under a separate contract. The road should be constructed a year prior to the start of construction of the remainder of the project. The bidders should be shown the site after the road improvement has been completed.

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APPENDIX A

FIELD INVESTIGATIONS

APPENDIX A

FIELD INVESTIGATIONS

A. Surface Geologic Mapping

The geology of the dam site, reservoir rim, borrow area, and access road was mapped using 1" = 50′ to 1" = 200′ scale topographic maps. Aerial photographs were used to supplement ground observations. Particular emphasis was placed on physical characteristics and structural features that would affect foundation conditions, availability of construction materials, reservoir rim stability, and reservoir seepage potential. The rock slope on the right abutment was examined in detail. The contact between in-place bedrock and the loose rocks was examined and a tape and hand level survey was made of this abutment along the center line of the Alternate A dam layout. Using the results of this survey, a schematic cross section was developed and is presented in Figure III-3.

B. Seismic Refraction Survey

A seismic refraction survey was performed at the site and is discussed in Appendix C of this report.

C. Subsurface Exploration

A total of six exploratory borings and sixteen exploratory trenches were logged and sampled by ESA engineering geologists and geotechnical engineers from July 14 through August 15, 1985. The borings included four holes in the dam foundation area and two holes in the borrow-auxiliary spillway area. Bore hole depths varied from 29.0 to 52.0 feet. Six trenches were located in the dam foundation area and eight trenches were located in the borrow-auxiliary spillway area. Trench depths varied from 5.0 to 11.2 feet. The locations of the borings and trenches are shown on Figures III-1 and III-5 and detailed field drilling and sampling logs are presented following the text of this appendix.

Rotary drilling and diamond coring were performed using a Longyear 35 skid mounted drill rig which was moved using an Allis Chalmers DH-6 tractor. Hole diameters varied from 3" to 5 5/8". Two holes (DH-7 and DH-11) were completed as wells with 4" slotted steel casing. The other holes (DH-8, DH-9, DH-10 and DH-12) had 2" Schedule

40 plastic PVC placed in them. The exploratory trenches were excavated using a International Harvester rubber tired backhoe with a two foot wide bucket.

D. Permeability Tests

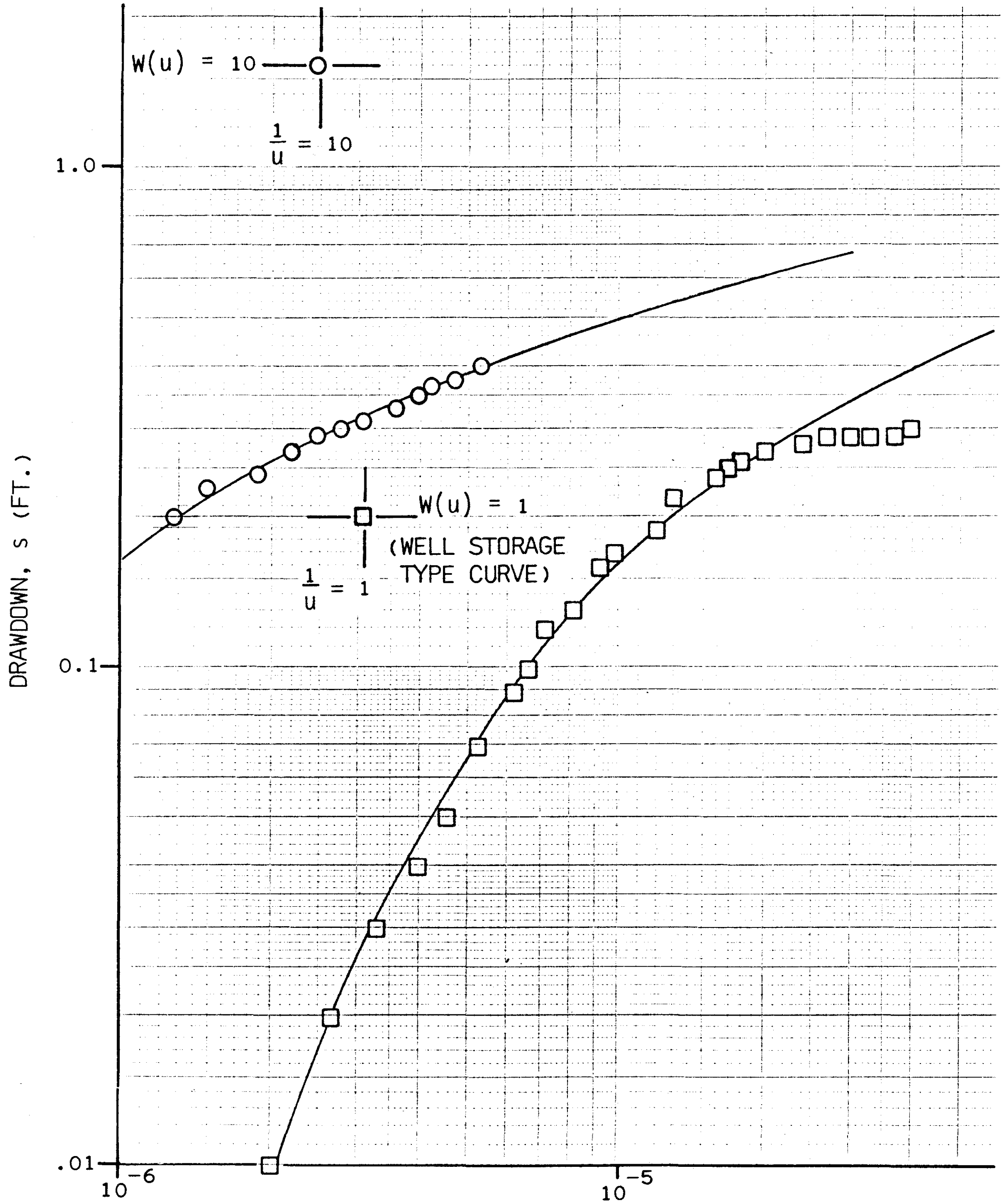
Water pressure tests were conducted in three of the exploratory boreholes (DH-8, DH-9, DH-10). The water pressure testing program utilized constant head pump-in tests with a single mechanical packer providing a seal at the desired interval. Holding tests were performed after flow during a packer test had stabilized and all necessary readings had been taken.

Multiple well pumping tests were performed at the dam site and auxiliary spillway-borrow area. DH-7 at the dam site was used as a pumping well and drawdown/recovery measurements were made in DH-8 and DH-1. Two constant discharge tests were performed at different discharge rates to provide a thorough analysis. A single constant discharge test was performed in the emergency spillway-borrow area using DH-11 as the pumping well with drawdown/recovery measurements recorded in DH-12.

Data plots for the pumping test results are presented in Figures A-1 through A-6. Two different analysis methods were used to evaluate the pump test data. The Theis Analysis method shown by Figures A-1 through A-3, is a graphical procedure for determining transmissivity values (permeability times the saturated thickness). The procedure involves matching actual measured data with theoretical type curves to determine the appropriate values for the Theis well function W(u), which is an expotential integral. The Theis well function varies with the Boltzmann variable (u), which is a function of t/r^2; where r is the distance between the pumped well and the observation well and t is time. Once the Theis well function is determined, the transmissivity can be calculated knowing the pump rate (Q) and the drawdown (s) corresponding to the match point. The match points and calculations for transmissivity are summarized in Figures A-1 through A-3 for Tests 1 and 2 in the foundation area and Test 3 in the borrow area.

For small values of the Boltzman variable ($u \leq 0.01$), the Theis well function W(u) can be approximated by the first two terms in an infinite series. This allows the drawdown to be expressed as a simple closed form function of u. This is the basis and restriction for using the Jacob method of analysis shown in Figures A-4 through A-6. In the Jacob Analysis, the drawdown versus time data is plotted on semi-log paper and the slope of the relationship is determined at large time (i.e., $u < 0.01$). The slope of the line is related to transmissivity as shown by the equations for T in Figures A-4 through A-6. The values s in those calculations is equal to the drawdown over one log cycle of time. Both the pumping drawdown and recovery drawdown data were analysed using the Jacob method.

The results of the pumping and packer test data analysis are presented in Section III.



$T = \frac{Q\,W(u)}{4\pi S}$, Q = 1728 gpd

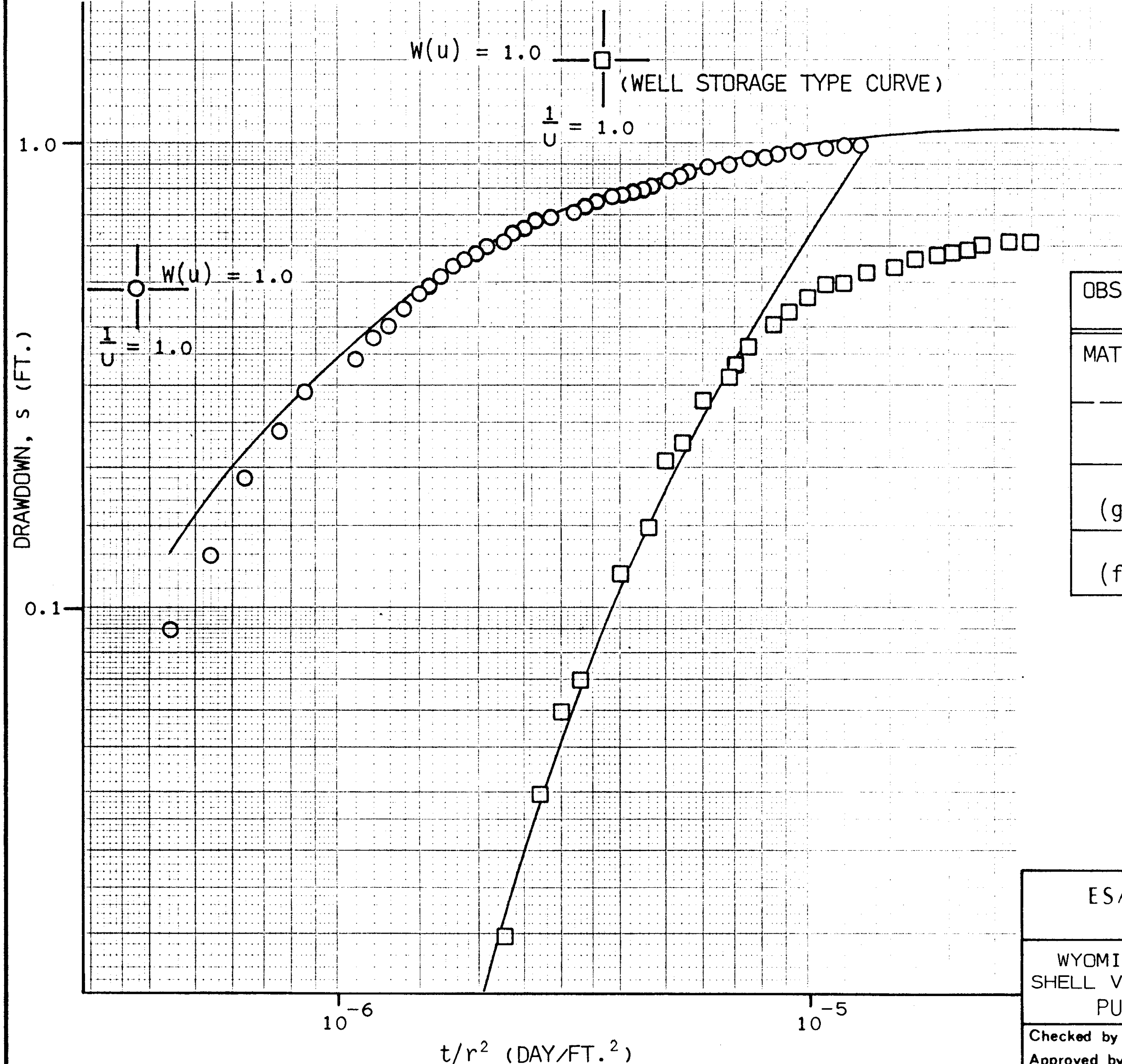
OBSERVATION WELL	DH-1	DH-8
MATCH POINT W(u)	10	1
s (ft.)	1.6	0.2
T (gpd/ft.)	860	688
T (ft.^{2}/yr.)	4.20×10^4	3.36×10^4

NOTE: PUMPED WELL DH-7
OBSERVATION WELLS
O DH-1
□ DH-8

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED – LEVEL III STUDY
PUMPING TEST 1 – THEIS ANALYSIS

Checked by	Date	Project No.	Figure No.
J. J. Hemule (Checked) / R. P. Volpe (Approved)	10-22-85 / 10/22/85	3235	A-1



$T = \frac{Q\,W(u)}{4\pi S}$, Q = 4320 gpd

OBSERVATION WELL	DH-1	DH-8
MATCH POINT W(u)	1.0	1.0
s (ft.)	0.49	1.5
T (gpd/ft.)	702	229
T ($ft.^2$/yr.)	3.43×10^4	1.12×10^4

NOTE: PUMPED WELL DH-7
OBSERVATION WELLS
○ DH-1
□ DH-8

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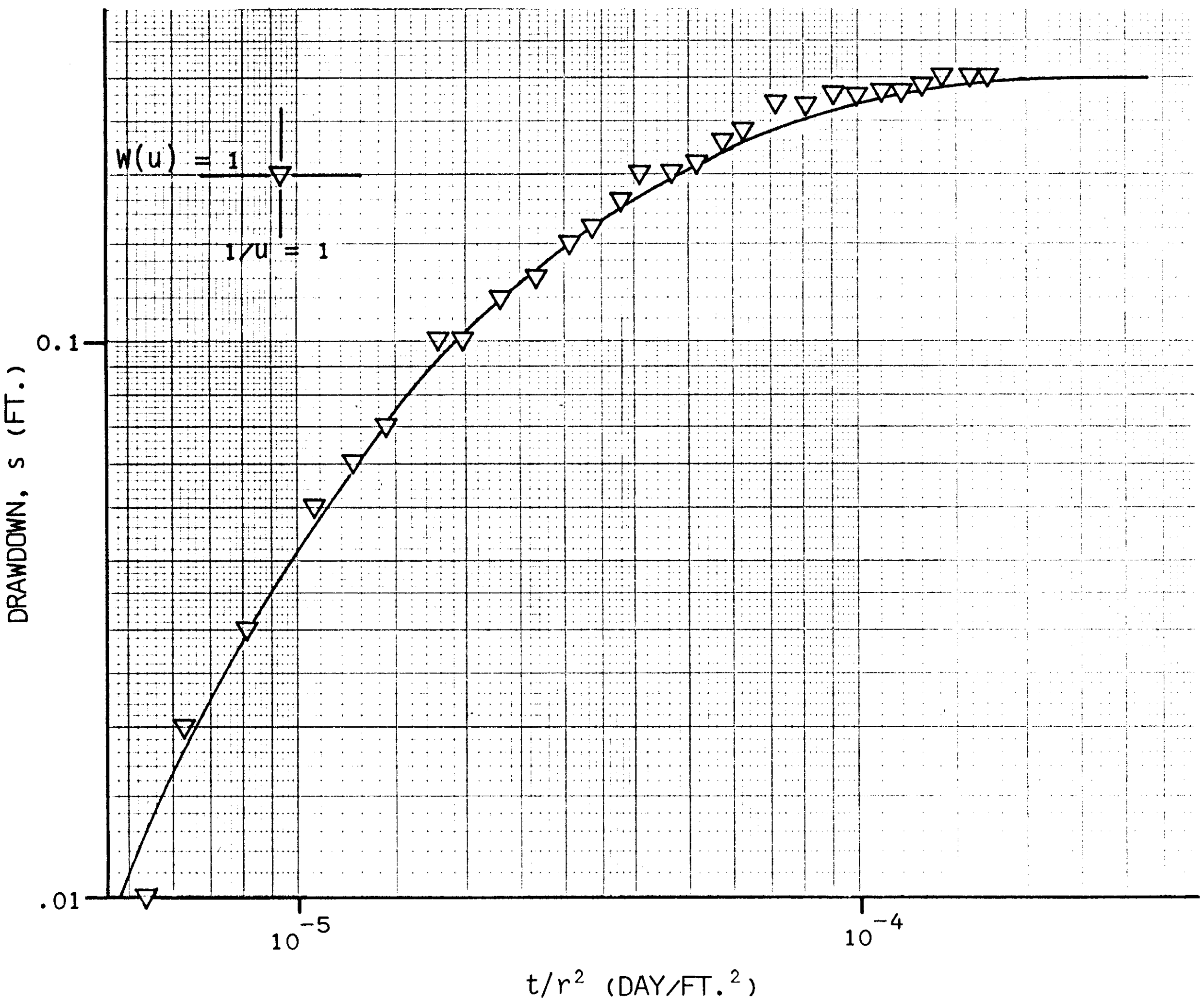
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
PUMPING TEST 2 - THEIS ANALYSIS

Checked by D.J. Hemke	Date 10-22-85	Project No. 3235	Figure No. A-2
Approved by R.R. Volpe	Date 10/22/85		



$$T = \frac{Q\,W(u)}{4\pi S}, \quad Q = 1728 \text{ gpd}$$

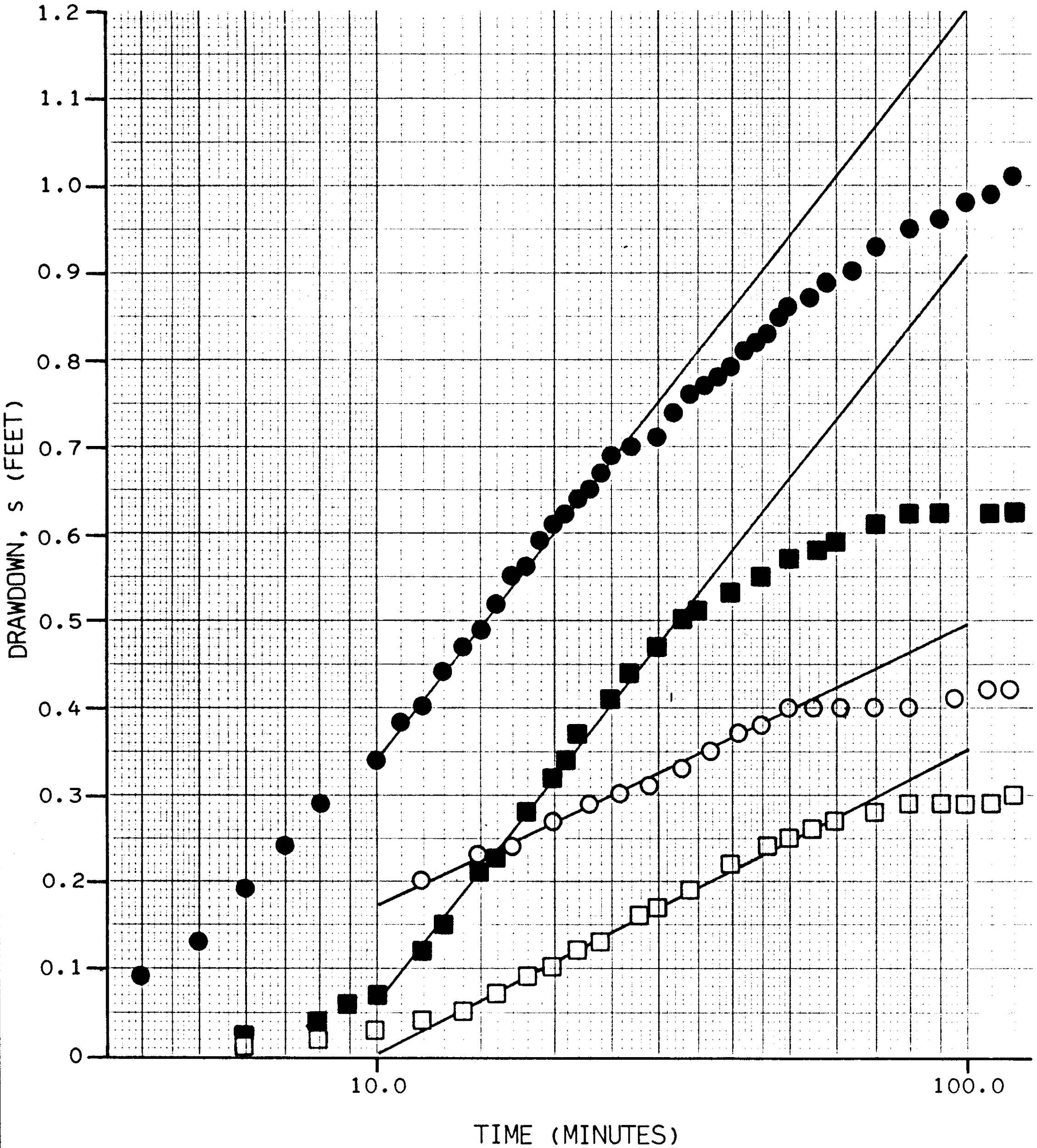
MATCH POINT	
W (u)	1
S (ft.)	0.2
T (gpd/ft.)	1719
T (ft.²/yr.)	8.39×10^{4}

NOTE: PUMPED WELL DH-11
OBSERVATION WELL DH-12

ESA Geotechnical Consultants
Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
PUMPING TEST 3 - THEIS ANALYSIS

Checked by D. J. Hemler	Date 10-22-85	Project No. 3235	Figure No. A-3
Approved by R. G. Volpe	Date 10/22/85		



$$T = \frac{2.303\ Q}{4\pi\Delta s}$$

TEST NO.	1		2	
Q (gpd)	1728		4320	
OBSERVATION WELL	DH-1	DH-8	DH-1	DH-8
Δs (ft.)	0.32	0.36	0.86	0.86
T (gpd/ft.)	990	880	920	920
T (ft.^{2}/yr.)	4.83×10^4	4.29×10^4	4.49×10^4	4.49×10^4

NOTE:
PUMPED WELL NO. DH-7
OBSERVATIN WELLS
○ DH-1 TEST 1
● DH-1 TEST 2
□ DH-8 TEST 1
■ DH-8 TEST 2

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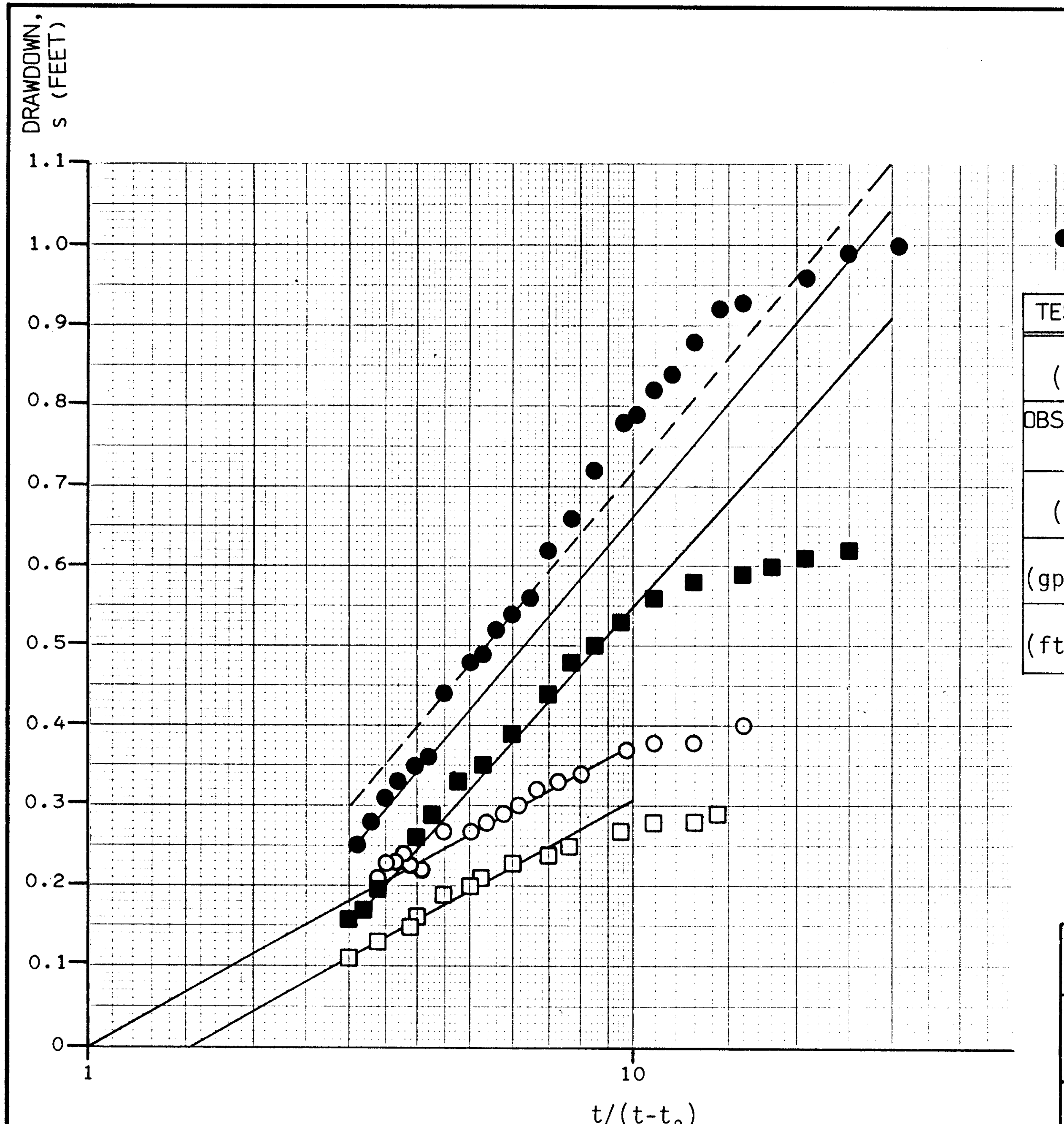
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
PUMPING TESTS 1 AND 2 - JACOB ANALYSIS

