GOOSEBERRY LEVEL III PROJECT
PHASE II
SUMMARY REPORT

February 1987

Submitted to

Wyoming Water Development Commission

by

Morrison-Knudsen Engineers, Inc.
1120 Lincoln Street, Suite 1200
Denver, Colorado 80203
# GOOSEBERRY LEVEL III PROJECT

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SUMMARY REPORT

PHASE II
GOOSEBERRY LEVEL III PROJECT

I. INTRODUCTION

The Gooseberry Level III Project, authorized by the 1984 Session of the Wyoming Legislature, was analyzed in several stages. The original schedule called for conducting the study in two phases. The first phase was to evaluate the geotechnical conditions at the site and determine the feasibility of construction, since the Level II study had contained no provision for geotechnical evaluation.

The second phase was contingent upon the findings of the first phase and was to develop designs and specifications for the dam construction, if found feasible. In actuality, denial of access to the site and opposition to the project resulted in a year's delay and a curtailment of the time-frame allotted to geotechnical exploration.

The first phase was subsequently divided into two parts, with the first covering the hydrology and economics, and the second the initial geotechnical investigation. The results of these efforts were described in reports dated February 1985 and January 1986. Based upon those studies, the project was approved for Phase II activities.

PURPOSE OF REPORT

This report has been prepared to describe the activities undertaken during the Phase II effort and the results of the analyses made. It documents the findings of the 1986 field exploration and the preliminary design effort. Based upon the design selected, cost estimates were developed and provided the Wyoming Water Development Commission for review.

SUMMARY OF RESULTS

The geotechnical exploration conducted during Phase I had been limited because
of the time of access dictated by a court order and the amount of allocated funding. The Phase II program revealed additional amounts of unsuitable foundation material under the damsite and significant amounts of colluvium covering the rock at the emergency spillway location.

Additional exploratory holes were drilled in the foundation area of the damsite. Several test pits and trenches were excavated in the foundation and at the location of the emergency spillway in an attempt to better define the overburden deposits and location of the bedrock. Test pits were also dug throughout the potential borrow areas to determine their extent and the characteristics of the material. Laboratory analyses were conducted on several samples taken from the borrow areas, as well as the rock core from the bore holes.

The geotechnical investigation indicated two areas of concern relative to the dam construction. One was the low density and strength of silt and sandy silt lenses in the foundation area. Another was the stability of and potential seepage through the abutments. Modification of the feasibility design was undertaken in order to develop a design which would provide for the safety and integrity of the structure.

A design flood analysis was made to estimate the spillway capacity required. Four flood levels were routed through the reservoir and a 22-mile downstream reach of Gooseberry Creek. Reservoir volume was varied along with spillway capacity. A dam breach condition was also simulated for each of the four scenarios. The results, although not reviewed and approved by the Wyoming State Engineer, indicate the required spillway design capacity must pass a flood having a magnitude of one-fourth the probable maximum flood.

The cost estimates, previously developed for this site, were updated using the information developed during the Phase II analysis. Items associated with the dam construction were also estimated, and a preliminary total cost was provided the WWDC. This total exceeded the quantity estimated during the Phase I analysis by a significant amount. Because of the potential for increased costs and the marginal feasibility of the project, the WWDC elected to terminate further design efforts on the project. The estimated project
construction costs and design parameters are discussed in following sections of this report.
II. RESULTS OF 1986 FIELD INVESTIGATIONS

The 1986 field exploration program was an extension of that conducted the previous year. The major objectives were to better define the foundation characteristics at the damsite and to locate materials suitable for construction of the dam. Associated with the location of these materials was an extensive laboratory testing program to determine the characteristics of the various materials.