Checked by D. J. Hamby	Date 10-22-85	Project No. 3235	Figure No A-4
Approved by R. R. Volpe	Date 10/22/85		



$$T = \frac{2.303\ Q}{4\pi\Delta s}$$

TEST NO.	1		2	
Q (gpd)	1728		4320	
OBSERVATION WELL	DH-1	DH-8	DH-1	DH-8
Δs (ft.)	0.37	0.38	0.80	0.75
T (gpd/ft.)	856	834	990	1056
T (ft.^{2}/yr.)	4.18×10^4	4.06×10^4	4.83×10^4	5.15×10^4

NOTE: PUMPED WELL DH-7
OBSERVATION WELLS
○ DH-1 TEST 1
● DH-1 TEST 2
□ DH-8 TEST 1
■ DH-8 TEST 2

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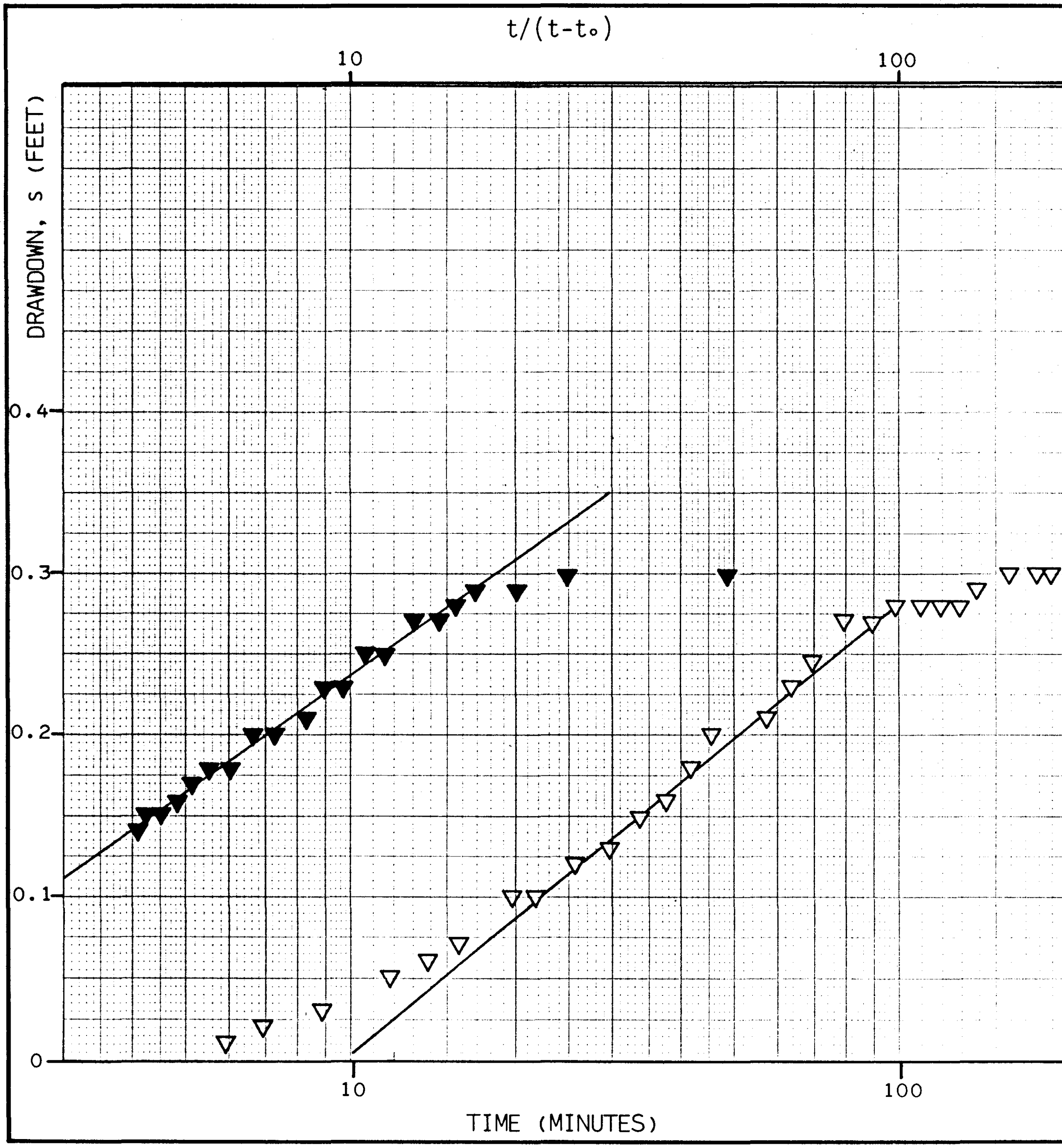
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
RECOVERY TESTS 1 AND 2 - JACOB ANALYSIS

Checked by D. J. Hamby Date	Project No.	Figure No.
Approved by R. K. Volpe Date 10/22/85	3235	A-5



$$T = \frac{2.303\,Q}{4\pi \Delta S}$$

TEST	PUMP	RECOVERY
Q (gpd)	4320	4320
ΔS (ft.)	0.27	0.24
T (gpd/ft.)	2932	3299
T (ft.^{2}/yr.)	1.43×10^5	1.61×10^5

TEST NO. 3
NOTE: PUMPED WELL NO. 11
OBSERVATION WELL NO. 12

▽ PUMPING TEST DATA
▼ RECOVERY TEST DATA

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Fort Collins, Colorado

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
PUMPING/RECOVERY TEST 3 - JACOB ANALYSIS

Checked by D.J. Hamle Date 10-22-85 / Approved by R.R. Volpe Date 10/23/85	Project No.	Figure No.
	3235	A-6

ESA Geotechnical Consultants

Soil and Bedrock Log

PROJECT 3235 SHELL VALLEY DATE DRILLED 8/1-6/85 HOLE NO. DH-7
LOCATION 45.8' N53W of DH-8 GROUND ELEV. 9224 (topo)
DRILLING CONTRACTOR ERICKSON FORD LOGGED BY SWB DEPTH TO GROUND WATER 4.0' 8/9/85
TYPE OF RIG LY-34 HOLE DIAMETER 5 5/8 HAMMER WEIGHT AND FALL 140 lbs 30 in
SURFACE CONDITIONS On slight hill on graded road TOTAL DEPTH 38.2 NO. CORE BOXES

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	ML SM	ALLUVIUM 0.0-15.0 BOULDERY SILTY SAND; Mod yel. brown (10YR5/4); ~20% non plastic fines, ~65% fine to coarse grained sand; quartz, mica, and feldspar evident; 15% gravels cobbles and boulders of granite.			RD			Moved rig to site 8/1 got bentonite 8/2, set rig up 8/3, began drilling 8/4
2.0								10:49 Mod chatter 2.0-2.5. Using 5 5/8" tricone bit with 5' stabalizing colar.
4.0		0.0-0.5 Topsoil, organic sandy silt; Grayish br. (5YR 3/2)						10:57 10:58 Mod chatter 2.6-5.0 Rig (4,000 lbs) is off ground, using 400 lbs hydraulic pressure. Drills very slow from 2.6-5.0
6.0		2.0-2.5 Cobble 2.6-5.0 Gray granite boulder 6.0-7.0 Gravelly silty sand 7.0-9.0 Gray granite boulder	S-1		C	1	1.6/2.0 = 80%	11:44 12:50 5.0-6.0 Drilled fast 2:18 6.0 Mod chatter
8.0								0-6.0 Used 25 lbs bentonite (water does not reculate). Had to replace swivel seal 2:21 8/5/85
10.0		10.0-13.0 Cobbly silty sand			RD			7.0 Very slow advance Tried core bit to make starter hole for big bit. Recovered 1.6' Hole measures 8.8
12.0		13.0-14.9 Granite boulder						10:12 10.0-13.0 Rig bouncing on rocks intermittantly w/ ~2" soft zones between Driller says attempts to sample this are futile Drilling takes ~45 min/0.4'
14.0		15.0-24.5 BOULDERY SAND TO BOULDERY SILTY SAND; as above except fines range from 0-20%, sand content increases.	S-2					3:04 15.0-16.8 attempt 4:34 Big bit 2" diameter ring sample Hole measures 13.0'
16.0	SP- SM	15.5-16.1 Silty Sand with gravel, ~20% nonplastic fines, ~70% fine to coarse grained sand, 10% gravel, iron oxide stained. 17.0-19.0 granite boulder 18.0-21.0 Decomposed granite Boulder	S-3 S-4					4:50 8/6/85 Sampler drops to 14.0, Pushed it to 14.8. Retrieved (S-2) cuttings. Tried hammering sample (30-40 blows/6") retrieved 1.6' ring, sample top 1.0 may be cuttings (S-3), S-4 9:08
18.0								
20.0								

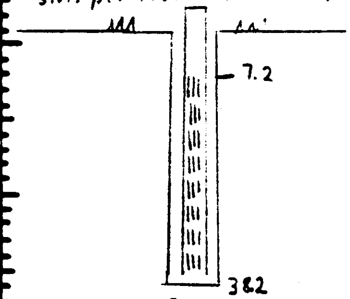
SHEET 1 OF 2

PROJECT 3235 Shell Valley **DATE DRILLED** 8/1-6/85 **HOLE NO.** DH-7

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %
20.0	SP-SM	15.0-24.5 Bouldery Sand to Bouldery Silty Sand (cont'd) 21.0-22.6 Granite Boulder			RD		
22.0							
24.0	SP-SM	24.5-37.5 Gravelly Sand to Gravelly Silty Sand; Moderate Yellowish Br. (10YR 5/4) 5-10% non plastic fines; aprox. 80% fine to coarse grained sand with the coarse grains dominating; aprox. 10-15% gravel, occasional cobbles	S-5 S-6				
26.0							
		27.2-27.5 Cobble					
28.0							
		29.5-29.8 Cobble(?)					
30.0							
32.0							
34.0							
36.0							
	Br	Bedrock 37.5-38.2 Granite					
38.0		38.1 B.H.					
40.0							
42.0							
44.0							

REMARKS

9:16 is definately in
9:46 place material, left it in rings for lab inspection
17.0-18.0 Drills hard
10:17 and slow - rig lifted off
10:44 ground.
10:45 18.0-19.6 Drills easier and faster. (Dg?)
10:51 19.6 rig bouncing slightly
11:00 Mixed Mud (30 lbs bentonite
12:01 1 qt polymer).
21.0 Rig bouncing and
12:07 raising up. Drilling very slowly to 22.6.
7 22.6-24.5 Drills quickly
12:15 return water is brown
6 24.5-25.1 Attempt a
12:24 sample. Hole measures 21.0 (slough). Sampler
add rod goes to 23.5, push it to
12:36 24.0. Hammer it to
4 25.1 (16,57 blows/6").
12:40 Recover 0.6' sample (S-6)
8 with 1.0' slough? (S-5)
12:48 24.5-26.0 Drills smooth
5 and faster than above
12:53 ave 4 mn/ft
27.2-27.4 Mod chatter
Lunch 29.5-29.8 Rig bouncing
1:57 29.5 Rig off ground
30.3 Bushy mod. chatter (squeeky)
2:01 to 31.0. Driller says its the
chuck head. 31.0 Rig on ground
2:10 34.5 Driller says attempts to sample would be futile
2:15 27.0-37.0 ave speed 5-8 mn/ft.
2:22 37.5 Mod chatter, rig off ground, slow drilling
2:35
2:49 38.2 Terminated hole
Placed 4" (I.D.) steel well casing with 16, 3" long 1/8" wide slots per foot as illustrated



SHEET 2 OF 2

ESA Geotechnical Consultants

Soil and Bedrock Log

PROJECT 3235 SHELL VALLEY **DATE DRILLED** 7/22-27/85 **HOLE NO.** DH-8
LOCATION 78' N6E from toe of dam, 127' S60E of DH-1 (on ℄ of prop. dam) **GROUND ELEV.** 9226 (Topo)
DRILLING CONTRACTOR ERICKSON-FORD **LOGGED BY** SWB **DEPTH TO GROUND WATER** 1.0 8/9/85
TYPE OF RIG LY-34 **HOLE DIAMETER** NX **HAMMER WEIGHT AND FALL** 140 lbs / 30 IN
SURFACE CONDITIONS Flat, rocky, slightly graded pad **TOTAL DEPTH** 52.0 **NO. CORE BOXES** 2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	ML SM	ALLUVIUM 0.0-7.0 BOULDERY SILTY SAND, Mod. yel. br. (10YR 5/4), ~20% non plastic fines, ~65% pred. fine grained sand; quartz and mica evident; 15% gravels, cobbles and boulders of granite			RD			7/22/84 Using 2 5/16" new tricone bit. Light chatter. 1.5' v. slow advance, rig (4,000 lbs) slightly off ground
2.0								2.8 drill rate picks up - drills quiet 4.5-4.7 drills smooth and fast
4.0	SM↓ SM	0.0-0.5 Topsoil-org. sandy silt 1.5-2.8 Gray Granite boulder ~1.0 wet 4.5-4.7 Silty Sand lens, no gravels or cobbles.						10:11 10:19
6.0	SP SM	6.7-7.0 Sand, multi colored, coarse grained, subrounded, clean, horizontal contact to: ½" sandy silt lens then:	S-1	2 17 39				10:55 6.1-7.6 SPT 11:05 56 blows / ft first 6" slough, blow count misleading, shoe slightly damaged
8.0		7.0-17.0 GRAVELLY SILTY SAND; Multi colored to Mod Yel Br (10YR 5/4), fines are variable from 10 to 30%; 60-80% fine to coarse grained sand, varies from well to poorly graded, ~20% pred. gravels with some cobbles and boulders, gravels are subrounded.						11:15 7.5 rig "bounces" 11:19 for ~6" 9.5 Mod chatter, rig bouncing 10.5 Drilling smoother
10.0								11:24 11:26
12.0		7.5 Cobble 13.9 Decomposed granite cobble						Some water is coming up outside of the hole ~6" away like a small fountain. Added ~1¼ cups of Insta Pac 425 liquid polymer to drill water.
14.0		14.6 cobble 15.0 fines increase 15.9-16.2 Clean Sandy Gravel						14.6 Mod brief chatter 15.0 Return water is carrying fines - Yel. brown colored. 11:33
16.0	GP SM		S-2	43				16.0' Set steel casing size NW (3") 15.9-16.5 SPT 43/5.5 Terminated test. 16.5 Return water clear
18.0	SP- SM	17.0-40.0 BOULDERY SAND TO BOULDERY SILTY SAND; Mod. yel. br (10YR 6/4); fines vary from 0 to 15%, 75% fine to coarse grained sand, up to 25% grav., cobbles, boulders. 17-18.0 Decomposed granite boulder						5:02 5:04
20.0			S-3		C	1		

SHEET 1 **OF** 3

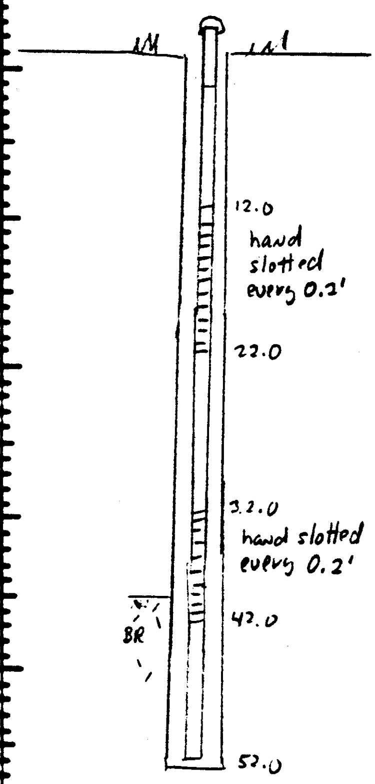
PROJECT 3235 SHELL VALLEY DATE DRILLED 7/22-27/85 HOLE NO. DH-8

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	SP-SM	17.0-40.0 BOULDERY SAND TO BOULD. SILTY SAND (CONT'D) 18.0-20.8 Very coarse grained pink granite boulder very hard, fresh	S-3		C	1	2.5/3.0 = 83%	5:11 5:13 Tricone is taking 30min per ft. so driller decided to try to core at 19.5. 20.8-21.3 cored very quickly.
22.0		21.3-22.5 Boulder 22.7-23.0 Cobble	S-4		RD			5:18 10:03 7/24/85 NOTE: sample is w bag, numbered 1-4 top to bottom. Driller reports soft easy drilling 22.5-22.7 then mat'l is hard again. Ad-
24.0	SM	23.0-26.5 Silty Sand, ~10-20% non plastic fines, ~80% sand, 10% grav. No cobbles or boulders						vanced casing to 19.0, cleaned hole w/ tricone, advanced
26.0	SP-SM		S-5		C	2	3.5/3.5 = 100%	hole w/ tricone, mod
28.0		26.9-30.4 grey granite boulder	S-6					12:05 chatter. 23.0 smooth quiet drilling. At 1:33 25' attempted SPT recovered only slough from 19-23? (S-4)
30.0		30.4-31.2 gravelly silty sand				3	5.0/5.0 = 100%	1:38 1:39 28.5-29.0 Drilled quiet & smooth, then mod. chatter.
32.0		31.2-35.6 Grey granite boulder						31.0 heavy chatter fast drilling ↑bent rod 1:43 1:44 28.0-32.5 ave 2¼ min/ft 1:48 (Bit plugged) 3:05
34.0			Box 1					33.5-37.5 Bit pluggs occasionally 3:13 3:18
36.0		35.6-38.0 gravelly silty sand				4	2.8/5.0 = 56%	ave 2½ min/ft
38.0								3:23 3:24 3:26 (2 day delay while set casing shoe) 4:08 7/27/85 - pulled casing, put on new shoe and pushed casing to 38.0
40.0	Br (granite)	BEDROCK 40.0-52.0 GRANITE, Multi colored gray, from Medium light gray (N6) to grayish black (N2)				5	4.0/5.0 = 80%	4:12 4:13 39.6-40.0 drilled easier. (washed?)
42.0		med to coarse grained with orthoclase, quartz, and biotite evident, little fractured to						4:20 4:22 Varies from 3 to 5 min/ft. 4:27 4:47
44.0		massive, hard, fresh, very strong				6	4.0/4.0	

SHEET 2 OF 3

PROJECT 3235 Shell Valley DATE DRILLED 7/22-27/85 HOLE NO. DH-8

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0	Br (granite) 12°	40.0-52.0 GRANITE (CONT'D) 44.7 fracture w/some gold colored mineralization (pyrite?) dips 12°	Box 1		C	6	4.0/4.0 = 100%	4:57 4:59 Note: core is broken but it looks like the drilling caused it.
46.0								5:16 Hole measures 47.0 8:50 42.0-47.0 Ran packer test at 10, 20, 30, 40 psi - took 0 gallons.
48.0	30° 0° 20°	49.4-49.8 Fine grained dike dipping 30° 49.7-50.4 Granite is fractured (core is broken), heavy gold (pyrite?) mineralization along fractures dipping 0-20°	Box 2			7	4.8/5.0 = 96%	8:55 8:56 ave 2½ min/ft 9:01 902 50.0-51.4 drills faster.
50.0								
52.0		BH - 52.0						906 52.0 Terminated hole 47.0-52.0 Ran packer test at 20, 30, 40, 50 psi. took 0 to .02 gpm. Installed 2" scedule 40 pvc piezometer as illustrated.
54.0								
56.0								
58.0								
60.0								
62.0								
64.0								
66.0								
68.0								



SHEET 3 OF 3

ESA Geotechnical Consultants

Soil and Bedrock Log

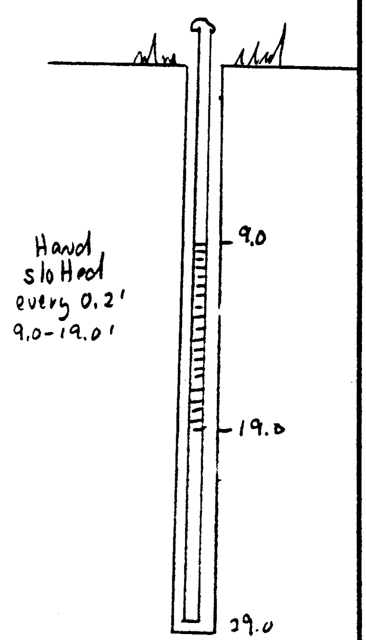
PROJECT 3235 SHELL VALLEY **DATE DRILLED** 8/1/85 **HOLE NO.** DH-9
LOCATION Base of right abutment (bottom of rock slope) 8.5' N of ℄ of Prop. Dam **GROUND ELEV.** 9232.5 (top)
DRILLING CONTRACTOR ERICKSON FORD **LOGGED BY** SWB **DEPTH TO GROUND WATER** 3.2 8/9/85
TYPE OF RIG LY-34 **HOLE DIAMETER** NX **HAMMER WEIGHT AND FALL** 140 lbs 30 in
SURFACE CONDITIONS Graded pad **TOTAL DEPTH** 29.0 **NO. CORE BOXES** 2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT Lift Core	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	SM	ALLUVIUM 0.0-8.5 BOULDERY SILTY SAND; Mod. yel. br (10YR 5/4)(?); ~20% non plastic fines, ~65% sand; 15% cobbles and boulders of granite. Boulders exhibit iron oxide stained fractures. 0.0-0.5 TOPSOIL, Sandy silt Mod brown (5YR 3/4); 80% non plastic fines, 20% fine grained sand; num roots and organics present.	S-1 S-2 S-3		C	1	5.0/5.0 = 100%	8:00 On site. Drilled 3" steel casing down. Used same casing bit as DH-10, then went in with NX diamond core bit. Casing is at 6.0. Using ~125 psi hydraulic pressure 9:29 NOTE: gave 0.8' sample away from run 1 9:35 9:36 ave 4 min/ft 6.5 Drills slower, light chatter 9:44 9:45 6.8-7.0 Drills fast, return water is brown 9:53 ~9.0 Bit plugged Hole 10:10 measures 9.0
2.0								
4.0								
6.0								
8.0	BR	BEDROCK 8.5-29.0 GRANITE: Multi colored pink, gray, and black euhedral quartz, feldspar, and biotite present med to coarse grained; Mod fractured with 0.2-0.5' spacing of fractures dipping 0-75°, many are heavily iron oxide stained, little weathered to fresh, very hard, mod. strong						
10.0	60					2	4.8/4.8 = 100	Drills smooth with light chatter ave 5 min/ft 10:20 10:21 11.5-13.0 Return water is greyish green, some "silt balls" present in cuttings. Drill rate increases to 2½ min/ft 10:26 10:27
12.0		11.5 Irregular contact to dark gray biotite granite; closely fractured with 0.2' fracture spacing with dips 0-90°; fine grained.						
14.0	90, Qtz, 35°	13.6 white quartz vein with grey granite from 3.7 on dipping 35°. Fractures are 0.5-1.0' apart	Box 1					10:30 13.0 Return water clears up. 13.8 Bit plugged 10:47 Hole measures 13.8
16.0	30, 55					3	5.2/5.2 = 100'	10:52 10:57 ~16.0 Return water is white, translucent Drills smooth with light chatter 2-2½ min/ft 11:01 11:02
18.0		18.5 Irregular 30° dipping contact to dk gray granodiorite(?) 19.5 Qtz vein dipping 75°						18.6 return water is greenish gray. Hole measures 19.0 11:08 11:54
20.0	75					4	4.8/4.8	

SHEET 1 OF 2

PROJECT 3235 SHELL VALLEY DATE DRILLED 8/1/85 HOLE NO. DH-9

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (Core)	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	Br	8.5-29.0 GRANITE (CONT'D) 20.0 Fractures in granodiorite(?) dip 30° to 50° every 0.2 to 0.5' Iron oxide staining is much lighter than above, mat'l is fine grained.	Box 1		C	4	4.8/4.8 = 100%	Drilled casing down 2.0' to 8.0'. Ran packer test at 10 and 20 psi (11.0-19.0') took max 2.4 gpm. Drills smooth with light chatter ave 2½ min/ft 2:00 2:01 2:06 2:07 2:10 2:30 Hole measures 23.8
22.0	30, 50, 55							
24.0	40	25.8-26.8 light grey granite				5	5.2/5.2 = 100%	Drills smooth and quiet ave 2 min/ft 2:34 2:35
26.0		28.1 light grey granite 28.5-28.7 Weathered(?) dk gray sandy zone, friable.	Box 2					28.5-28.7 Driller does not think mat'l is mechanically broken. 2:40 2:41
28.0	30							Terminated hole 29.0. Attempted packer test, had trouble seating packer, it leaked slightly. Tested 19-29' at 20 and 30 psi max flow 1.5 gpm. Installed piezometer as illustrated
30.0		BH 29.0						
32.0								
34.0								



Used schedule 40 PVC, 2" diameter

SHEET 2 OF 2

ESA Geotechnical Consultants

Soil and Bedrock Log

PROJECT 3235 SHELL VALLEY DATE DRILLED 7/28-30/85 HOLE NO. DH-10
LOCATION RIGHT ABUTMENT 188' N25W of end of dam (east side of spill.) GROUND ELEV. 9265.3 (T[illegible] hand level)
DRILLING CONTRACTOR ERICKSON FORD LOGGED BY SWB DEPTH TO GROUND WATER 16.5 8/9/85
TYPE OF RIG LY-34 HOLE DIAMETER NX HAMMER WEIGHT AND FALL NA
SURFACE CONDITIONS GRADED PAD ON ROCKS TOTAL DEPTH 48.8 NO. CORE BOXES

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0–3.0	SM	FILL 0.0-3.0 SILTY SAND, with boulders; Mod yel brown (10YR 5/4); ~15% non plastic fines; ~70% fine to coarse grained sand, ~15% cobbles and boulders.		core	C			Moved to site 12:30 Drilled 3" steel casing down using diamond impregnated casing shoe. Re-drilled using core bit from DH-8
3.0–8.0	B+C	ROCK SLOPE 3.0-21.6 BOULDERS AND COBBLES Multi colored light to dark gray or pink; on natural slopes the material is clast supported with little or no soil development. (here some fill has filtered down into the voids). Rocks are angular to sub angular, med - coarse grained granite; size varies from 3" to 14' 3.0-13.5 granite boulder	Box 1			1	5.0/5.0 = 100%	3.0 hit hard drilling 3.5 lost circulation 3:47 4.0 circulation 4:06 daylighting on 4:09 fill slope. 4:10 Run 1 left 0.8' core sticking up in hole 4:31
8.0–12.0						2	4.0/4.0 = 100%	Drills smooth w/ light to mod 4:38 chatter ave 3½ 4:44 min/ft. Drill return water is clear No zones of lost 4:57 circulation, no
12.0–17.0		17.0-20.7 granite boulder				3	1.8/5.0 = 36%	5:19 zones of rapid 5:27 drill advance. Run 2 got 4.9' 5:38 cont. core 13.0 5:39 bit plugged 13.5-14.0 Rapid advance 5:41 13.5-16.0 Return 5:42 water is brown drills 5:45 fast 1min/ft 7/29/85
17.0–20.0						4	4.4/4.6 = 96%	10:35 16.5 Drill rods [illegible] Drilled casing down to 17.0.

SHEET 1 OF 3

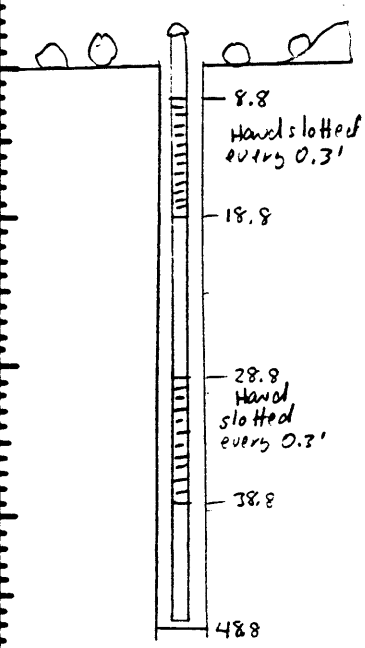
PROJECT 3235 SHELL VALLEY DATE DRILLED 7/28-30/85 HOLE NO. DH-10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	B+C	3.0-21.6 BOULDERS AND COBBLES (cont'd) ~20.7 Contact between boulders			C	4	4.4/4.6 = 96%	10:51 Hole measures 21.0 11:02
22.0	Br	BEDROCK 21.6-48.8 GRANITE; pink and gray, med to coarse grained granite; little weathered to fresh, v. hard	Box 1					11:21 Drills smooth with little to mod. chatter
24.0						5	5.0/5.0 = 100%	11:28 11:29 3½-4 min/ft
26.0	Qtz	25.4-25.6 Quartz vein, subhoriz. 25.6 Sharp clean contact to dark gray, fine to med. grained biotite granite (granodiorite(?))						11:38 11:39
28.0			Box 2					1:14 Drills smooth with little to mod chatter ave 5 min/ft
30.0						6	5.0/5.0 = 100%	1:24 1:25 recovered 4.3' of continuous unbroken core from run 6 1:33 1:42 Hole measures
32.0		32.5 fracture?						2:08 30.5 ~32.5 drills easier for ~0.2'
34.0						7	4.4/4.4 = 100%	2:19 2:20 recovered 5.1' from run 7
36.0	25 20 30 75	35.0 Rose quartz filled fracture, tight ~ 0.05', dipping 25° 35.8 and 36.0 heavy iron oxide stained fractures dipping 20° and 30° respectively in opposite directions						2:33 2:34-2:39 Hole measures 8:56 7/30/85 35.7. Ran packer test from 23-36 at 10, 20, 30 psi - took max 0.6 gpm
38.0	75	~36.0 Feldspars grade pink mat'l is dark grey and pink 37.0 Green (apetite) filled fracture, tight, dipping 75°				8	5.0/5.0 = 100%	9:07 9:08 Drills smooth with little to mod chatter ave 5 min/ft
40.0	10 20 20	39.0 Several fractures dipping 75° 39.3-39.8 Coarse grained dike dipping 20° 40.0 Dipping contact (20°) to pink and gray, med to coarse grained granite						9:18 recovered 4.7 from run 8 but got core in next 2 runs 9:25 Hole measures 40.4 9:44 Return circulation difficult to establish 41.0-41.3
42.0			Box 3			9	5.0/5.0 = 100%	
44.0								

SHEET 2 OF 3

PROJECT 3235 SHELL VALLEY DATE DRILLED 7/28-30/85 HOLE NO. DH-10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
44.0	Br	21.6-48.8 GRANITE (cont'd) 45.4-45.5 Subhorizontal "broken" zone, some silt present. 46.0-46.3 Rose quartz dike dipping 50°. Very hard. Iron oxide (?) stained on one surface	Box 3		C	9	5.0/5.0 = 100	10:09 10:10 10:16 Drilled smooth with light chatter ave 6 min/ft Recovered 5.2 from run 9
46.0	QRTZ							
48.0						10	2.8/2.8 = 100%	10:39 10:55 10:56 11:18 Drills smooth with mod. chatter ave 8 min/ft, mat'l is harder to drill than above. Took 22 min for 48.0-48.8. Cathead rope frayed as pulled rods out. Hole measures 48.5 Recovered 2.9 from run 10. Terminated hole 48.8. Packer tested 35.8-48.8 at 30,40,50 psi took max of 0.09 gpm. Installed 2" scedule 4 pvc as illustrated
		BH 48.8						
50.0								
52.0								



SHEET 3 OF 3

ESA Geotechnical Consultants

Soil and Bedrock Log

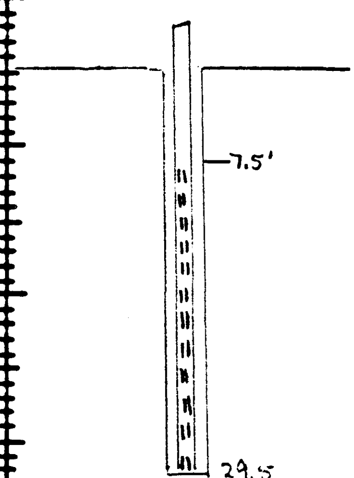
PROJECT 3235 SHELL VALLEY DATE DRILLED 8/8/85 HOLE NO. DH-11
LOCATION 278' NW of DH-12; 87.3' NW from intersection of Lake Adelaide and Shell Reservoir Roads GROUND ELEV. 9220 (topo)
DRILLING CONTRACTOR Erickson Ford LOGGED BY CJP DEPTH TO GROUND WATER ___
TYPE OF RIG LY-34 HOLE DIAMETER 5 5/8 HAMMER WEIGHT AND FALL 140 lb 30 in
SURFACE CONDITIONS Flat and grassy TOTAL DEPTH 29.5 NO. CORE BOXES 0

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	ML / SM	ALLUVIUM 0.0-21.5 SILTY SAND; Yellowish brown (10YR 5/4); 10-40% non plastic fines, 60-90% predominantly fine and med grained sand; qtz mica, and feldspar evident, occasional cobble or boulder of granite, Mat'l appears to be lenses of sand and silty sand interbedded. 0.0-0.1 Topsoil, Sandy Silt Mod. Br (5YR 4/4), ~90% non plastic fines, ~10% fine grained sand, roots present 0.0-2.0 Pink granite boulder, sub rounded, slightly weathered 5.0-5.5 Silty Sand, ~30% non plastic fines, 68% fine and coarse, skip graded sand, subrounded to subangular; ~2% gravel. 9.5-15.0 Mat'l is denser with occasional cobbles			RD			8:30 Drillers on site (got fuel and mud) Reggie Brown drilling. 9:00 start drilling w 5 5/8" tricone w/ 5' stabilizing collar (9:00) Used backhoe to excavate 2.0 start the hole. Removed 2.0' boulder Drills smooth and quiet
2.0								
4.0								
6.0			S-1 S-2	35				5.0 Attempt to hammer a California Ring sampler (2" diameter) Recovered 0.5 sample (S2) 0.2 slough (S-1)
8.0								Drills smooth and quiet to 9.5
10.0								9.5-15.0 Light chatter, rig bouncing off ground. drill rate slows
12.0								12:18 12.0 attempted to push a shelby tube - no recovery.
14.0				63				11:23 add rod mix mud 11:46 13.0-15.0 Very slow drilling 10-20 min/ft 12:11 1:44 15.0 attempt to hammer California Ring Sampler Hole measures 14.5. No recovery 1:53 1:54 15.0 Rig is not off ground
16.0								
18.0								18.0-18.8 Drills faster 18.8- Rig briefly off ground (2:11)
20.0								

SHEET 1 OF 2

PROJECT 3235 SHELL VALLEY DATE DRILLED 8/8/85 HOLE NO. DH-11

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0		0.0-21.5 SILTY SAND (CONT'D)			RD			20.0 Attempted to sample had difficulty getting the 5 5/8" bit out of the hole, installed NX casing to 20.0, tried to core at 21.0.
		WEATHERED BEDROCK			C			3:35
22.0	Dg / Weath Br / 20 / 10 / 70	21.5-25.5 GRANITE; Multi colored, starts with 6" of decomposed granite then grades into competent granite with highly weathered fractures every 0.1'. Fractures are clay, silt, and sand filled gradational contact to:	S-3			1	2.5/3.5 = 71.4%	3:36 3:37 21.0-22.0 Drills fast ~1 min/ft. Return water is brown and sandy 3:46
24.0								4:15 22.0-23.7 Heavy Chatter, drills slower 1½ min/ft return water is milky
26.0	Br granite / 20 / 65	BEDROCK 25.5-29.5 GRANITE, light gray to pink; med. grained, fractures every 0.2'. Some clay and silt coats the fractures and many fractures are iron oxide stained. Rock is weak.	S-4			2	5.0/5.0 = 100%	4:18 4:19 25.4 return water is pink to 26.5
28.0			S-5					26.5-28.0 return water varies white to tan, 28.0-28.5 pink again. Drills smooth w/ light to mod chatter 1½ to 2 min/ft. 28.5-29.5 3 min/ft 4:23 4:24 4:27
30.0		BH 29.5						Terminated hole at 29.5'. Reamed hole 8-12:00 w/ 5 5/8" bit 8/8/85. Installed 4" slotted steel casing as illustrated. There are 16 slots per ft. They are 3" long ⅛" wide. Washed hole with lake water for 2 hrs. Used a total of 4, 50lb bags of bentonite on this hole
32.0								
34.0								
36.0								
38.0								
40.0								
42.0								
44.0								



SHEET 2 OF 2

ESA Geotechnical Consultants

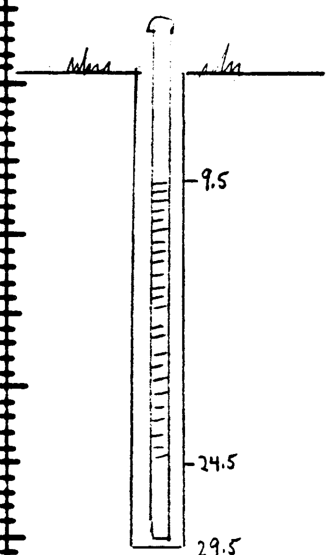
Soil and Bedrock Log

PROJECT 3235 SHELL VALLEY **DATE DRILLED** 8/7/85 **HOLE NO.** DH-12
LOCATION 59.5' NW along Lake Adelaide Road from intersection with Shell Reservoir Rd. **GROUND ELEV.** 9280 (topo)
DRILLING CONTRACTOR Erickson Ford **LOGGED BY** SWB **DEPTH TO GROUND WATER**
TYPE OF RIG LY-34 **HOLE DIAMETER** NX **HAMMER WEIGHT AND FALL** 140 lbs 30"
SURFACE CONDITIONS Flat and grassy **TOTAL DEPTH** 29.5 **NO. CORE BOXES**

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
0.0	ML SM	ALLUVIUM 0.0-21.5 SILTY SAND; yellowish brown (10 YR 5/2); 10-40% non plastic fines, 60-90% predominantly fine and med grained sand; quartz, feldspar, and mica evident, occasional cobble. Mat'l appears to be interbedded lenses of sand and silty sand			Casing			8:40 Rig on site, brought other equipment, put casing down 1:07
2.0 – 6.0								Casing goes down smooth and fast, no large rocks encountered, casing shoe is diamond impregnated, cuttings wash to outside, inside casing is clean, no need to redrill.
6.0 – 8.0		0.0-1.0 Topsoil, Sandy Silt Mod. brown (5 YR 4/4); ~90% non plastic fines, ~10% fine grained sand; num. rootlets.						
10.0				20 14 11				9.6-11.1 Took SPT, no sample recovered 25 blows/ft (possibly pushing a rock)
11.1 – 12.0	SP SM	11.1-11.2 v. fine grained sand 11.2-11.7 Gravelly Sand, ≈5% fines, 80% med to coarse grained sand; 20% gravel; subangular lithics of granite	S-1	6 7 6				tried 11.1-12.6 with a new shoe 13 blows/ft recovered 1.0/1.5 sample
12.0 – 14.0	SM	11.7-12.3 Silty sand, ~40% non plastic fines, ~60% fine grained sand.						2:09 2:22 Drills smooth and quiet ave 4 min/ft only fine grained cuttings return, rest are outside casing
15.0		15.0-15.5 gravelly zone						15.0 light chatter to 15.5
16.0		16.6-17.0 Gravelly zone						2:34 2:41 16.6-17.0 light chatter
17.4		17.4-17.5 Gravelly zone						2:45 17.4-17.5 light chatter 2:46 16.0-18.0 ave :2 min/ft
18.0 – 20.0				21				2:48 Hole measures 19.7 2:52 Try SPT 19.7-20.6

PROJECT 3235 SHELL VALLEY DATE DRILLED 8/7/85 HOLE NO. DH-12

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	SPT (6")	DRILL MODE	RUN NO.	CORE REC. %	REMARKS
20.0	SM	0.0-21.5 SILTY SAND (cont'd)		40/64	Casing			Recovered gravels and broken cobbles- cuttings from above. cleaned out hole with NX size tricone
		WEATHERED BEDROCK			RD			
22.0	Dg to granite	21.5-24.5 GRANITE; multi colored; varies from in place although essentially decomposed granite to granite with highly weathered fractures approximately every 0.1'. Fractures are clay, silt, and sand filled. gradational contact to:	S-2		C	1	2.5/2.5 = 100%	3:53 22.0 attempted SPT - 20 blows with <0.1' advance. Tried to core 3:56 3:57 Cores irregularly - fast then very fast - in softer zones ave 1 min/ft. 3:58 4:23 Hole measures 24.0
24.0	Br 35	BEDROCK	S-3					24.5 added Liquid Polymer to drill water
26.0	90	24.5-29.5 GRANITE; light gray to pink, med. grained, fractures every 0.2' but fractures are tighter than above. Some clay and silt coat fractures but rock does not fall apart when handled.	S-4			2	5.0/5.0 = 100%	4:27 4:28 24.5-27.5 Drills slower and more uniform
28.0	65		S-5					4 27.5 Return water is pink 4:33 27.8 Return water is brown 4:34 24.5-29.5 goes at 2 to 2½ min/ft 4:38
30.0		BH 29.5						Pumped drilling fluid out of hole.
32.0								NOTE: Core in S-3 is fairly good quality, core in S-4 and S-5 is more broken. Driller suspects he broke S-5 when he washed the hole.
34.0								Terminated hole 29.5 Installed 2" scedule 40 PVC Piezometer as illustrated
36.0								
38.0								
40.0								



SHEET 2 OF 2

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 1 (T1) SHEET NO. 1 OF 1

PROJECT Shell Valley ℄ Left Abutment, Elev. 9300' BY DJH DATE 07-16 85

warm
dry

T-1 Location: Left abutment on ℄ of proposed raised dam
Elev: 9300 ft (measured 27' by hand level above DH-2)
Orientation: Approx. perpendicular with dam axis (N-S)
Size: 4-6 ft. deep × 2-3 ft wide × 22 ft long
Groundwater: No water encountered
Excavation notes: (a) C & M Construction, Ben Menzel operator, Harvester 3800 backhoe with 2 ft wide bucket
(b) Start time: Approx. 1:20 pm
(c) Complete time: Approx. 2:20 pm, backfilled trench
(d) Trench lengthened to excavate around boulders too large to remove
(e) refusal at 6.1 ft due to very large boulder or bedrock

DEPTH (ft)	USCS	DESCRIPTION
0.0 - 4.9	SM	Silty Sand, gravelly, with cobbles and large boulders; Moderate yellowish brown (10YR 5/4); Approx. 20-30% non-plastic fines, 30-40% med. to coarse sand, 10-15% gravel, 5-10% cobbles, many boulder size rocks; - removed 3 to 4, 3.0'-4.0' size boulders, subrounded, weathered to < 1/4" with some iron oxide zones more weathered. - composition of cobbles and boulders varies from coarse grained gray or pink granite to dark gray monzonite
4.9 - 6.1	SP (Dg?)	Coarse Sand, cobbley; Yellow Brown (10YR 5/2); Less than 5% fines; 70-80% med. to coarse, subangular to angular sand or decomposed granite; 20-30% cobbles.

note: no samples taken

R. Volpe Supervising
Log by D. Hamberg, Sampling by S. Bilodeau

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench-2 (T-2) SHEET NO. 1 OF 1
PROJECT SHELL VALLEY Borrow - Emergency Spillway Area BY DJH (SWB) DATE 7/16/85
warm
dry

T-2 Borrow Area - Emergency Spillway ~64' perpendicular (bearing N46E) from the ℄ of road to ℄ of trench; 375' N along road from intersection of first turnoff. ele 9282 (Topo)

Ben Mensel operating Harvester 3800 backhoe w/ 2' bucket width - start excavation: 3:50
finish " : 4:45
Backfilled trench, took 2 photos #'s 24, 25

LOG	DEPTH	DESCRIPTION
0 SW S-1 B-1	0.0-6.5	SW - Sand to Silty Sand; Dk yel br (10YR 4/2) 5% non plastic fines; 50-60% med to coarse grained, subrounded to subangular sand; 10-20% gravel; 5-10% cobble to boulder sized granites (6 to 12 1 to 3 ft size boulders; 2 to 3 4 to 6' boulders) dry (contact is roughly horizontal)
6.5 S-2 B-2 SW (Dg?)	6.5-10.5	SP (Dg?) - Sand (Decomposed Granite?); matrix is between Dk yel orange and Mod yel brown (10YR 5/4 - 10YR 6/6), sand clasts are multi colored (white, black and pink together on some larger grains); <5% fines, non plastic; 80-90% coarse, subangular to angular sand (granite fragments); few (<1%) cobble or boulder sized fragments; moist.
TD 10.5 (refusal)	10.5	Refusal - two boulders (one at each end of the trench) prevent the hoe from going deeper.

Backfilled trench.

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JOB NO. 3235 SUBJECT Trench 3 (T-3) SHEET NO. 1 OF 1
PROJECT Shell Valley Borrow - Emergency Spillway Area BY DJH DATE 07-17-85
cool
overcast

T-3 Location: Low point in borrow pit, south of access road, bet. DH101 & DH104
Elev: 9277 ft (Topo map)
Orientation: Approx. E-W
Size: 5 ft deep x 2 ft wide x
Groundwater: Water encountered at 3.5 ft. Flow into trench estimated at 1 to 2 gpm (R. Volpe).
Excavation notes: (a) C & M Construction - Ben Menzel operator, Harvester 3800 Backhoe with 2 ft bucket.
(b) Start excavation time: 8:40 am
(c) Backfill trench: 9:20 am
(d) Refusal at 5.0 ft due to bedrock.

DEPTH (ft)	SAMPLE NO.	USCS	DESCRIPTION
0.0 - 5.0	S-1 S-2 (from spoil)	SW-GW (Dg)	Clean Sand and Gravel, well graded; Multicolored from dark yellowish orange (10 YR 6/6) to dark yellowish brown (10YR 4/2); less than 5% fines; 40-50% fine to coarse, subangular to angular sand; 40-50% angular gravel (max. size 3/4"); 5-10% cobble size granite pieces; no large boulders. - moist to saturated bet. 0.0 - 3.5 ft. - color change from yellowish orange (S-1) to dark yellowish brown (S-2) from west end of trench to east end of trench, very distinct vertical line separating colors - material is very dense below approx. 3.5'-4.0'
∇ 3.5			
5.0	Bedrock -		Joint orientation: Strike N 60°E Dip 27°NW

Note: Took Photos 1 and 2 from Roll 2.

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JOB NO. 3235 SUBJECT Trench 4 (T-4) SHEET NO. 1 OF 2

PROJECT Shell Valley Borrow - Emergency Spillway Area BY DJH DATE 07-17-85

Cool overcast

T-4 Location: Near Control Point DOG 9285.49, approx. 100' south of rim of borrow pit, bet. DH 104 and DH 103.

Elev: 9285

Orientation: Approx. NW-SE

Size: 10.9 ft deep x 2-3 ft wide x ft long

Groundwater: Water encountered at 9.1 ft. Flow into trench estimated at 3 to 5 gpm (R. Volpe).

Excavation Notes: (a) C & M Construction - Ben Menzel operator, Harvester 3800 Backhoe w/ 2 ft wide bucket

(b) Start excavation time: 9:40 am

(c) Backfill trench: 10:15 am

(d) Termination at 10.9 ft due to water inflow.