The Upper Gooseberry site is located at the head of a canyon formed at the contact between Cody Shale and the overlying Mesa Verde sandstone. Both formations are of Cretaceous age. The canyon is formed where the creek passes from the easily erodible shale formation upstream, characterized by wide meanders, to the more resistant sandstone units where lateral movement is restricted. Where the shale weathers and erodes in a uniform pattern producing a relatively gentle topography, the sandstone erodes in blocks that are loosened by stress relief toward the valley and form steep cliffs. This is especially true on the left side of the valley, which is a southern exposure characterized by about 80% rock outcrops. The right side or northern exposure is characterized by a near vertical cliff of outcropping sandstone at the base of the abutment above which weathering is deeper, as evidenced by gentler slopes and greater vegetative cover. The contact between the Cody Shale and Mesa Verde sandstone is transitional over hundreds of feet, with sandstone and shale/claystone interbedded. The number of sandstone interbeds increases upward through the section, as the occurrence of claystone decreases. The sandstone layers form resistant ridges within outcrops of this unit.

DAM FOUNDATION

Type of Exploration
A surficial geologic map was prepared of the area encompassing the dam foundation and spillway. The map is on Drawing 84-595-2 and is included in Appendix G to this report. The map shows the contacts between the principal geologic units identified at the site and the attitude (strike and dip) of
those units. The map also indicates the attitude of the principal sets of joints in the rock mass.

A joint survey, consisting of a statistical analysis of the rock joints measured in the field, was compiled. The results of that survey are expressed as a contour density plot on a stereographic projection of the poles of the principal joints on Drawing 84-595-2.

Subsurface investigation in 1986 consisted of 18 additional bore holes, 10 test pits, three trenches in the vicinity of the emergency spillway, and 4550 linear feet of seismic refraction lines. The locations of these explorations are shown on Figure 1. Logs and photographs of the bore holes, test pits, and trenches are included in Appendix G. The total footage of drill core, including the holes drilled in 1985, was 1550 feet.

The bore holes were generally advanced through the soil overburden by means of a 6-inch diameter hollow stem auger or cased in conjunction with a tricone rotary bit. Standard penetration tests were performed, and disturbed soil samples were obtained at five-foot intervals. Undisturbed samples were obtained with Shelby tubes and ring samplers. Constant head permeability tests were carried out periodically in the soil portion of the boreholes. Bedrock was cored with an NQ diameter diamond bit, attached to a wire line equipped double barrel sampler. Upon completion of the bore holes the bedrock was water pressure tested utilizing a double packer at 13-foot intervals. All bore holes were logged and the rock core classified geomechanically using the rock quality designation (RQD) method.

Test pits and trenches were excavated with a backhoe in order to allow visual examination of the subsurface strata and to obtain samples for laboratory testing.

Seismic refraction surveys were accomplished using a 12-channel seismograph. Shock waves were initiated by small explosive charges.
FIGURE 1

LEGEND
- Test Pit
- Bore Hole
- Auger Hole
- Seismic Reflection Survey Line
- Shot Point

UPPER GOOSEBERRY DAM
GEOLOGIC INVESTIGATIONS

GOOSEBERRY LEVEL III PROJECT

WYOMING WATER DEVELOPMENT COMMISSION
Results
Surficial geologic mapping focused on accurately locating the contacts between the principal bedrock units exposed in the abutments. The strike of the Mesa Verde Formation at the site is N10°W and the dip is about seven degrees northeast. The principal bedrock units are consistent over the project area as shown on the map and geologic section. The oldest unit mapped is the Cody Shale/Mesa Verde transition unit (Kmi), and consists of thinly bedded claystone/shale and sandstone. Exposures are poor at the axis but good outcrops are visible just upstream of the dam. The base of the Mesa Verde Formation is clearly defined by a massive buff sandstone member (Kmb) about 40 feet thick which forms steep outcrops at the base of each abutment. The rock is generally sound to slightly weathered at the surface, with outcrops showing signs of calcium carbonate case hardening. Joints are widely spaced and consist mainly of closed bedding plane joints, and open joints of tectonic origin with attitudes of N15°W, 70°-80°SW and N36°E, 65°-70°NW, and open stress relief joints parallel to the weathering surface.