DEPTH (ft)	SAMPLE No.	USCS	DESCRIPTION
0 - 1.5		ML-SM	Topsoil, Silty Fine Sand or Sandy Silt, with organics and roots; Moderate Brown (5 YR 3/4); 40-50% nonplastic fines, 40-50% fine sand; 5-10% organics. - moist
1.5 - 4.1	S-1	SW-SM	Clean to Slightly Silty Sand, gravelly; Dark Yellowish Brown (10 YR 4/2); 5-10% nonplastic fines, 20-30% fine sand, 40-50% med. sand, 5-10% coarse sand, 5% gravel (1/2" max. size); few cobbles and boulders - very moist
4.1 - 6.3	S-5	SM-SC	Silty, Clayey Sand, gravelly; Dark Yellowish Brown (10YR 4/2); 20-30% low plasticity fines; 50-60% fine to med. subangular sand; 10-20% coarse subangular sand, 5-10% subangular gravel (1/2" max. size). Irregular, non-continuous contact to:
(5.7)	S-2	SW (Dg)	Clean Sand, gravelly: Dark Yellowish Orange (10 YR 6/6); less than 5% nonplastic fines, 20-30% angular fine sand, 20-30% angular med. sand, 10-20% angular coarse sand, 5-10% subangular to angular gravel (max. size 1/2"); material composed of disintegrated granite (Dg) pieces and micaceous sand flakes

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 4 (T-4) SHEET NO. 2 OF 2

PROJECT Shell Valley Borrow-Emergency Spillway Area BY DJH DATE 07-17-85

cool
overcast

DEPTH (ft)	SAMPLE NO.	USCS	DESCRIPTION
6.3 - 10.9	B-1, S-3	SW (Dg)	Clean Coarse Sand, gravelly; Multicolored from Dark Yellowish Orange (10 YR 6/6) to Dark Yellowish Brown (10 YR 4/2); less than 5% non plastic fines; 10-20% angular fine to med sand; 60-70% subangular to angular coarse sand; 10-20% subangular - angular gravel (max size 3/4"); Color change from Orange to Brown at 9.1 ft indicated by a distinct horizontal line (note: same depth as water table); Material composed of disintegrated granite pieces grading coarser with depth.
(9.1)	S-4 (color change)		

Note: Photos 3 and 4 from Roll 2.

R. Volpe Supervising
Log by DJH, Sampling by SWB.

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 5 (T-5) SHEET NO. 1 OF 1

PROJECT Shell Valley Borrow - Emergency Spillway Area BY DJH DATE 07-17-85

cool drizzle

T-5

Location : 25.5 ft East of Adelaide Lake access road, 234 ft NW along road from the A.L. access road intersection with road to Shell Lake.

Elev : 9278 (topo)

Orientation : Parallel with access road (NW-SE)

Size : 9.1 ft deep x 2-3 ft. wide x

Groundwater : Observed water running into trench through a coarse wedge on the sidewall between approximately 4.5 to 5.8 ft depth. Noticed increased inflow with time to > 1-2 gpm before trench was backfilled.

Excavation notes : (a) C:M Construction, Ben Menzel operator, Harvestor 3800 backhoe with 2 ft wide bucket

(b) Start excavation time : 10:30 am

(c) Backfill trench : 11:20 am

(d) Refusal at 9.1 ft due to large boulders at both ends of trench

DEPTH (ft)	SAMPLE NO.	USCS	DESCRIPTION
0 - 5.8	B-1, S-1	SM	Silty Sand, gravelly with cobbles and boulders; Moderate Yellowish Brown (10YR 5/4); 10-20% nonplastic fines, 60-70% fine to coarse subrounded sand, 10% subrounded to subangular gravel, 10% cobble to boulder size rocks (ave. cobble size 3-4", ave. boulder size 1-2 ft) - dry to slightly moist
▽ - - 4.5			
(5.3-5.8)	S-2	SM	Sand fraction slightly coarser gradation - no significant change in other gradation notes above. - moist
▽ - - 5.8			
5.8 - 9.1		SM (Dg)	Silty Coarse Sand, gravelly with cobbles and boulders; Multicolored from Dark Yellowish Orange (10YR 6/6) to Dark Yellowish Brown (10YR 4/2); 10-20% nonplastic fines; 10-20% subangular to angular fine-med sand, 40-50% subangular to angular coarse sand; 10-20% cobble to boulder size - significant increase in number of boulders between 8.0 to 9.1 ft - wet
(8.0-8.5)	B-2		

note: Photo 5 - Roll 2 (after trench backfilled only)

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 6 (T-6) SHEET NO. 1 OF 1

PROJECT Shell Valley Borrow - Emergency Spillway Area BY DJH DATE 07-17-85

Rain
Cool

T-6 Location : 100 ft North of Adelaide Lake perimeter road intersection with road to Shell Lake from where road bends, Approx. 150 ft SE of DH 104.

Elev. : 9285 (topo)

Orientation : (NW-SE)

Size : 8.0 ft deep x 2-3 ft wide x

Groundwater : no water encountered

Excavation notes : (a) C & M Construction, Ben Menzel operator, Harvester 3800 Backhoe w/ 2 ft wide bucket.
(b) Start excavation time: 1:04 pm
(c) backfill trench : 1:43 pm
(d) refusal at 8.0 ft due to large boulder

DEPTH (ft)	SAMPLE NO.	USCS	DESCRIPTION
0 - 1.0		ML	<u>Silty Topsoil</u> ; Moderate Brown (5YR 4/4) with roots and organics, - dry
1.0 - 2.0	S-1	SM	<u>Silty Sand</u>, gravelly with some cobbles and few boulders ; 15-20% nonplastic fines, 30-40% fine to med. subrounded to subangular sand ; 20-30% subrounded to subangular coarse sand ; 5-10% gravel (max. size 1") - dry to moist
2.0 - 8.0	S-2	SM	<u>Silty Sand</u>, gravelly with cobbles and boulders increasing with depth ; 20-30% nonplastic fines, 20-30% fine - med subrounded - subangular sand ; 20-30% coarse sand ; 5-10% gravel ; many 3-4" size cobbles and 1-2' size boulders. Some cobbles completely decomposed and visible on sidewall. - moist
(4.0-8.0)	B-1 S-3	SM	- Same soil gradation as above, but noticable increase in number of 1-2' size boulders and increase in moisture, 4'-8' size boulders present below ~ 4.0 ft

note: Photos # 6 & 7 - Roll 2 (Trench open)
16 - Roll 2 (Trench closed · 7-18-85)

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 7 - (T-7) SHEET NO. 1 OF 1

PROJECT Shell Valley Borrow - Emergency Spillway Area BY DJH DATE 7-17-85

Rain
Cool

T-7 Location: 180 ft North of access road from Shell Lake, near DH 103

Elev: 9286

Orientation: NW-SE

Size: 11.2 ft deep x 2-3 ft wide x

Groundwater: no water encountered

Excavation notes: (a) C&M Construction, Ben Menzel operator, Harvester 3800 Backhoe w/ 2 ft bucket

(b) Start excavation: 2:00 pm

(c) Backfill trench: 2:45 pm

(d) Stop excavation at 11.2 ft - end of backhoe reach

DEPTH (ft)	SAMPLE NO.	USCS	DESCRIPTION
0 - 1.0		ML	Silty Topsoil; Moderate Brown (5YR 4/4), with roots and organics. - dry
1.0 - 4.5	S-1	SM	Silty Sand, gravelly w/ cobbles and boulders; Dark Yellowish Brown (10YR 4/2); 15-20% nonplastic fines, 70-80% fine to med., subrounded to subangular sand; 5% gravel to cobble size (ave. cobble size 3-4"); several 1-3' size and one ~4' size boulder removed near surface - moist
4.5 - 11.2 (4.5) (5.0-6.0)	 S2 B-1	SP (Dg)	Clean Coarse Sand, gravelly. Multicolored from Dark Yellowish Orange (10YR 6/6) to Dark Yellowish Brown (10YR 4/2); less than 5% fines, 70-80% med. to coarse, angular sand, 20-30% subangular to angular gravel. Gradation gets coarser with depth and percentage of gravel size increases. - moist

note: all material was easily rippable, although appeared to be very dense near bottom of trench

note: Photos 8 & 9 - Roll 2

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 8 (T-8) SHEET NO. 1 OF 1

PROJECT Shell Valley High Water Level for Proposed Dam on South Wall of Reservoir BY DJH DATE 7-17-85

Rain
Cold

T-8 Location: 160 ft north of access road intersection with road to Buckley Creek, between DH 106 and DH 109

Elev: 9276 (topo) - 26.8 ft above reservoir water level measured by hand level

Orientation: NW-SE

Size: 9.0 ft deep x 2-3 ft wide x 31.8' long

Groundwater: encountered water at 3.5 ft depth, appeared to be seeping in through SM material (2.5-9.0 ft) near contact with SM-SC material (0.8-2.5 ft)

Excavation Notes:
(a) C&M Construction, Ben Menzel operator, Harvester 3800 Backhoe w/ 2 ft bucket
(b) Start excavation 3:00 pm
(c) End excavation 3:45 pm - left trench open overnight to observe water inflow
(d) backfill trench 8:30 am (7-18-85)

DEPTH	SAMPLE NO.	USCS	DESCRIPTION
0-0.8		ML-CL	Silty, Clayey Topsoil; Moderate brown (5YR 3/4); 80-90% low plasticity fines, 10-20% fine sand. -moist
0.8-2.5	B-1 S-1	SM-SC	Silty, Clayey Sand: Moderate yellowish Brown (10YR 5/4), 30-40% low plasticity fines; 50-60% fine to med. rounded sand (1/4" max. size) 0-5% gravel, -moist
2.5-9.0 ▽ 2.5 ▽ 3.5	B-2 S-2	SM	Silty Sand, gravelly with boulders and few cobble size rocks; Dark yellowish brown (10YR 4/2); 20-30% low plasticity fines; 20-30% subrounded fine to med sand, 30-40% coarse sand, 5-10% gravel; few cobble sized rocks and several 2'-3' boulders.

note: sides began sloughing in approx. 30 min. after opening trench

note: left trench open overnight to observe water inflow - after 13 hrs very little water had entered trench

note: Photos 10, 11 (trench open) and 14 and 15 (7-18-85)

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 9 (T-9) SHEET NO. 1 OF 1
PROJECT Shell Valley Borrow - Emergency Spillway Area BY DJH DATE 7-17-85

T-9 Location: Low point in borrow pit north of access road near DH 105
Elev. : 9272 (topo)
Orientation : E-W
Size : 7.5 ft deep x 2-3 ft wide x 40 ft long
Groundwater : encountered saturated soil at 7.0-7.5 ft. but little water seeping into trench.
Excavation notes : (a) C & M Construction, Ben Menzel operator, Harvester 3800 Backhoe w/ 2' bucket
(b) Start excavation 4:05 pm
(c) Backfill trench 5:10 pm
(d) Had to extend length of trench due to encounter with several very large (>5') boulders
(e) refusal at 7.5 ft due to large boulder

DEPTH (ft)	SAMPLE NO.	USCS	DESCRIPTION
0-5.0 (East end of trench)	B-1 S-1	SM	Silty sand, gravelly with cobbles and boulders; Moderate yellowish brown (10 YR 5/4); 15-20% nonplastic fines, 20-30% subrounded fine to med. sand, 20-30% coarse sand, 10-20% gravel; cobbles (ave. size 3") and boulders (2-3') throughout - note: encountered 2 or 3 very large boulders (>5') at approx. 2' to 3' so extended length of trench. - moist to very moist below 2'
(West end of trench) 0-0.6		ML	Silty topsoil; - dry
0.6-1.9	S-2	SM-SC	Silty, Clayey Sand, gravelly with cobbles and boulders; Dark Yellowish Brown (10 YR 4/2); 30-40% low plasticity fines; 10-20% fine - med. sand; 20-30% coarse sand; 5-10% gravel. - moist
1.9-7.0	-	SM	[See description for 0-5.0' material at E. end]
▽ 7.0-7.5	S-3	SM	Silty Sand, gravelly with cobbles and boulders; Dark Yellowish Brown (10YR 4/2); 30-40% low plasticity fines, 5-10% fine to med. sand, 30-40% coarse sand, 5-10% gravel. - saturated

note Photos 12-13, Roll 2

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 10 (T-10) SHEET NO. 1 OF 1
PROJECT Shell Valley Left Abutment, ℄ Dam BY SWB/DJH DATE 7-18-85

T-10 - Left Abutment, ℄ Dam Elev. 9254 (Topo & Hand level)
aproximately 100' N85E of DH-2, required little terrain or vegitation alteration for access (one 2" tree pushed over, one 2½' rock moved)
Start excavation: 9:35 am, C&M Construction; Ben Menzel
Complete excavation: 11:10 am (had to make trench 10' wide to get boulders out)
Backfilled trench

LOG	SAMPLES	DEPTH	Description
0 SM 2.5 SM TD 9.0	B-1 (S-1) B-2 (S-2) S-3	0.0-2.5 SM	COBBLY SILTY SAND; Mod yel br (10YR 5/4); ~20% non plastic fines; ~70% coarse to fine grained, subangular, sand with quartz, feldspar, and biotite evident, ~10% cobbles and boulders, gen subangular; ~3 max; 0.3' min; 0.4' average; granite and quartz monsonite present, little weathered, dry. 0.0-0.1: Topsoil; Greyish br (5YR 3/2); ~90% non plastic fines; ~10% sand; mica evident, num. roots ~1.5 Color change to dk yel br (10YR 4/2) sand fraction grades coarser; granite lithics present 2.0-3.0 gradational irregular contact to:
		2.5-9.0 SM	COBBLY SILTY SAND, Dk yel br (10YR 4/2) 5-10% non plastic fines, 75% fine to coarse grained, subrounded to sub angular sand with quartz, feldspar, mica, and granite lithics present, ~10% cobbles and boulders as above, dry 8.0 material becomes damp, fines increase to 30%, sand grades finer.

SAMPLE NOTES
B-1 1.8-2.3
S-1 2.0-2.5
B-2 2.8-3.3
S-2 3.0-3.5
S-3 7.0-7.5

Refusal due to boulders
Pocket Penetrometer readings 0.5 2.0-3.5
1.0-4.0 4.5+
Took photos 17 (open) 18 (filled) roll 2.
SAMPLE NOTES: Took large samples from trench wall at spot intervals specified. Put S-1 in B-1, S-2 in B-2, B samples are ~35 lbs

LOG by SWB SAMPLING BY DJH

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 11 (T-11) SHEET NO. 1 OF 1
PROJECT SHELL VALLEY Left abutment ℄ BY SWB, DLH DATE 7/18/85

T-11 Left Abutment, ℄ Dam ELEV 9239 (Topo & hand level)
aproximately 180' N85E of DH-2, required little terrain or vegetation alteration for access.

Start excavation: 11:40 am C&M Construction; Bw Menzel
Complete excavation: 12:45 pm Trench is 7' wide due to boulders we had to excavate

LOG	SAMPLES	DEPTH	DESCRIPTION
		0.0-6.0 SM	COBBLY SILTY SAND; Mod Brown (5YR 4/3) ~30% non plastic fines, ~35% fine to med. grained, subangular to subrounded sand with feldspar, quartz & mica evident, ~25% subrounded gravels and cobbles, mostly granite and monzonite; ~10% subrounded boulders of granite and monzonite.
SM	S-1, B-1, B-2, S-2		0.0-0.2 TOPSOIL, Greyish br (5YR 3/2); ~85% non plastic fines, ~15% sand; mica evident, numerous roots and organics.
			3.0 grades slightly finer
TD-6.0			3.5 gravel content increases, fines decrease to ~15%
			6.0 Refusal due to boulders

SAMPLE NOTES
S-1 2.5-3.0
B-1 2.5-3.5
S-2 3.5-4.0
B-2 3.5-6.0
all from spoil pile

Pocket penetrometer 4.5+ at depths >1.0'

Took Photo 19 (Spoil Pile) Roll 2, 20 (Trench open)

Log by SWB, sampling by DJH

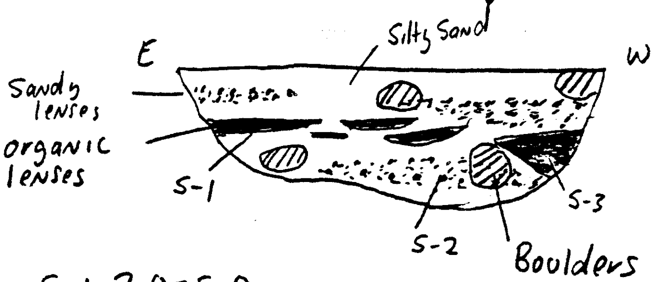
ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT TRENCH 12 (T-12) at SHEET NO. 1 OF 1
PROJECT SHELL VALLEY toe of existing dam ~70' from ℄ BY SWB DJH DATE 7/18/85

T-12 Toe of existing dam, near center, elev. ~9230 (Topo)

Start excavation 2:15 pm C+M CONSTRUCTION Ron Menzel
Complete excavation 3:15 pm

Log	Samples	Depth	Description
SP-SM	B-1	0.0-7.0'	SW-SM BOULDERY SAND AND BOULDERY SILTY SAND, color varies from light br (5YR 5/6) to Dk yel br (10YR 4/2). Mat'l is interbedded bouldery sand; <5% non plastic fines, 85% med. coarse grained sand with some med and fine grained sand, 10% boulders; with bouldery silty sand, 20% non plastic fines, 70% fine to coarse grained sand, 10% boulders, boulders up to 4' present, lots of mica flakes evident, saturated
	S-1		
SP-SM	S-2		0.0-0.4 TOPSOIL
(SO-SC)	S-3	Discontinuous lenses at 3.0-5.8	SO-SC ORGANIC SANDY SILT TO ORGANIC SANDY CLAY; Brownish Black (5YR 2/1); Mat'l varies in composition along trench; at east end it is very micaceous, ~70% mica, 30% organics, non plastic; at west end it is less micaceous ~10% mica, ~30% organics, mod plastic, mat'l has about 30% sand, some boulders still present, saturated
TD-7		7.0	Hole caving due to water infilling, Backfilled trench



Trench is 22' long 6-7' deep

Took photos 21, 22, 23 of open trench roll 2, 24 of covered trench

S-1 3.0-5.0
S-2 4.0-7.0
S-3 3.8-4.3
B-1 1.0-7.0 (from spoil pile)

Log by SWB sampling by DJH

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 13 (T-13) left SHEET NO. 1 OF 1
PROJECT Shell Valley abutment near toe of proposed dam BY SWB/DJH DATE 7/18/85

T-13 Left abutment near downstream toe of proposed dam, elev 9234 (topo) about 90' west of T-11, required little terrain or vegetation alteration for access

Start excavation 4:20 pm C+M Construction Ben Menzel
Complete excavation: 5:30 pm
Covered trench 7/19/85

Log	Samples	Depth		Description
SM	S-1	0.0-2.0	SM	Bouldery to cobbly silty sand, Mod brown (5YR 4/3); ~30% non plastic fines, ~35% pred fine w/some med grained, micaceous sand, 35% gravels, cobbles, and boulders w/ boulders dominating, granite and quartz monzonite present in subrounded clasts.
SW-SM	S-2			
	S-3			>1" Topsoil
		2.0-4.0	SP-SM	Bouldery to gravelly sand to bouldery to gravelly silty sand; as above except fines decrease to 5-10%; sand is coarser - coarse to fine grained, and gravel content increases; gravels are sub angular.

Sample Notes
S-1 1.0-2.0
S-2 2.0-4.0
S-3 5.0-6.0
Did not take a 35 lb B because mat'l was similar to T-11

6.5 Refusal due to numerous boulders, trench measures 13.5' long 6' wide. Removed 6 boulders >3'
Pocket penetrometer 4.5+ except for first 1.0', occasional spot reading of 3.0-3.5
Took photos 25, 26, 27, of Roll 2, 1 (covered) of Roll 3
spoil pile

log by SWB sampling by DJH

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 14 (T-14) ~20' of DH-7 as staked SHEET NO. 1 OF 1
PROJECT SHELL VALLEY BY SWB DJH DATE 7/19/85

T-14 Near center of dam, approaching downstream toe

Start excavation: 9:50 C&M Construction - Ben Menzel
Complete excavation: 11:00
Backfilled Trench

LOG	SAMPLES	DEPTH		DESCRIPTION
0.0		0.0-2.0	SM	BOULDERY SILTY SAND; Mod brown (5YR 4/6) ~25% non plastic fines, ~35% fine to coarse grained, subrounded to subangular sand, ~10% gravels and cobbles, 30% subrounded boulders, mica present in fines and sand, quartz & feldspar evident, granite lithics present in gravels, cobbles and boulders are granite and monzonite, dry
SM	S-1			0.0-0.3 Topsoil Greyish br (5YR 3/2), ~80% non to slightly plastic fines, ~10% fine to coarse grained sand, ~10% organics.
2.0		2.0-4.0	SM	GRAVELLY TO COBBLY SILTY SAND; Mod Brown (5YR 4/4), as above except ~20% fines, 35% sand, 30% gravels and cobbles and 10% boulders.
SM	S-2, B-1			~3.5 fines decrease to 15%, sand is pred. coarse grained.
4.0	S-3	4.0-6.0	SP	GRAVELLY SAND; Mod br (5YR 4/4) with multi colored - white, pink, black, clasts, gen ~5% fines, ~45% med to coarse grained, subrounded sand, 30% subrounded gravels, 10% cobbles and boulders.
SP	S-4			4.5 seep present, material resembles old stream channel with clean graded sand beds. material is v. moist below this zone
6.0	S-5			

Trench measures 13' long 7' wide, removed 3 boulders at 3' to 3½' size, ~10 boulders of 1'-2' size.
6.0 Refusal due to nested boulders

Sample Notes

S-1	1.5-2.0
S-2	2.0-3.5
S-3	3.5-4.0
S-4	at 4.7
B-1	3.5-4.5
S-5	4.0-6.0

Took #2, 3, 4 photos roll 3

log by SWB sampling by DJH

ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 15 (T-15) SHEET NO. 1 OF

PROJECT Shell Valley East Side Reservoir Wall BY DJH DATE 7-19-85

T-15 Location : ~330 ft. NW of T-8 N35°E - 313' from DH-11
10.4 ft by hand level above reservoir w/L - high water mark
Elev : 9256.3'
Orientation: Parallel with reservoir shoreline (E-W)
Size : 8.5 ft. deep x 2 ft wide x 22 ft long
Groundwater : Observed water seeping in at isolated pts. on sidewalls bet. 5.0-6.0' at slow rate
Excavation notes : Start time : 2:30 pm
Complete time - backfill : 3:45 pm

Log	Samples	Depth	Description
0 (ML) 0.8 (SM-SC) 2 (SC) 4 (SM-SC) 8.5 T.D.		0 - 0.8	Topsoil (ML)
	S-1	0.8 - 2.0	(SM-SC) Silty, Clayey Sand, gravelly; Dark Yellowish Orange (10YR 6/6); 30-40% low plasticity fines, 50-60% fine to medium sand with interspersed dark micaceous specks, 5-10% gravel; very few cobble size - moist
	S-2	2.0 - 4.0	(SC) Clayey Sand, gravelly, with cobbles; Dark Yellowish Orange (10YR 6/6); 30-40% low to med. plasticity fines, 20-30% fine to med. sand, 20-30% coarse grained, subangular sand, 5-10% subrounded to subangular gravel - very moist
	B-1 (S-3)	4.0 - 8.5	(SM-SC) Silty, Clayey Sand, gravelly; Dark Yellowish Brown (10YR 4/2); 20-30% low plasticity fines, 40-50% med to coarse grained, subrounded to subangular sand, 10-20% gravel - few cobble size - very moist to saturated 5.0-8.5' 5-10% cobble size - wet at 5.0' saturated at 6.0'
	S-4	3.0-3.5'	SP - Micaceous Sand Pocket; Dusky Brown (5YR 2/2) 6 in diameter pocket of dark, angular, micaceous sand

Soil Layers Linear, Horizontal, and Distinctive

Took Photos 6, 7 & 8, Roll 3

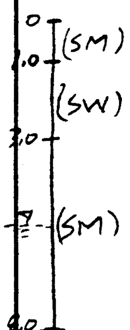
ESA Geotechnical Consultants

JOB NO. 3235 SUBJECT Trench 16 (T-16) SHEET NO. 1 OF 1

PROJECT Shell Valley Low Water Level - East side of Reservoir BY DJH DATE 7-19-85

T-16 Location: ~174' N (N28°W) of T-15, ~336' E of T-5

Elev : 5.2' above reservoir w/L = 9251
Orientation: Parallel with shoreline (NW-S
Size : 8.0 ft deep x 2ft wide
Groundwater : Water seepage between 5.2 to 5.4'; sidewalls saturated below 5'
Excavation notes : Start time: 4:00 pm
Complete time: 5:12 pm backfilled trench

Log	Samples	Depth	Description
	S-1	0 - 1.0'	(SM) Silty Sand, gravelly with cobbles and boulders; Mod. Yellow Brown (10 YR 5/4); 10-20% non plastic fines, 50-60% fine to medium subrounded to subangular sand, 10-20% coarse sand, 5-10% gravel 5-10% cobbles - moist
	S-2	1.0' - 3.0'	(SW) Clean Sand, gravelly, with cobbles. Mod. Yellowish Brown (10 YR 5/4); <5% fines, 70 - 80% fine to coarse graded subrounded to subangular sand, 10-20% subrounded to subangular gravel, 5% cobbles
	S-3	3.0 - 8.0'	(SM) Silty Sand, gravelly, with cobbles and boulders; Dk. Yellow Brown (10 YR 4/2); 20-30% low plasticity fines, 40-50% med. to coarse grained, subrounded to subangular sand 10-20% gravel to cobble size wet at 4.0 ft, saturated at 5.4 feet 3-4, 1.0-2.0 ft size boulders (Very similar to T-15 material between 4.0 - 8.5 ft depths

Took Photos 9, 10, 11 - Roll 3

Water observed seeping in at 2 isolated locations at 5.2 to 5.4 ft depths.

Sidewalls below 5.0' depth saturated

WATER PRESSURE TEST CALCULATION SHEET

Proj. No. 32??

BORING NO. ΔH-8   BORING SIZE NX   DEPTH TO GROUND WATER 4.0 FT   HEIGHT OF PRESSURE GAGE ABOVE GROUND (Δh) 3.3 FT

HOLE RADIUS (r) 0.13 FT   Cu $f(\frac{L}{b+\Delta h}, \frac{H}{r})$, Cs $f(\frac{L}{r})$ (From USBR G-97 Chart) $F = Cu \cdot r$ or $F = (Cs+4)r$ $K = \frac{Q}{FH}$ (70267)

TEST NO.	LENGTH OF TEST INTERVAL L (FT)	WATER LOSS Q (GPM)	(a) APPLIED PRESSURE (PSIx2.31) (FT)	(b) DEPTH TO CENTER OF TEST INTERVAL (FOR TESTS ABOVE WATER TABLE) (FT)	(c) DEPTH TO WATER TABLE (FT)	Δh (FT)	(d) FRICTION HEAD LOSS (FT)	EFFECTIVE WATER PRESSURE IN TEST INTERVAL H (a+(b or c)+Δh-d) (FT)	Cu or Cs VALUE	F VALUE	CALCULATED PERMEABILITY COEFFICIENT K (FT/YR)
1	5.0 (42-47)	0.02	69.3	—	4.0	3.3	—	76.6	63	8.4	2.2
2	5.0 (47-52)	0.02	115.5	—	4.0	3.3	—	122.8	63	8.4	1.4

WATER PRESSURE TEST CALCULATION SHEET — Proj No 235

BORING NO. DH-9 BORING SIZE NX DEPTH TO GROUND WATER 3.2 FT HEIGHT OF PRESSURE GAGE ABOVE GROUND (Δh) 2.3 FT

HOLE RADIUS (r) 0.125 FT

Cu $f(\frac{L}{b+\Delta h}, \frac{H}{r})$, Cs $f(\frac{L}{r})$ (From USBR G-97 Chart)

F = Cu·r or F = (Cs+4)r

$K = \frac{Q}{FH}$ (70267)

TEST NO.	LENGTH OF TEST INTERVAL L (FT)	WATER LOSS Q (GPM)	(a) APPLIED PRESSURE (PSIx2.31) (FT)	(b) DEPTH TO CENTER OF TEST INTERVAL (FOR TESTS ABOVE WATER TABLE) (FT)	(c) DEPTH TO WATER TABLE (FT)	Δh (FT)	(d) FRICTION HEAD LOSS (FT)	EFFECTIVE WATER PRESSURE IN TEST INTERVAL H (a+(b or c)+Δh-d) (FT)	Cu or Cs VALUE	F VALUE	CALCULATED PERMEABILITY COEFFICIENT K (FT/YR)
1	8.0 (11-19)	1.06	23.1		3.2	2.3	—	28.3	90	11.75	224
2	8.0 (11-19)	2.43	46.2		3.2	2.3		51.7	90	11.75	281
3	10.0 (19-29)	1.11	46.2		3.2	2.3	—	51.7	110	14.25	106
4	10.0 (19-29)	1.42	69.3		3.2	2.3	—	79.8	110	14.25	94

WATER PRESSURE TEST CALCULATION SHEET

Proj No 3235

BORING NO. DH-10 BORING SIZE NX DEPTH TO GROUND WATER 16.5 FT HEIGHT OF PRESSURE GAGE ABOVE GROUND (Δh) 2.5 FT

HOLE RADIUS (r) 0.125 FT

Cu $f(\frac{L}{b+\Delta h}, \frac{H}{r})$, Cs $f(\frac{L}{r})$
(From USBR G-97 Chart)

$F = Cu \cdot r$
or
$F = (Cs+4)r$

$K = \frac{Q}{FH}(70267)$

TEST NO.	LENGTH OF TEST INTERVAL L (FT)	WATER LOSS Q (GPM)	(a) APPLIED PRESSURE (PSIx2.31) (FT)	(b) DEPTH TO CENTER OF TEST INTERVAL (FOR TESTS ABOVE WATER TABLE) (FT)	(c) DEPTH TO WATER TABLE (FT)	Δh (FT)	(d) FRICTION HEAD LOSS (FT)	EFFECTIVE WATER PRESSURE IN TEST INTERVAL H (a+(b or c)+Δh-d) (FT)	Cu or Cs VALUE	F VALUE	CALCULATED PERMEABILITY COEFFICIENT K (FT/YR)
1	13.0 (23-36)	0.13	23.1		16.5	2.5	-	42.1	130	16.75	13
2	13.0 (23-36)	0.53	46.2		16.5	2.5	-	65.2	130	16.75	34
3	13.0 (23-36)	0.60	69.3		16.5	2.5	-	88.3	130	16.75	28.5
4	13.0 (35.8-48.8)	0.03	69.3		16.5	2.5	-	88.3	130	16.75	1.4
5	13.0 (35.8-48.8)	0.05	92.4		16.5	2.5		111.4	130	16.75	1.9
6	13.0 (35.8-48.8)	0.08	115.5		16.5	2.5		134.5	130	16.75	2.5

APPENDIX B

LABORATORY INVESTIGATION

APPENDIX B

LABORATORY INVESTIGATION

A. INTRODUCTION

This appendix includes a discussion of test procedures and results of the conventional laboratory investigation performed by ESA Geotechnical Consultants for the engineering evaluation of Lake Adelaide Dam. The purpose of the laboratory investigation was to study the soil engineering characteristics of the various foundation and embankment materials, in order to determine the soil engineering parameters to be used in the various engineering analyses associated with the Level III design studies. The investigation program was carried out employing, wherever possible, currently accepted test procedures of the American Society of Testing and Materials (ASTM). Certain Phases of the investigation, such as the triaxial testing, were carried out employing laboratory testing techniques which have not yet been standardized by ASTM. These testing procedures were carried out in accordance with methods developed by our firm or by other researchers in the field of soil engineering.

Samples used in the laboratory investigation were obtained from various drill holes and trenches during the course of the field investigation. Identification of each sample is by trench or drill hole number, sample number, and depth as defined by the drill hole and trench logs presented in Appendix A. All of the various laboratory tests performed during the course of the investigation are described below. A discussion of the material properties test results, and an interpretation of their engineering significance, is presented in Chapter V of this report.

B. INDEX PROPERTIES TESTING

In the field of soil mechanics and earth dam design, it is advantageous to have a standard method of identifying soils and classifying them into categories or groups that have similar or distinct engineering properties. The most commonly used method at present is the Unified Soils Classification System (USCS) as described by ASTM D2487-69. The USCS is based on recognition of the various types and significant distribution of soil constituents, considering individual grain-size, magnitude and type of gradation characteristics, and plasticity of materials.

Since all soils are a natural product of geological and environmental factors, their engineering properties are inherently variable, especially when compared to other typical construction materials such as steel or concrete. From a macroscopic standpoint, this variability certainly influences the major soil engineering properties of strength, compressibility, and permeability. For economic reasons, however, it is not possible nor economically desirable to perform sophisticated engineering properties tests on each different sample type found at a particular project site. Instead, as in the approach used in this investigation, an attempt is made to delineate completely the range of pertinent engineering properties for all of the major soil types encountered. Then, by interpolating the USCS results, a far more realistic picture can be obtained of the materials' engineering properties variability.

The index properties test results presented in this report include the determination of natural water content and in-place dry density, specific gravity, Atterberg Limits and grain-size distribution for both the foundation and borrow area materials.

1. <u>Natural Water Content and Dry Density</u>

Due to the coarse grained nature of the foundation and borrow area materials it was not practical to attempt to recover undisturbed samples. Two drive samples were attempted in drill hole DH-7 with limited success. Water contents were determined for all bulk samples and are summarized on Table B-1. The two drive samples were collected in 2.5 inch diameter rings and the dry density results are also presented on Table B-1. The list of sample numbers and depths presented on Table B-1 is a handy reference of all samples.

2. <u>Grain-Size Distribution</u>

The gradation characteristics of the borrow area and foundation materials were determined in accordance with ASTM designation D421-58. Due to the large grain size of the foundation and borrow area materials, the samples tested in the laboratory were limited to approximately 6 inches for the maximum particle size. If necessary, the entire sample was mechanically sieved down to the 3/4-inch size and then a representative portion of the minus 3/4 inch fraction was used to determine the remaining gradation. The final results reflect the engire gradation of the sample.

The samples were soaked in water until individual soil particles were separated and then washed on a #200 mesh sieve. That portion of the material retained on the #200 mesh sieve was oven-dried and then mechanically sieved. A hydrometer analysis was performed for those materials with more than approximately 30% passing the #200 sieve. The hydrometer analysis was performed on the minus #40 material. Sodium hexametaphosphate was used as a dispersing agent. The grain-size distribution test results are presented on Figure B-1, sheets 1 through 8.

3. Atterberg Limits

Most of the materials received in the laboratory were nonplastic. The liquid and plastic limits for those samples determined to be plastic were determined in accordance with ASTM designation D423 and D424. Results of the Atterberg Limits are presented on Figure B-2.

4. Specific Gravity

Specific gravity determinations were made primarily on samples used for compaction or triaxial testing in accordance with ASTM designation D854-58. The specific gravity test results are presented in Table B-2.

5. Soundness Tests

The general soundness of the granitic rock at the site was evaluated using the resistance to degradation of large-size coarse aggregate by abrasion and impact in the Los Angeles Machine (ASTM C535) and the soundness of aggregates by use of sodium sulfate (5 cycles - ASTM C88). These results are presented on Table B-3.

C. ENGINEERING PROPERTIES TESTING

The engineering properties testing performed in conjunction with the Level III studies consisted of compaction, permeability and triaxial shear strength tests. The principal focus of these tests was to determine the engineering properties of compacted materials from the borrow area for use in constructing the new dam to be located downstream of the existing Lake Adelaide Dam.

1. Compaction Tests

Compaction tests were performed on the minus 3/4-inch fraction of materials in accordance with ASTM D-698 Method C except for trench T-5, B-1 which was performed in accordance with ASTM D-1557, Method C. The compaction results were used to control the compacted dry densities for samples used for permeability and triaxial shear testing. The compaction test results are presented in Figure B-3, sheets 1 through 4.

2. Triaxial Shear Tests

a. Fabricated Sample Preparation - Samples for triaxial testing were handled in the same fashion as those for compaction testing in that they were initially scalped on the 3/4 inch sieve in order to limit the size (diameter) of sample. Each sample was moisture conditioned to a water content approximately equal to the optimum moisture content as determined from the compaction test results. The samples were then compacted in a 2.875-inch diameter mold to a dry density equal to approximately 95% of the maximum dry density as determined from the compaction test. For materials from trench T-5, B-1, since the compaction standard was higher than ultimately selected for use in the triaxial test, it was necessary to perform a single point using ASTM D-698 procedures and use this point for control of the triaxial samples. A height to diameter ratio of approximately 2.0 was achieved during sample preparation.

b. Sample Saturation and Consolidation - After trimming, the initial weight and volume measurements were determined; the sample was placed in the triaxial cell, encased in a rubber membrane and sealed to the bottom pedestal and topcap with rubber "O" rings. After securing the triaxial chamber, the cell was filled with water, fitted with a 0.5-inch diameter stainless steel piston for load application, and transported to the saturation bay.

Our main laboratory is equipped with a panel of 3 bays, with individual pressure control to each bay, such that 3 triaxial samples can be simultaneously saturated and/or consolidated at different individual pressures. Bleeding air regulators capable of delivering air pressure up to 200 psi are used to control the top, bottom, and chamber lines leading to the triaxial cells. Each saturation bay is also equipped with constant diameter Pyrex sight tubes, each with a cross-sectional area of 0.155 square inch, which connect with the base of the triaxial cell, and thus to the sample. The sight tubes are easily read to the nearest 0.01 cubic inch.

Where specimens were to be saturated, a back pressure of at least 30 psi was necessary to obtain a sufficient degree of saturation prior to the consolidation phase of the test. In order to determine whether the back pressure applied was causing complete saturation, Skempton's "B" parameter was measured for all samples. A value in excess of 0.95 was considered to represent a fully saturated condition. After achieving complete saturation, the samples were either consolidated isotropically or failed without consolidation.

c. ICU Tests - A total of 6 specimens were tested under isotropically-consolidated-undrained conditions. The chamber pressure was increased to a value in excess of the back pressure by an amount equal to the designated consolidation pressure. The top and bottom drainage lines were simultaneously opened, and the total volume of water expelled from the samples was monitored as a function of time. In some cases, strips of filter paper, placed along the sides of the specimen during setup, were used to accelerate the consolidation process. Once consolidation was complete, the samples were failed in an undrained condition with pore pressure, axial load, and sample deformation monitored as described in (d) below. The results of the ICU triaxial shear testing are presented on Drawing B-4, sheets 1 and 2.

d. Sample Failure - All triaxial specimens were failed by compression loading at a constant rate of strain while maintaining a constant minor principal stress. The rate of strain selected for sample failure, which varied between 5.0 and 10.0 percent per hour, was dependent upon the materials consolidation characteristics. The axial load and pore pressure readings were obtained during the test using an automatic scanning technique. The adopted failure criterion used for the presentation of the Mohr circle of stress for the triaxial tests was the point of maximum principal effective stress ratio.

3. Permeability Tests

Permeability tests were performed using either constant head or falling head test procedures as appropriate. All samples were initially compacted using the same procedures as described above for the triaxial tests. For those samples that did not have a compaction test to determine density control, a maximum dry density was estimated based on gradation characteristics.

All samples except those from T-4, B-1 and T-7, B-1, both of which are decomposed granite from the borrow area, were back pressure saturated prior to performing the permeability tests. All samples except the decomposed granite, were also consolidated prior to testing to an effective consolidation pressure ranging from 5 psi to 40 psi.

The results of the permeability tests are presented on Table B-4.

TABLE B-1

MOISTURE CONTENT SUMMARY SHEET

TRENCH NO.	SAMPLE NO.	DEPTH (ft.)	MOISTURE CONTENT (%)
T-2	B-1	0-6.5	5.7
T-2	B-2	6.5-10.0	5.1
T-3	S-1	0-5.0	4.3
T-4	S-1	1.5-4.1	6.5
T-4	S-5	4.1-6.3	6.6
T-4	B-1	6.3-10.9	3.0
T-5	B-1	0-5.8	5.6
T-5	B-2	8.0-8.5	5.7
T-6	S-2	2.0-4.0	7.0
T-6	B-1	4.0-8.0	6.5
T-7	B-1	5.0-6.0	3.6
T-8	B-1	0.8-2.5	15.6
T-8	B-2	2.5-9.0	5.3
T-9	B-1	0-5.0	7.4
T-10	B-2	2.8-3.3	3.3
T-11	B-1	2.5-3.5	5.7
T-11	B-2	3.5-6.0	3.5
T-12	S-2	4.0-7.0	9.0
T-12	S-3	3.5-4.3	20.1
T-13	S-3	5.0-6.0	5.7
T-14	S-2	2.0-3.5	7.9
T-14	B-1	3.5-4.5	3.3
T-14	S-5	4.0-6.0	6.4
T-15	S-1	0.8-2.0	13.9
T-15	S-2	2.0-4.0	11.8
T-15	B-1	4.0-8.5	8.7
T-16	S-2	1.0-3.0	7.6
T-16	S-3	3.0-8.0	7.1
DH-7	S-4	15.5-16.1	9.0
DH-7	S-6	24.5-25.1	15.6

NOTES:

1. Samples designated by S were approximately 2 pound samples from the side wall of the trenches.
2. Samples designated by B were approximately 50 pound samples from the interval noted.
3. Samples designated by DH were drive samples 2.5 inches in diameter.

TABLE B-2

SPECIFIC GRAVITY RESULTS

TRENCH NUMBER	SAMPLE	SIZE TESTED	SPECIFIC GRAVITY	ABSORPTION (%)
T-5	B-1	MINUS NO. 4	2.67	-
T-15	B-1	MINUS NO. 4	2.66	-
COMBINED*	-	6" - 1/2"	2.69	0.6

*Approximately 5 pounds of representative coarse material was selected from several different trenches for testing.

TABLE B-3

SOUNDNESS TEST RESULTS

LA ABRASION (ASTM C535)	PERCENT WEAR = 14% (1000 revolutions)	
SODIUM SULFATE (ASTM C88)	SIEVE SIZE	PERCENT LOSS
	3/4" - 1 1/2"	0.17
	1 1/2" - 2 1/2"	0.19
	TOTAL LOSS =	0.36%

Note: The materials for the LA Abrasion and Sodium Sulfate soundness tests were obtained as representative coarse grains (in accordance with the specific test requirement) from virtually all samples.

TABLE B-4

SUMMARY OF PERMEABILITY TEST RESULTS

TRENCH	SAMPLE	USCS	PASSING NO. 200 (%)	$\bar{\sigma}_c$ (psi)	γ_d (pcf)	w (%)	e	k (cm/sec)
T-4	B-1	SW-SM	3.0	0	118.2	14.8	0.40	5.2×10^{-3}
T-5	B-1	SM	22.6	10	132.4	9.7	0.26	1.4×10^{-6}
T-5	B-1	SM	22.6	20	132.5	9.6	0.26	4.1×10^{-7}
T-6	B-1	SM	31.5	5	127.5	11.4	0.30	6.6×10^{-6}
T-7	B-1	SW-SM	6.0	0	120.4	14.1	0.38	4.5×10^{-3}
T-8	B-1	SC	33.5	5	110.5	18.9	0.50	1.1×10^{-7}
T-9	B-1	SM	31.6	5	127.4	11.4	0.30	1.1×10^{-5}
T-11	B-2	SM	12.2	5	131.0	10.0	0.27	1.8×10^{-6}
T-15	Combined*	SM	18.4	10	130.5	10.2	0.27	6.6×10^{-5}
T-15	Combined*	SM	18.4	20	130.7	10.2	0.27	5.0×10^{-5}
T-15	Combined*	SM	18.4	40	129.9	10.4	0.28	3.3×10^{-5}

*Sample combined from T-15/B-1, T-15/S-3, T-8/B-1, T-8/S-2

Where: $\bar{\sigma}_c$ = effective consolidation pressure

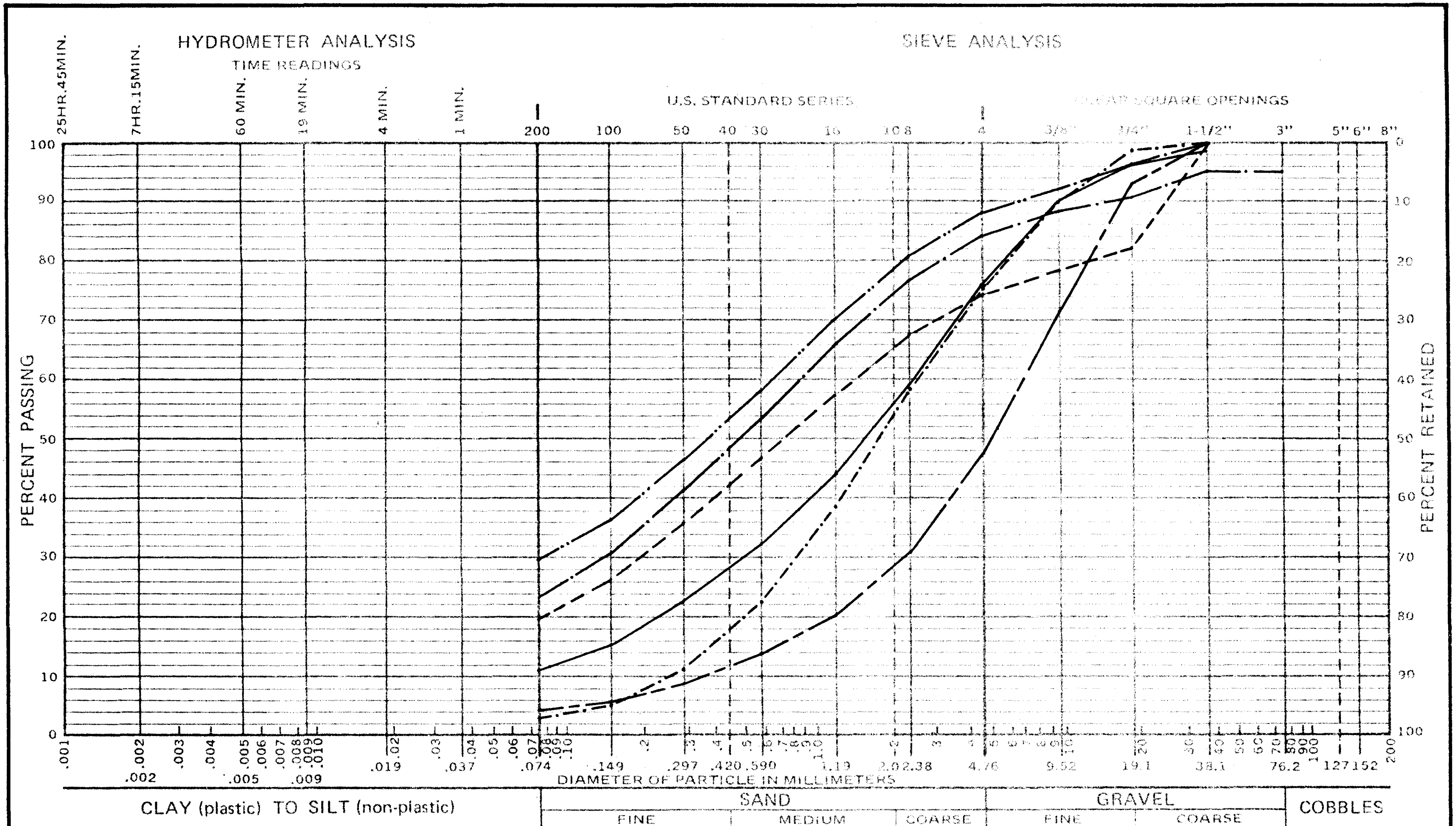
γ_d = dry density

w = water content

e = void ratio

k = coefficient of permeability

Notes:
1. All samples were initially compacted to approximately 95 percent of the maximum dry density defined by ASTM D-698.
2. Samples from T-4 and T-7 were tested in accordance with ASTM D-2434. All other samples were saturated using back pressure, consolidated to the pressure indicated, and then tested in accordance with the U.S. Army Corps of Engineers Method (EM1110-2-1906 - Laboratory Soils Testing Manual, Nov., 1970).
3. The permeability results presented have been corrected to 20°C.