The massive buff sandstone is overlain by a sequence of thinly bedded shale and sandstone (Kmc) about 20 feet thick, containing one or more coal seams near the base ranging to 12-inches in thickness. The unit is weaker and more highly weathered than the sandstone units above and below it. Due to the importance of this relatively weaker rock unit to the stability and seepage characteristics of the foundation, its upper and lower contacts were mapped in great detail, aided by several surveyed reference points. The lower contact of this unit, with the underlying buff sandstone, occurs at about elevation 6320 on the left abutment just above the crest of the dam. Good exposures of the shaley zone, containing coal, are visible near the creek on the right abutment downstream of the dam. Moving upstream toward the dam axis, the contact becomes buried by colluvial and alluvial terrace deposits. Structural contours of the upper and lower contacts were developed based on the surficial mapping and bore hole data in order to project this important unit into the proposed dam foundation.

The youngest unit of the Mesa Verde formation mapped was the resistant white massive sandstone (Kmw) unit that forms good outcrops on both abutments. The white sandstone unit is about 80 feet thick. It showed the same physical
characteristics as the tan unit described previously with regard to weathering and jointing.

The 1986 subsurface investigation detected the same soil and bedrock units as identified during the 1985 investigation, but provided more detailed information as to the lateral extent and thickness of the alluvial and colluvial deposits in the valley floor, as well as the quality of the bedrock underlying the alluvium and on the abutments.

A few of the bore holes in the 1985 investigation had detected the occurrence of a low density alluvial silt and silty sand underlying surficial sand and gravel deposits and overlying a denser layer of silty gravel with sand in contact with the bedrock. Test pits TP-43 through TP-47, auger holes AH-25 and AH-34 through AH-39, and drill holes DH-26 and DH-27, all installed during 1986, were located in the floodplain. Seismic refraction lines were run along the proposed dam centerline. The investigation revealed an upper layer of recent stream alluvium composed of loosely consolidated, alternating lenses of brown silty sand, sand, and poorly sorted sandy gravel varying in thickness from zero to 16 feet. Underlying the surficial deposits in nearly every bore hole and test pit was from one to 32 feet of loosely consolidated gray, silty sand and clayey sand with low SPT blow counts, ranging from 2 to 16. The underlying alluvium in contact with bedrock in each of the bore holes was dense silty gravel with sand. Bedrock underlying the floodplain was claystone with sandstone interbeds of the transition unit between the older Cody Shale and younger Mesa Verde formation. The claystone was dark gray-to-brown, thinly bedded and moderately jointed. Permeability ranged from $2 \times 10^{-3}$ to $8 \times 10^{-4}$ cm/sec.

Bore holes DH-23 and DH-24A were angle drilled into the left abutment bedrock. Borehole DH-24A, at the base of the abutment, penetrated 23 feet of sandy clay colluvium before contacting light brown to gray, sound, thinly interbedded sandstone and claystone of the lower Mesa Verde transition sequence. Core recovery was excellent, and RQD was good to 42 feet and excellent below that point to the bottom of the hole at 52.0 feet. The packer test in the upper ten feet of bedrock resulted in a coefficient of permeability of $9.0 \times 10^{-4}$ cm/sec. Bore hole DH-23 was angle drilled on the
proposed dam axis at approximately the crest elevation into an outcrop of massive, sound, tan sandstone. Near 100 percent core recovery was logged to the final depth of 105 feet. Jointing was very widely spaced and RQD approached 100 percent. Permeabilities calculated from water pressure tests ranged from $1.0 \times 10^{-3}$ cm/sec near the surface to less than $1.0 \times 10^{-5}$ cm/sec with depth.