KEY:
- ———————— T-2, B-1
- —·——————·— T-2, B-2
- ——————— — — T-3, S-1
- — — — — — — T-4, S-1
- ————··———— T-4, S-5
- —·—·—·— T-4, B-1

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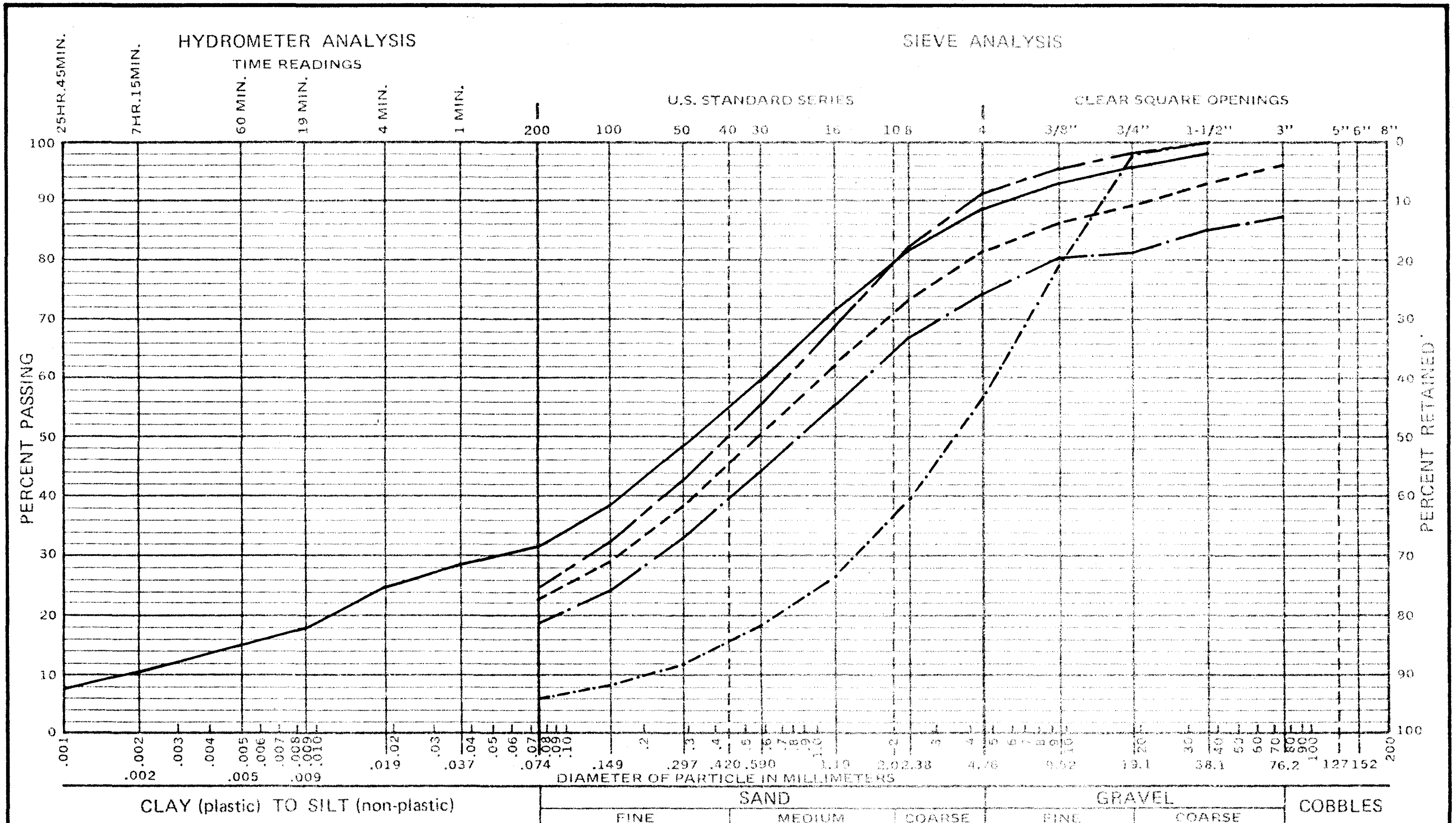
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY

GRADATION TEST RESULTS

Checked by D. J. Hamber	Date 10-22-85	Project No. 3235	Figure No. B-1
Approved by R. L. Volpe	Date 10/22/85		

SHEET 1 OF 8



KEY:
- ——————— T-5, B-1
- —·——·— T-5, B-2
- ————— — — T-6, S-2
- ——————— T-6, B-1
- —·—·—·— T-7, B-1

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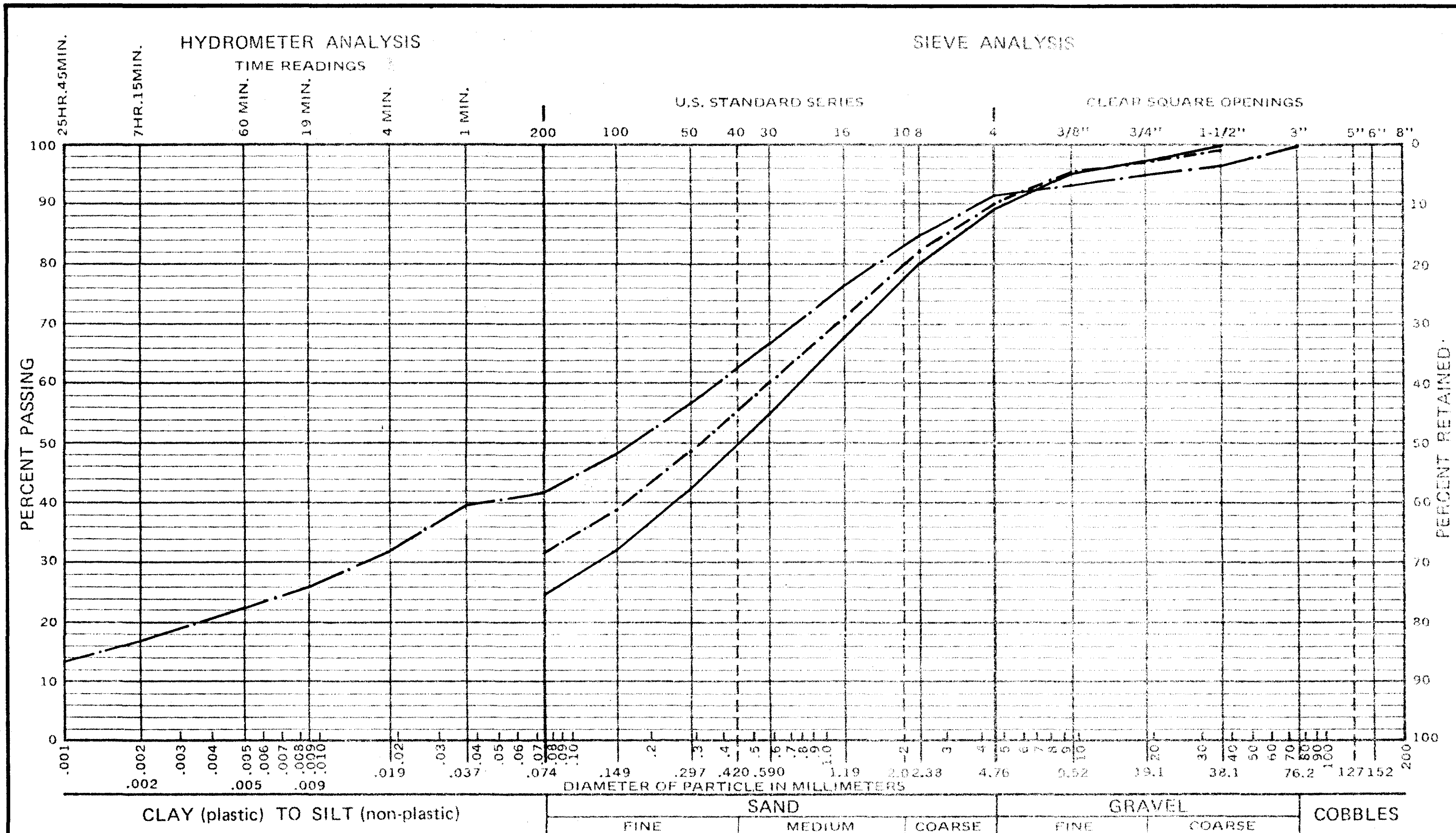
SHELL VALLEY WATERSHED - LEVEL III STUDY

GRADATION TEST RESULTS

Checked by D. J. Hamby	Date 10-22-85	Project No.	Figure No.
Approved by T. R. Volpe	Date 10/22/85	3235	B-1



KEY:
- —·— T-8, B-1
- ——— T-8, B-2
- —·—·— T-9, B-1

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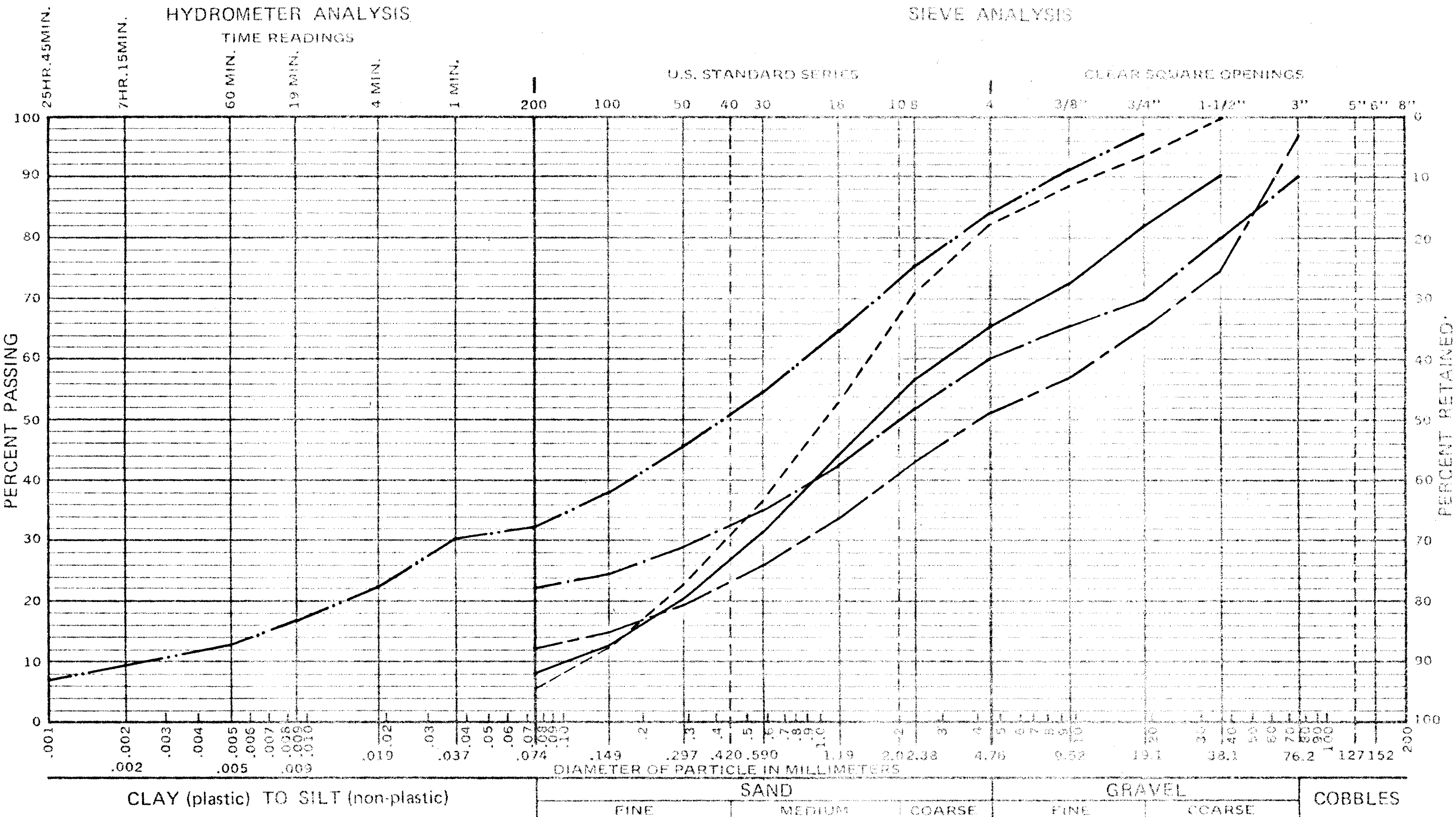
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY GRADATION TEST RESULTS			
Checked by D. J. Humber	Date 10-22-85	Project No. 3235	Figure No. B-1
Approved by R. R. Volpe	Date 10/22/85		

SHEET 3 OF 8



KEY:
- – – – – – – T-10, B-2
- —·—·— T-11, B-1
- —— – —— T-11, B-2
- ———— T-12, S-2
- —··— T-12, S-3

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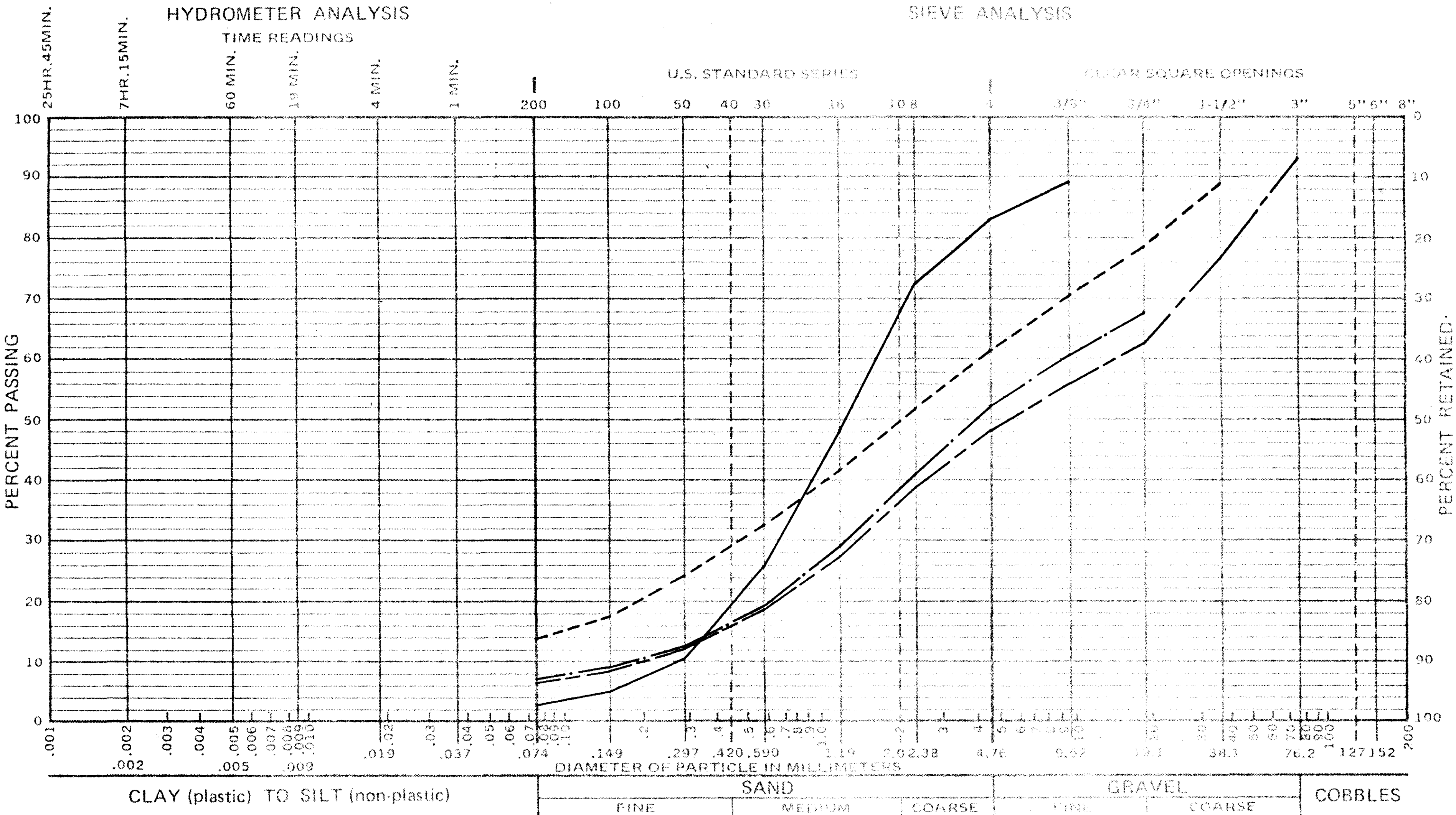
SHELL VALLEY WATERSHED - LEVEL III STUDY

GRADATION TEST RESULTS

Checked by D. J. Hamby Date 10-22-85	Project No.	Figure No.
Approved by R. L. Volpe Date 10/22/85	3235	B-1



KEY:
- - - - - - - T-13, S-3
—·——·— T-14, S-2
—— — — —— T-14, B-1
———————— T-14, S-5

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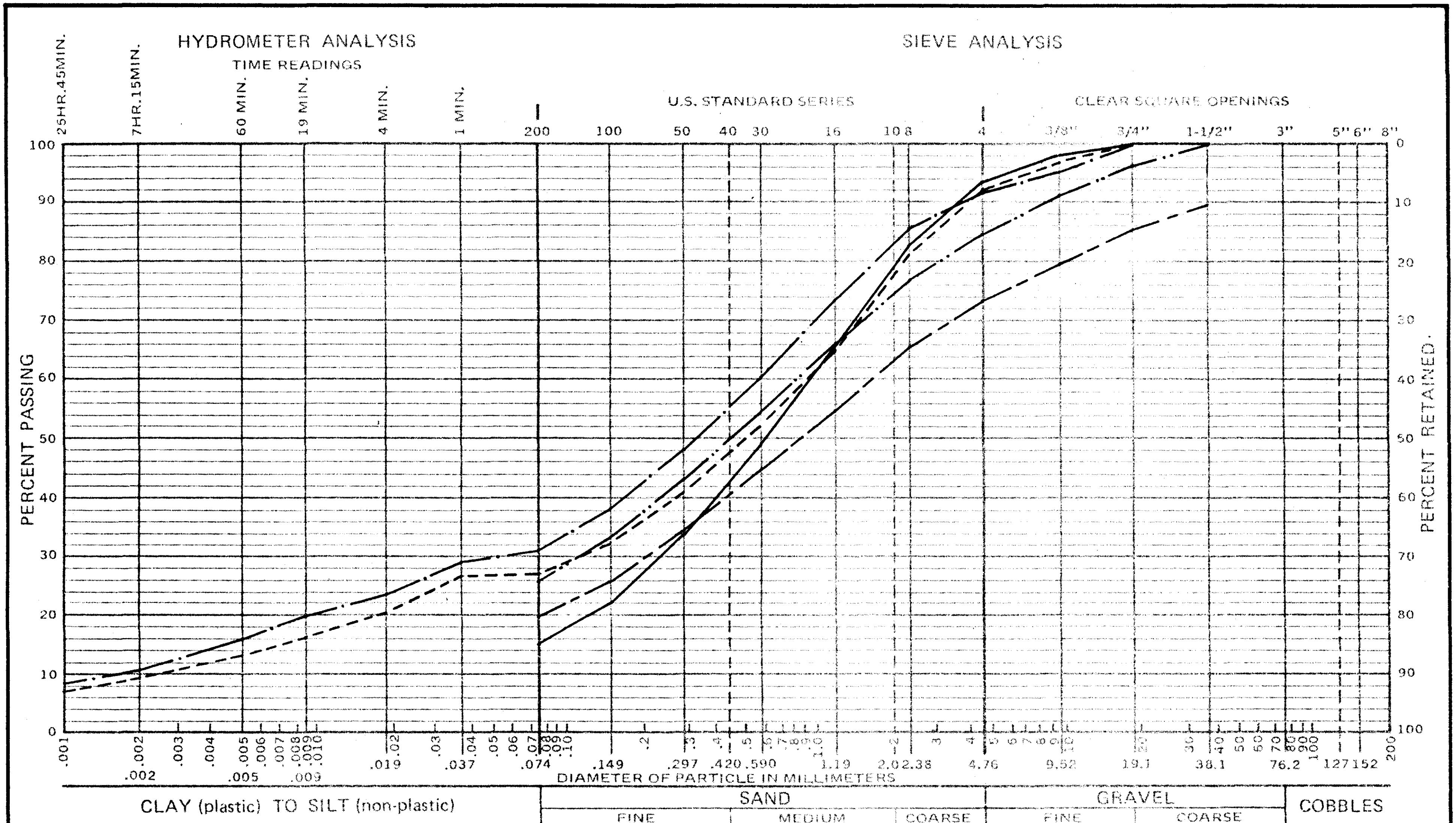
SHELL VALLEY WATERSHED - LEVEL III STUDY
GRADATION TEST RESULTS

Checked by D. J. Hamby	Date 10-22-85	Project No.	Figure No.
Approved by R. K. Volpe	Date 10/22/85	3235	B-1



KEY:
- - - - - - T-15, S-1
—·—·— T-15, S-2
— — — T-15, B-1
———— T-16, S-2
—··— T-16, S-3

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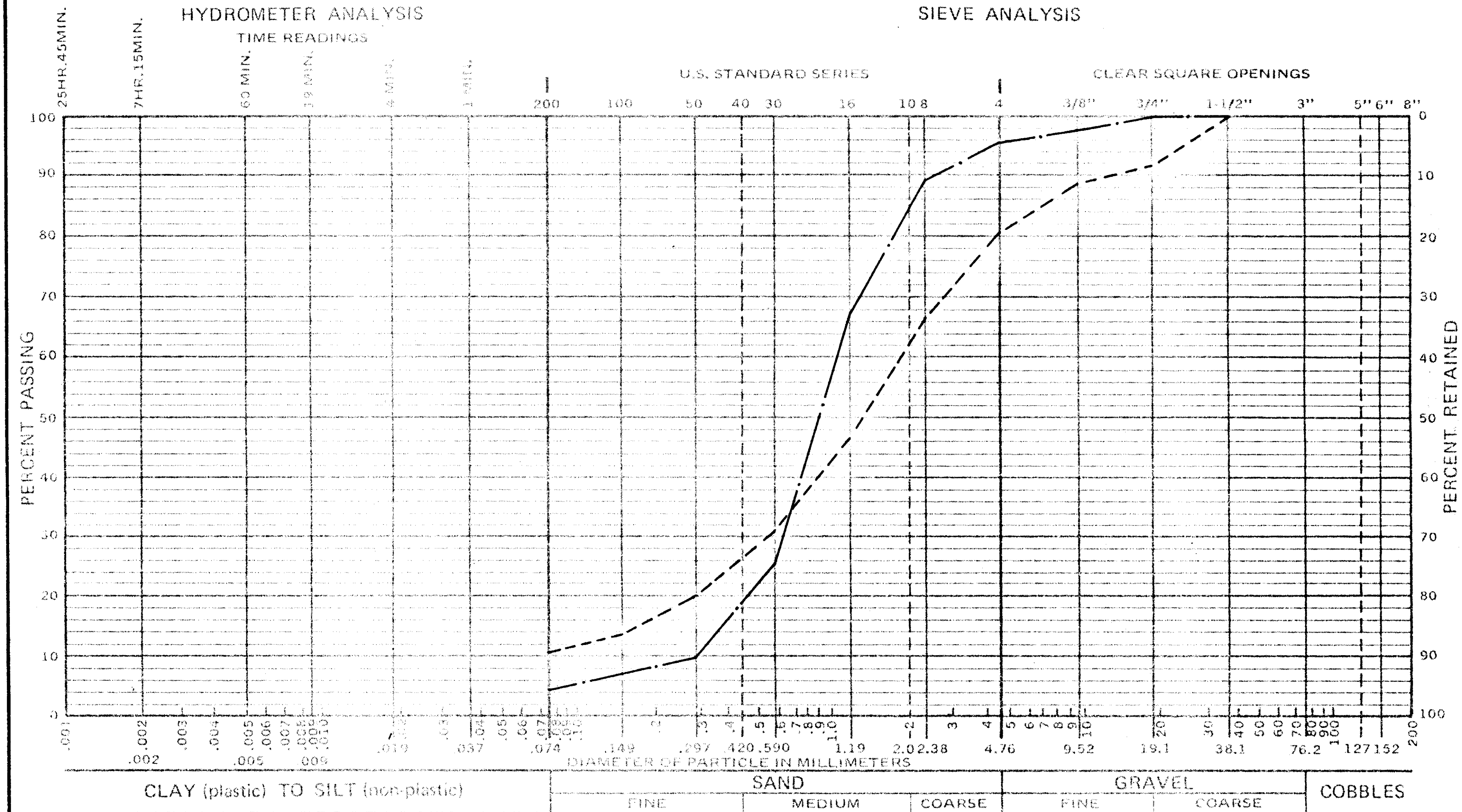
SHELL VALLEY WATERSHED - LEVEL III STUDY

GRADATION TEST RESULTS

Checked by D.J. Hemby Date 10-22-85	Project No.	Figure No.
Approved by R.L. Volpe Date 10/22/85	3235	B-1



KEY: – – – – – – DH-7, S-4
—·——·— DH-7, S-6

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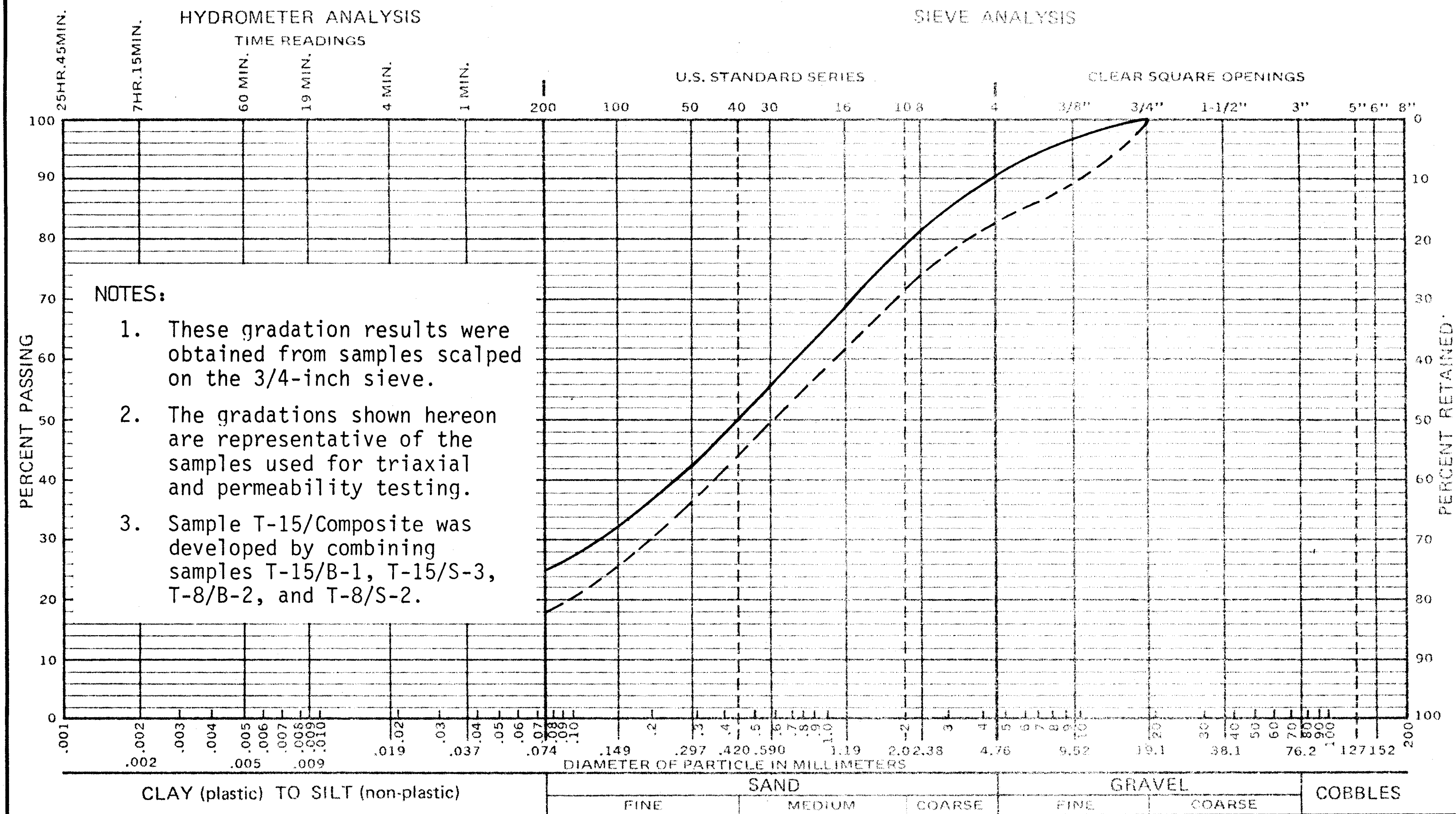
GRADATION TEST RESULTS

Checked by D. D. Hamby Date 10-22-85		Project No.	Figure No.
Approved by R. L. Volpe Date 10/22/85		3235	B-1



KEY:

——————— T-5, B-1

– – – – – T-15/COMPOSITE

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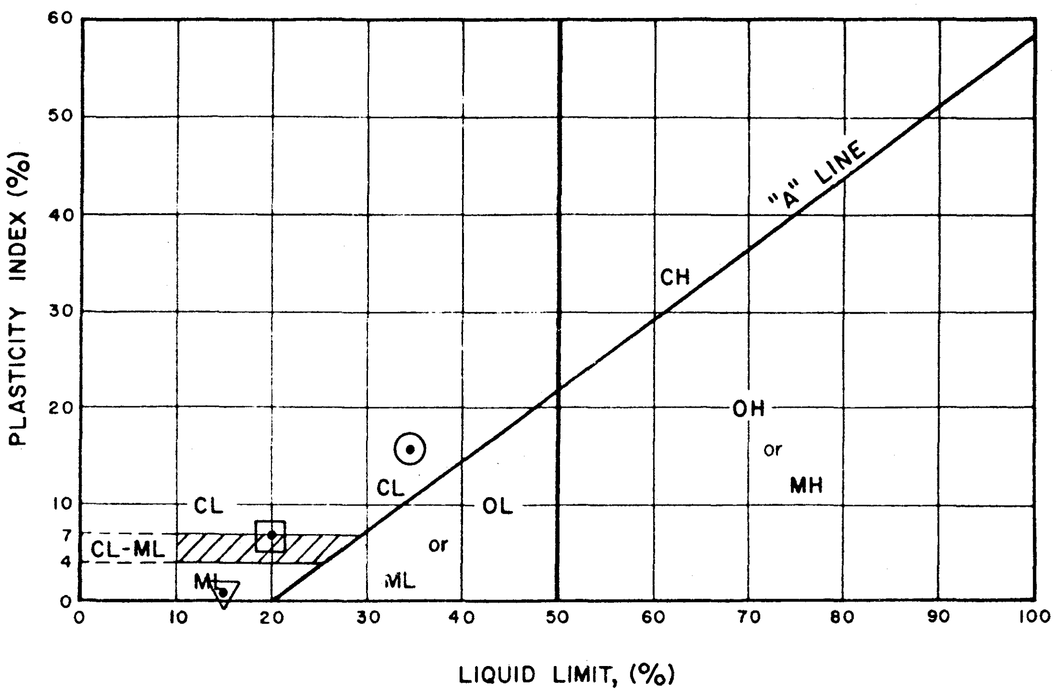
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY GRADATION TEST RESULTS		
Checked by D. J. Hemly Date 10-22-85 Approved by R. J. Volpe Date 10/22/85	Project No. 3235	Figure No. B-1

SHEET 8 OF 8



NOTE: The Atterberg Limits test results shown represent only those materials which exhibited plastic behavior. The majority of soil samples taken from the borrow and dam site areas were non-plastic.

SYMBOL	BORING NO.	DEPTH, FT.	LIQUID LIMIT, %	PLASTICITY INDEX, %	USC SYMBOL
⊙	T-8/B-1	0.8 - 2.5	35	16	SC
⊡	T-15/S-2	2.0 - 2.5	20	7	SC-SM
▽	T-15/B-1	4.0 - 8.5	15	1	SM

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WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
SUMMARY OF ATTERBERG LIMITS

Checked by D. J. Hamby	Date 10-22-85	Project No. 3235	Figure No. B-2
Approved by R. L. Volpe	Date 10/22/85		

SAMPLE NO. T-5/B-1 DEPTH 2.0-3.0 FT. ELEVATION 9274 FT.

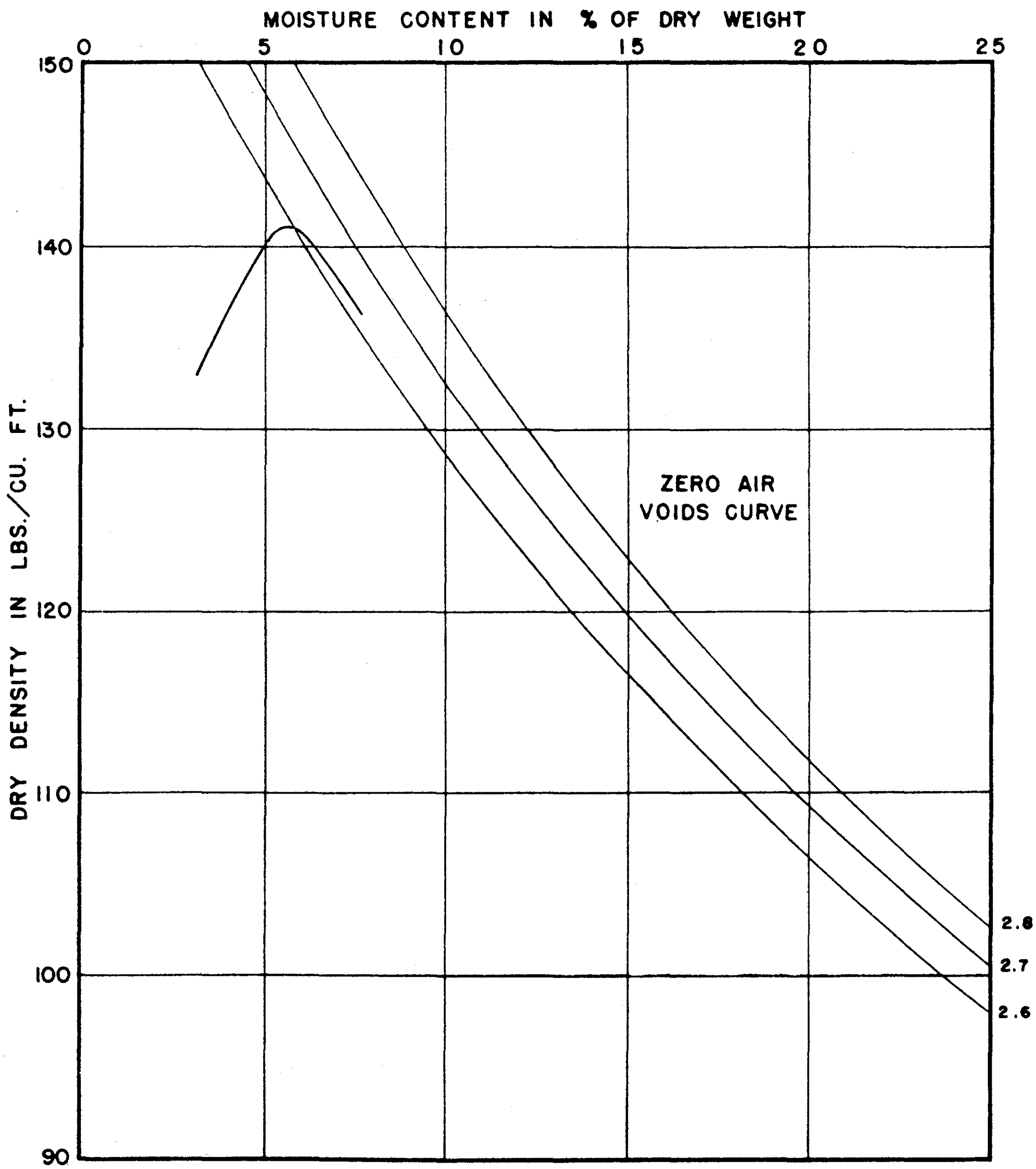
SOIL GRAVELLY, SILTY SAND (SM)

LOCATION ADELAIDE LAKE BORROW AREA

OPTIMUM MOISTURE CONTENT 5.7%

MAXIMUM DRY DENSITY 141.0 pcf

METHOD OF COMPACTION ASTM D-1557 METHOD C



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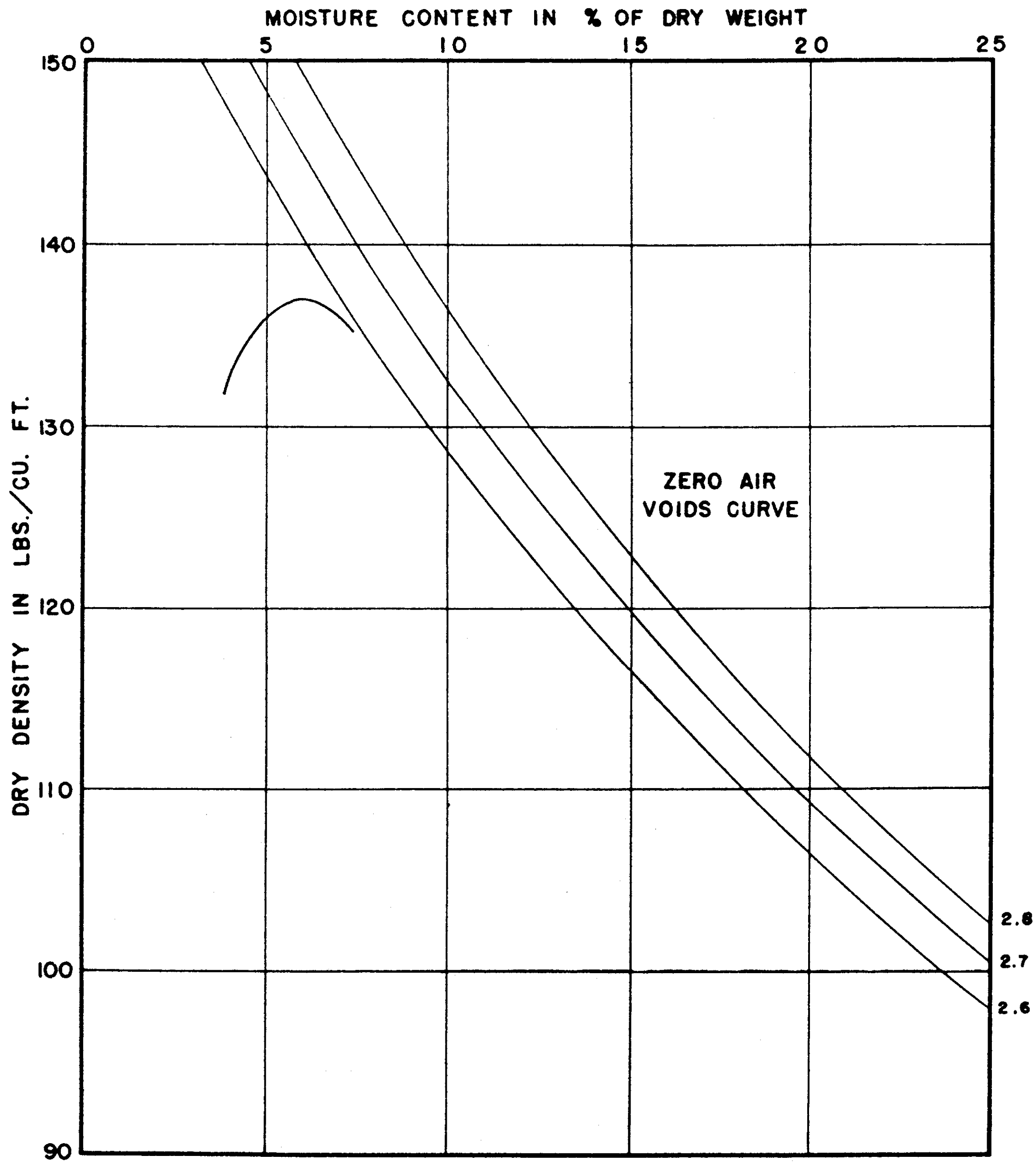
SHELL VALLEY WATERSHED - LEVEL III STUDY

COMPACTION TEST RESULTS

Checked by D. J. Hamel Date 10-22-85 Approved by R. L. Volpe Date 10/22/85	Project No. 3235	Figure No. B-3

SHEET 1 OF 4

SAMPLE NO. T-6/B-1 DEPTH 4.0-8.0 FT. ELEVATION 9283 FT.
SOIL GRAVELLY, SILTY SAND (SM)
LOCATION ADELAIDE LAKE BORROW AREA
OPTIMUM MOISTURE CONTENT 6.0%
MAXIMUM DRY DENSITY 137.0 pcf
METHOD OF COMPACTION ASTM D-1557 METHOD A



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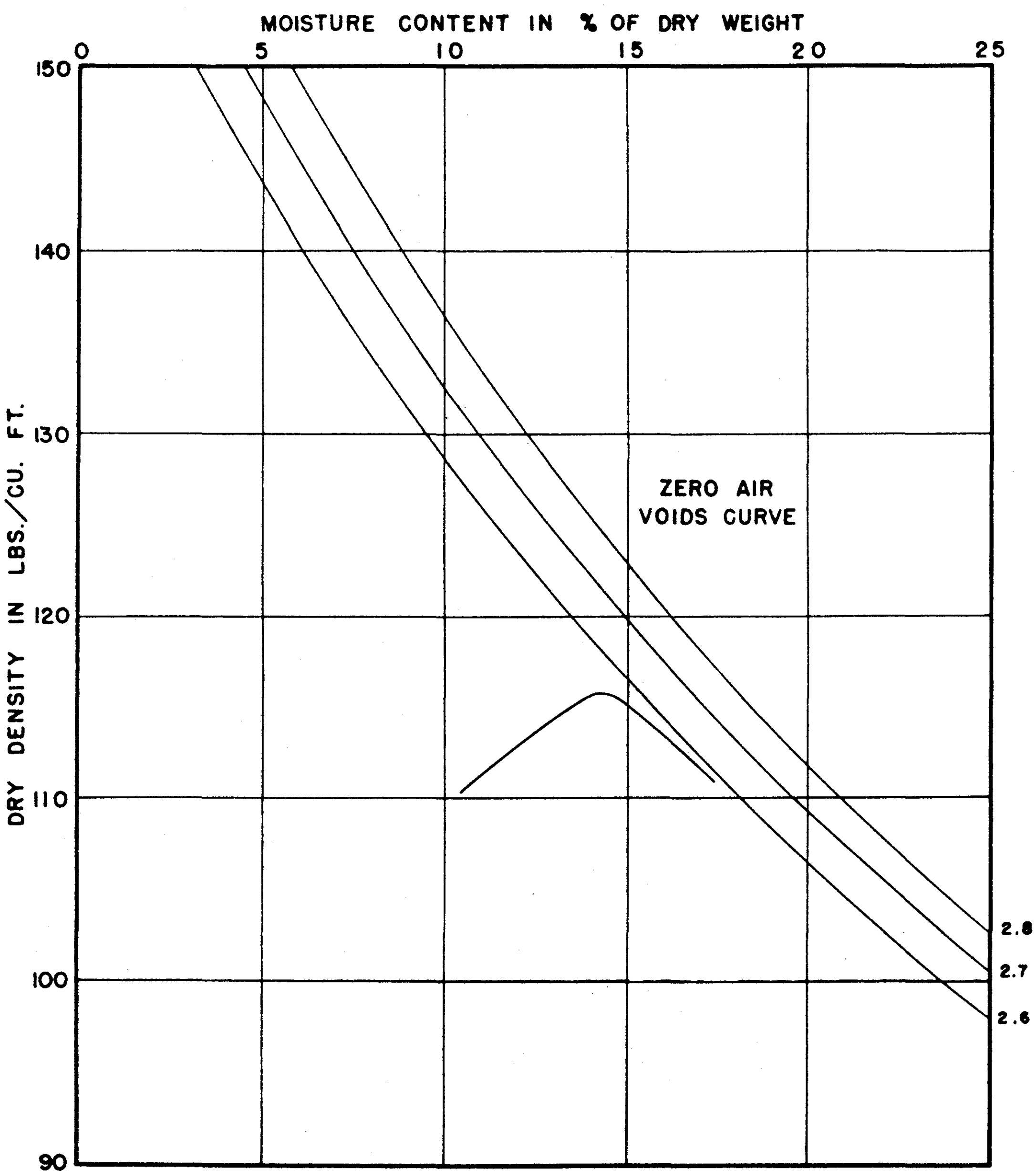
SHELL VALLEY WATERSHED - LEVEL III STUDY
COMPACTION TEST RESULTS

Checked by D. J. Humber Date 10-22-85		Project No.	Figure No.
Approved by R. K. Volpe Date 10/22/85		3235	B-3

SAMPLE NO. T-8/B-1 DEPTH 0.8-2.5 FT. ELEVATION 9273 FT.
SOIL CLAYEY SAND (SC)
LOCATION ADELAIDE LAKE BORROW AREA
OPTIMUM MOISTURE CONTENT 14.5%
MAXIMUM DRY DENSITY 116.0 pcf
METHOD OF COMPACTION ASTM D-698 METHOD C



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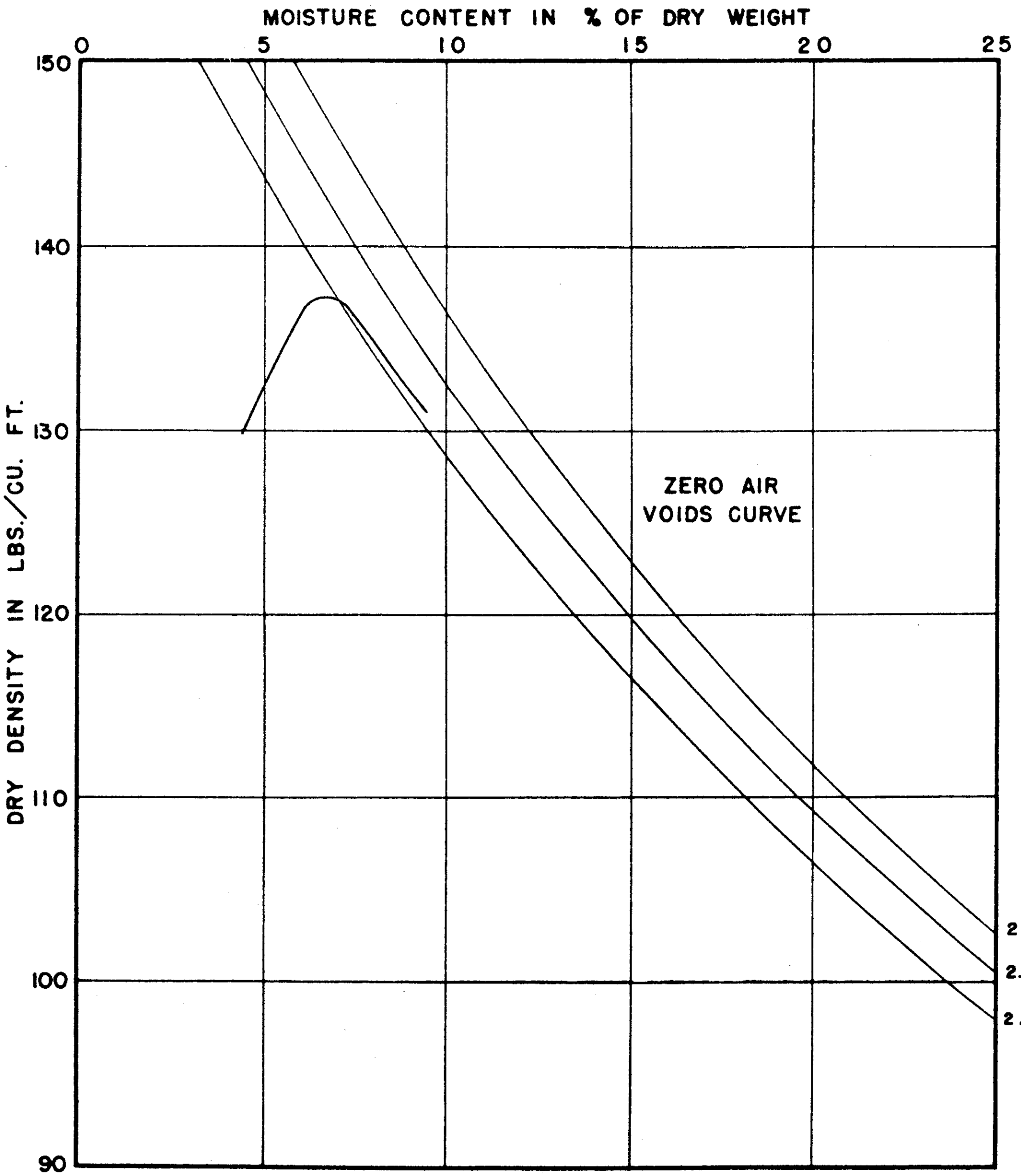
SHELL VALLEY WATERSHED - LEVEL III STUDY
COMPACTION TEST RESULTS

Checked by D. J. Hawke Date 10-22-85	Project No.	Figure No.
Approved by R. A. Volpe Date 10/22/85	3235	B-3

SAMPLE NO. T-15* DEPTH 2.5-9.0 FT. ELEVATION 9273/9256 FT.
SOIL GRAVELLY, SILTY SAND (SM)
LOCATION ADELAIDE LAKE BORROW AREA
OPTIMUM MOISTURE CONTENT 6.7%
MAXIMUM DRY DENSITY 137.0 pcf
METHOD OF COMPACTION ASTM D-698 METHOD C

*SAMPLE COMBINED FROM T-15/B-1, T-15/S-3, T-8/B-2, T-8/S-2



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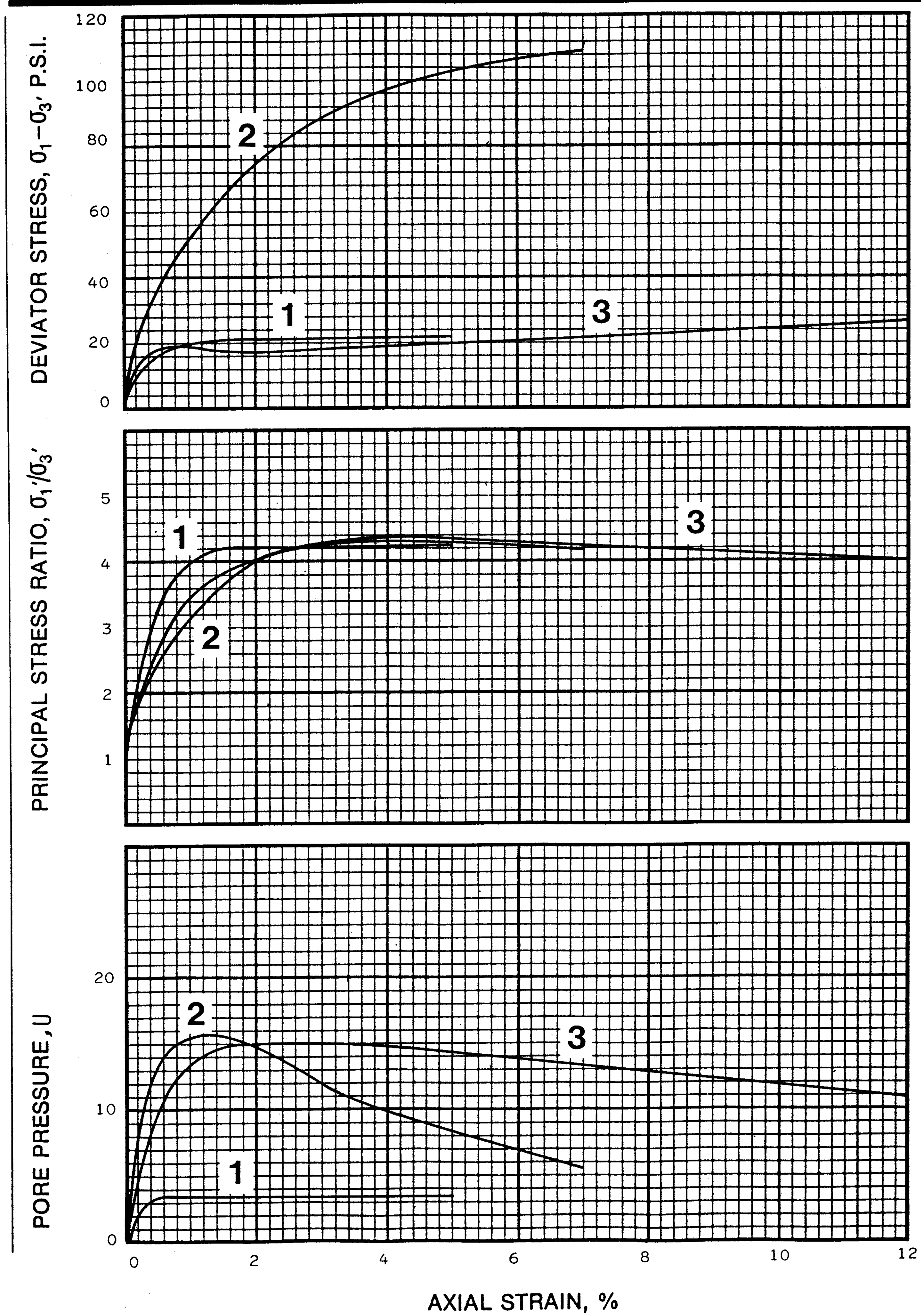
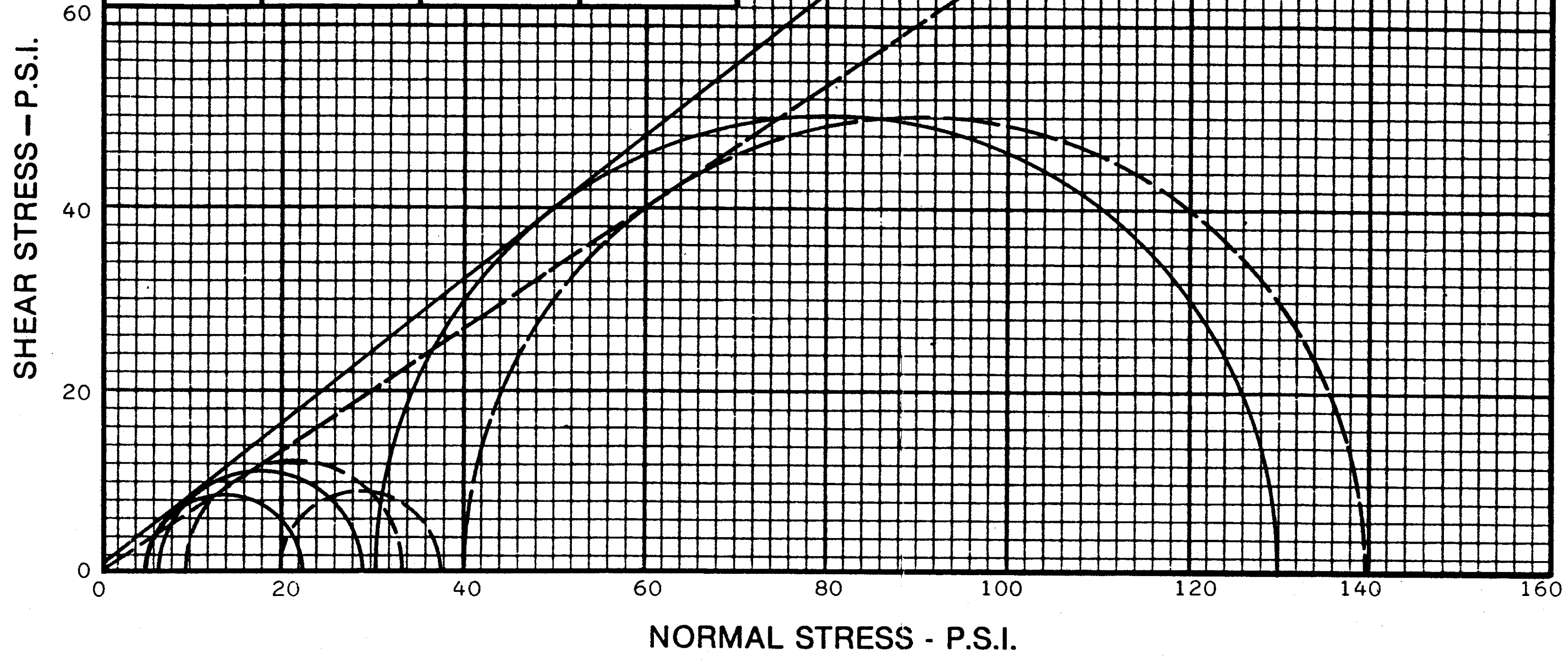
WYOMING WATER DEVELOPMENT COMMISSION SHELL VALLEY WATERSHED - LEVEL III STUDY COMPACTION TEST RESULTS		
Checked by D.J. Hamel Date 10-22-85 Approved by R.L. Volpe Date 10/22/85	Project No. 3235	Figure No. B-3

SHEET 4 OF 4

Approved for publication ____________ by ____________



SHEAR VALUES			
EFFECTIVE STRESSES (———)		TOTAL STRESSES (— — —)	
Ø'= 38.4°	C'= 0.1 P.S.I.	Ø= 33.7°	C= 0 P.S.I.



SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
1 (STAGE 1)	T-5	B-1	2.0-3.0	SM	5.98	2.88	131.6	7.4	FABRICATED
1 (STAGE 2)	T-5	B-1	2.0-3.0	SM	5.75	2.88	136.5	8.3	FABRICATED
2	T-5	B-1	2.0-3.0	SM	5.96	2.88	135.0	7.5	FABRICATED

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS $\sigma_1 - \sigma_3$ (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1 (STAGE 1)	1	10	22.2	3.3	6.7	28.9	STAGED ISOTROPIC
1 (STAGE 2)	2	40	99.9	9.7	30.3	130.2	$\overline{CU}$
2	3	20	17.6	14.8	5.2	22.8	ISOTROPIC $\overline{CU}$

TRIAXIAL COMPRESSION TESTS

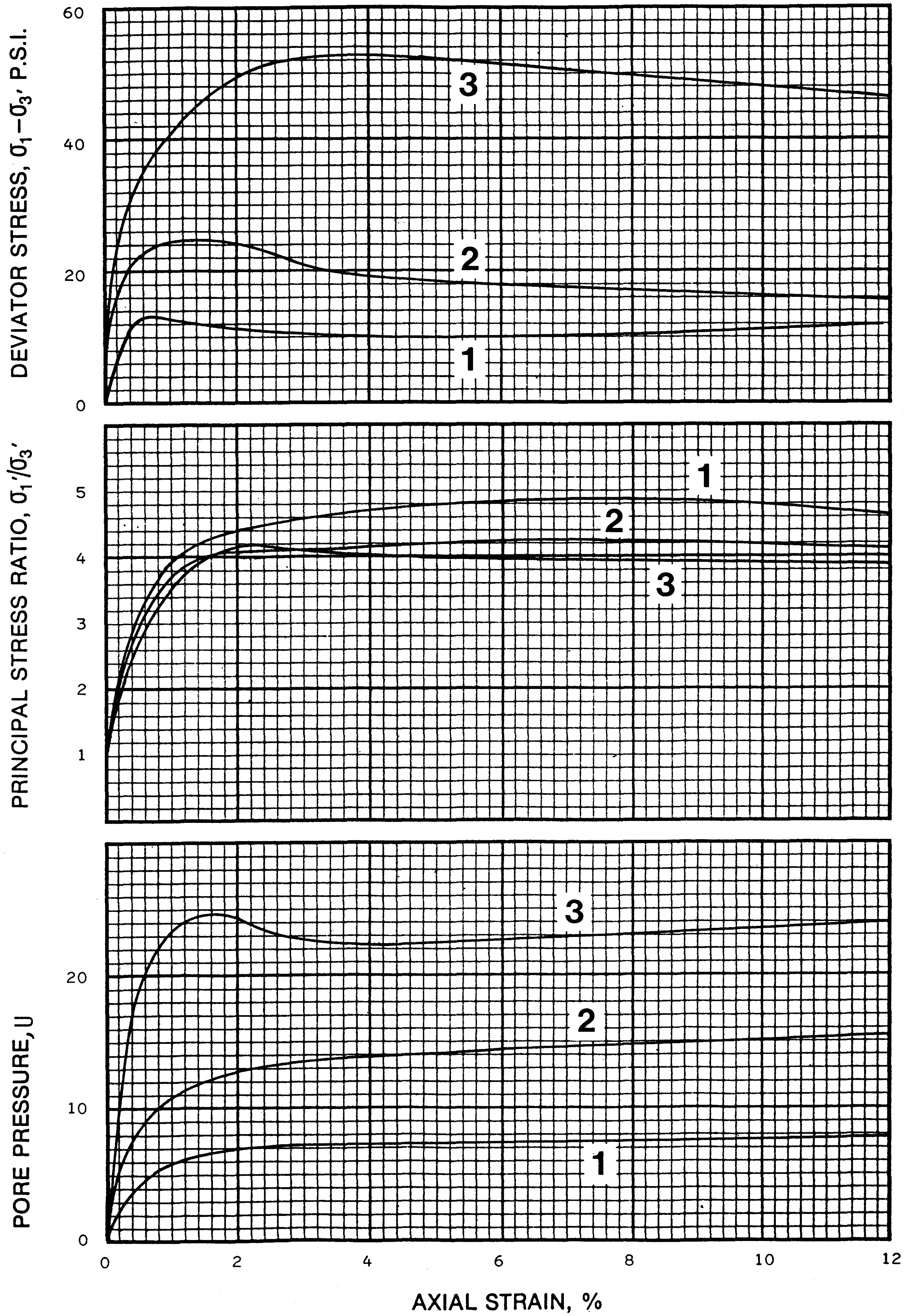
ESA Geotechnical Consultants
Fort Collins, Colorado

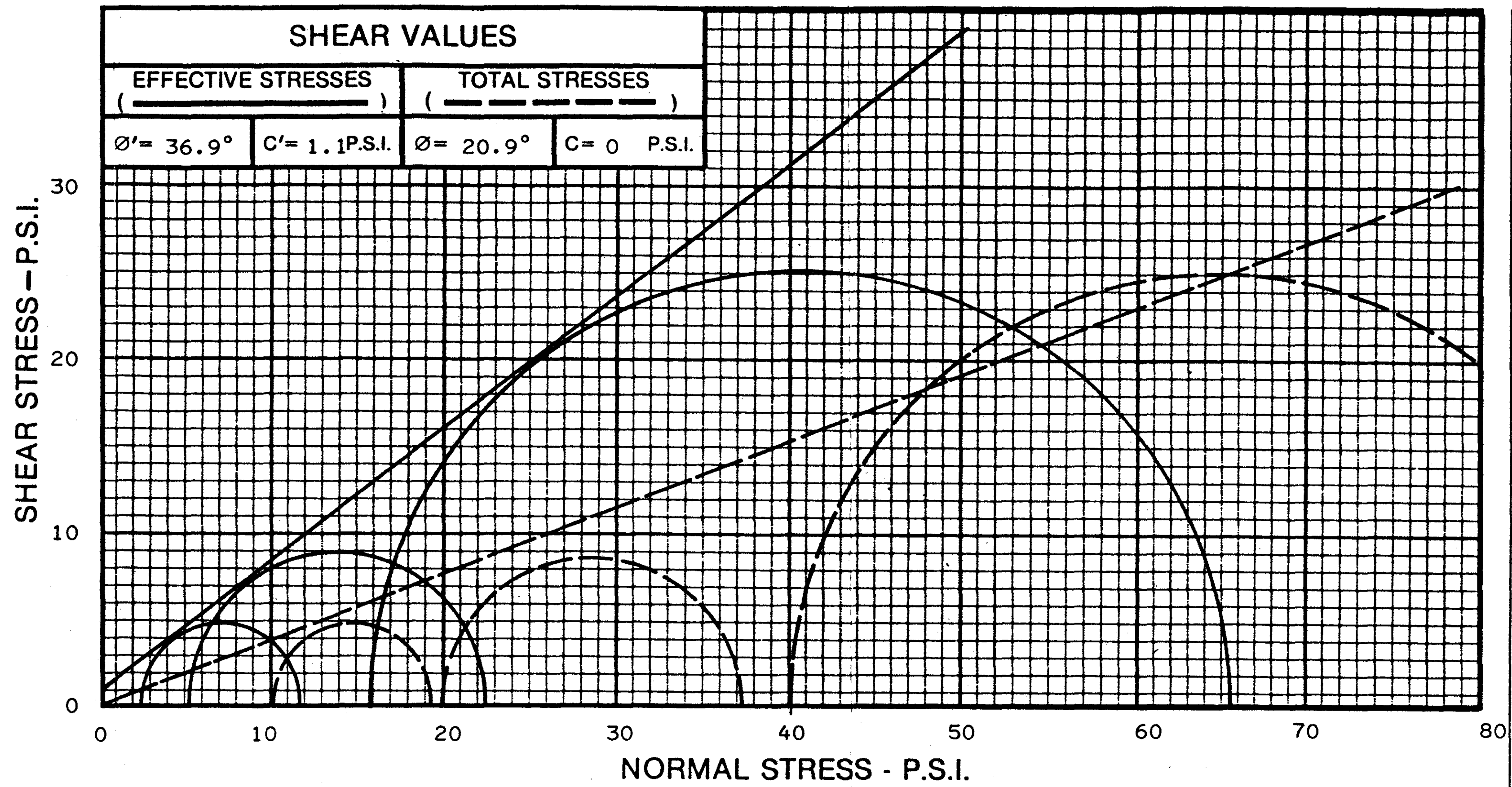
WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
TRIAXIAL SHEAR TEST RESULTS

Project No. 3235

Figure No. B-4
SHEET 1 OF 2

Approved for publication ____________ by ____________





SPECIMEN NUMBER	SPECIMEN LOCATION			INITIAL SPECIMEN DATA					SAMPLE TYPE
	BORING NUMBER	SAMPLE NUMBER	DEPTH (FEET)	SOIL CLASSIFICATION	LENGTH (INCHES)	DIAMETER (INCHES)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (PERCENT)	
1	T-15,T-8	COMPOSITE	2.5-9.0	SM	5.98	2.88	130.0	6.8	FABRICATED
2	T-15,T-8	COMPOSITE	2.5-9.0	SM	5.98	2.88	130.7	6.3	FABRICATED
3	T-15,T-8	COMPOSITE	2.5-9.0	SM	5.99	2.88	129.9	6.5	FABRICATED

SPECIMEN NUMBER	SYMBOL	EFFECTIVE CONSOL. PRESSURE σ_{3C} (P.S.I.)	TEST VALUES AT FAILURE (MAXIMUM σ_1'/σ_3')				TEST TYPE
			TOTAL DEVIATOR STRESS σ_1-σ_3 (P.S.I.)	PORE PRESSURE CHANGE ΔU (P.S.I.)	MINOR EFFECTIVE STRESS σ_3' (P.S.I.)	MAJOR EFFECTIVE STRESS σ_1' (P.S.I.)	
1	1	10	9.3	7.6	2.4	11.7	ISOTROPIC $\overline{CU}$
2	2	20	17.0	14.7	5.3	22.3	ISOTROPIC $\overline{CU}$
3	3	40	49.7	24.2	15.8	65.5	ISOTROPIC $\overline{CU}$

TRIAXIAL COMPRESSION TESTS

ESA Geotechnical Consultants
Fort Collins, Colorado

Project No. 3235

WYOMING WATER DEVELOPMENT COMMISSION
SHELL VALLEY WATERSHED - LEVEL III STUDY
TRIAXIAL SHEAR TEST RESULTS

Figure No. B-4
SHEET 2 OF 2

APPENDIX C

SEISMIC REFRACTION SURVEY

APPENDIX C

SEISMIC REFRACTION SURVEY

A. INTRODUCTION

A total of seven seismic refraction lines with a combined spread length of 3,780 feet were recorded in the vicinity of Lake Adelaide during the week of June 21-26, 1985. The purpose of these lines was to evaluate the depth to and characteristics of various subsurface materials to provide preliminary design information for the proposed expansion of Lake Adelaide. For the purposes of this report, a seismic refraction line consists of 12 geophones spaced at equal intervals of 10 or 30 feet along a straight line and monitored simultaneously while explosive charges are detonated, or while a large sledge hammer is impacted 10 or 15 feet off both ends of each line.

Locations of the seismic refraction lines are shown on Figures III-1 and III-5 of this report. Subsurface layers were distinguished on velocity zones constructed from interpretation of the seismic refraction data. The depths and velocities of these layers are summarized in Table C-1.

B. METHOD AND EQUIPMENT

The seismic refraction survey procedure used for lines 1 through 5 consisted of placing 12 geophones in as straight a line as possible (in plan) spaced at 10-foot intervals along as constant a slope as possible (in profile). A sledge hammer was impacted 10 feet off both ends and in the center of each line to serve as an energy source. The seismic refraction procedure used for lines 6 and 7 consisted of placing 12 geophones spaced at 30-foot intervals along as constant of a slope as possible (in profile) and explosives were detonated at 15 feet off both ends and in the center of each line to serve as an energy source. The explosion or impact produced seismic compression waves were refracted through subsurface materials and received by the geophones. Signals from the energy source initiation and geophones were monitored (amplified and filtered) simultaneously by a 12-channel seismograph, stored in a stacker memory for addition of subsequent signals or further amplification and/or filtering, and displayed graphically in analog form on a digital CRT. Hard copy records, produced by a printer on impace sensitive paper, were catalogued and returned to the office for data reduction and interpretation.

The data reduction and interpretation procedure consisted of visually picking first breaks of compression waves on the hard copy recods, plotting of time-distance graphs, determination of apparent velocities, application of static corrections (shot depth and topography), refinement of apparent velocities, correlation of results with geologic factors and the preparation of interpreted subsurface velocity profiles.

The equipment used for the seismic refraction survey consisted of 12 geophones at one time of 60 HZ natural frequency. The geophones were connected to 10-or 30-foot take-out spacing cables using split take-out connectors. A Bison Model 8012 Geo Pro 12-channel seismograph was used throughout the entire investigation. The energy source was provided by Kinepak, Inc. two-component explosives in 1/3-pound cartridges (ammonium nitrate and nitro methane) detonated with Dupont geophysical blasting caps connected to an SIE model PCD-49R blaster. A 16-pound sledge hammer equipped with a solid state piezoelectric trigger was also used as an energy source in lines 1 through 5.

C. LIMITATIONS

The subsurface velocity profiles presented in this report represent the most reasonable interpretation of seismic refraction survey data based on our knowledge of existing geologic conditions. The results are presented for design information only and are not intended to serve as information for determining construction procedures.

Although in general the seismic refraction data quality for this survey was good, the reliability of data was limited due to irregular terrain and many large boulders scattered at the surface and near surface. These factors invariably produced some scatter in the recorded data, limiting the accuracy of first break compression wave picks.

The seismic refraction method used has some inherent limitations such as the possibility of undetectable hidden layers, blind zones and velocity inversions. Variations in the degree of weathering of the bedrock surface and the proximity to the surface of various horizons may result in possible innacuracies and data scatter. In addition, the impact hammer method may not produce adequate energy to provide sufficient data for making reliable picks under certain conditions.

The maximum depth of reliable seismic information obtained during this survey can be assumed to be approximately one-third of the length of the individual lines, with information at maximum depth underlying the middle one-third of the line. For example, a seismic refraction line 300 feet in length will typically yield reliable data on materials to a depth of about 100 feet beneath the middle 100 feet of the line. The most reliable data available were used to estimate depths reported in Table C-1. These data supplemented drill hole and test pit data and were all used in development of sections in the dam site foundation area and in the borrow area.

TABLE C-1

SUMMARY OF SEISMIC REFRACTION SURVEYS

LINE* NO.	ENERGY SOURCE	LENGTH (feet)	LAYER	DEPTH (feet)	VELOCITY (ft./sec.)	MATERIAL DESCRIPTION
1-1	Hammer	120	A	0-5	900	Loose to Firm Till
			B	5-30	5,150	Dense Till
			C	> 30	19,950	Bedrock
1-2	Hammer	120	A	0-6	1,250	Firm Till
			B	6-33	8,175	Dense Till
			C	> 33	19,900	Bedrock
1-3	Hammer	120	A	0-11	1,170	Firm Till
			B	11-44	8,330	Dense Till
			C	> 44	19,900	Bedrock
2-1	Hammer	120	A	0-10	1,760	Firm Till
			B	10-41	4,325	Dense Till
			C	> 41	19,760	Bedrock
2-2	Hammer	120	A	0-11	1,250	Firm Till
			B	11-43	6,190	Dense Till
			C	> 43	20,000	Bedrock
2-3	Hammer	120	A	0-5	560	Loose to Firm Till
			B	5-38	6,020	Dense Till
			C	> 38	20,000	Bedrock
3-1	Hammer	120	A	0-6	780	Loose to Firm Till
			B	6-40	6,760	Dense Till
			C	> 40	18,030	Bedrock
3-2	Hammer	120	A	0-3	670	Loose to Firm Till
			B	3-47	4,230	Dense Till
			C	> 47	19,990	Bedrock
4-1	Hammer	120	A	0-6	1,400	Firm Till
			B	6-30	4,260	Dense Till
			C	> 30	19,950	Bedrock
4-2	Hammer	120	A	0-10	1,550	Firm Till
			B	> 10	20,000	Bedrock
4-3	Hammer	120	A	0-10	1,380	Firm Till
			B	> 10	20,000	Bedrock
5-1	Hammer	120	A	0-14	1,500	Firm Till
			B	> 14	20,000	Bedrock
5-2	Hammer	120	A	0-5	630	Loose to Firm Till
			B	5-42	6,265	Dense Till
			C	> 43	19,990	Bedrock
5-3	Hammer	120	A	0-4	690	Loose to Firm Till
			B	4-46	7,340	Dense Till
			C	> 46	19,425	Bedrock
5-4	Hammer	120	A	0-9	1,320	Firm Till
			B	9-29	7,385	Dense Till
			C	> 29	19,860	Bedrock
7-1	Explosives	390	A	0-29	3,570	Dense Till
			B	> 29	19,420	Bedrock
7-2	Explosives	390	A	0-39	1,340	Firm Till
			B	> 39	19,290	Bedrock
7-3	Explosives	390	A	0-33	3,460	Dense Till
			B	> 33	18,720	Bedrock

> is used to denote "greater than".

*Lines not listed (1-4, 1-5, and 6) had unreliable results.