One bore hole, DH-30, and two test pits, TP-48 and TP-49, were located on the right abutment terrace deposit between the vertical cliff at the base of the abutment and the spillway. They showed a maximum depth of 18 feet of silty sand and sandy clay colluvium overlying up to eight feet of poorly-graded, well-rounded sand and gravel of alluvial origin. The bedrock in hole DH-30 was massive buff (tan to light brown) sandstone with occasional claystone interbeds. The rock was slightly weathered and moderately fractured as evidenced by the core recovery that ranged from 11 to 93 percent and RQD from 0 to 64 percent. Packer test permeabilities were uniformly $5.0 \times 10^{-4}$ cm/sec. No sign of the interbedded sandstone/shale and coal sequence was detected.

**Geologic Section**

A geologic section along the centerline of the proposed dam, based on geologic mapping, seismic refraction, and bore hole and test pit data is shown on Figure 2. At the dam axis the alluvial flood plain is 450 feet wide. The Quaternary alluvium overlies bedrock to a depth of 52 feet and is composed of silt, silty sand, and gravel. The recent alluvial flood plain is overlain on the left abutment by a Quaternary alluvial/colluvial fan approximately 20 feet thick. This deposit is comprised of silt, silty sand and angular blocks of sandstone debris derived from the adjacent sandstone cliffs and the narrow draws that feed at right angles into Gooseberry Creek just upstream and downstream of the axis.

A Quaternary alluvial and colluvial terrace occurs at approximately elevation 6360 on the right abutment just above the steep bedrock outcrops. The deposit consists of about 13 feet of colluvial silty sand, overlying approximately 6 feet of alluvial sand and gravel. The gravel is characterized by sound, well rounded cobbles of metamorphic and igneous origin, up to 12 inches in diameter.
Bedrock stratigraphy at the axis starts in the foundation below the alluvium in the transition zone between the Cody Shale and Mesa Verde sandstone (Kmi). The rock mass is a complex interbedded sequence of sandstone and claystone or shale. The upper limit of this transition zone is marked by a massive buff sandstone (Kmb) unit at approximately elevation 6370 on the left abutment and 6310 on the right. The dam foundation below the contact, including the valley floor underlying the alluvium, is in the transition zone. The buff sandstone unit is about 40 feet thick forming the steep cliffs at the base of each abutment. The resistant sandstone unit is overlain by a 20-foot thick sequence of shale, thinly bedded sandstone and sinuous veins of coal to one foot thick (Kmc). At the dam axis the coal seams are located near the base of the unit. Because of the weak rocks that comprise this unit, weathering is more intense and has produced gentler slopes. On the right abutment, at the axis, the coal/shale sequence is completely buried. The weak nature of this unit and subsequent erosion by ancient Gooseberry Creek may have caused the bench in the bedrock of the right abutment that is covered by the alluvial/colluvial terrace deposits. The youngest stratigraphic unit at the damsite that enters into the geotechnical analysis of the foundation is the massive white sandstone (Kmw) that outcrops on both abutments. It has a thickness of approximately 80 feet.

**SPILLWAY LOCATION**

**Subsurface Investigation**

Boreholes DH-31 and DH-32 were drilled along the centerline of the proposed unlined side channel emergency spillway alignment on the right abutment. Neither bore hole, however, detected sound bedrock at a high enough elevation (6406 at the control section invert) to support the original design. If the same alignment were to be maintained, the spillway design would have to be changed to include a large volume of backfill concrete, a reinforced concrete slab and structural concrete chute walls. Because of the economic impact of such a change in design, the focus of the investigation was moved further up the abutment where outcrops of white massive sandstone were mapped. One bore hole, DH-33, was drilled to a depth of 68.7 feet, approximately equivalent to the invert elevation of the spillway channel. The rock core was massive,
fine-grained, tan and gray sandstone, weak to moderately cemented with widely spaced joints, but weak and friable, especially when wet.

Three trenches were excavated perpendicular to the alignment of the spillway in order to verify the top of sound rock over the entire area of the spillway approach channel and chute, and to visually assess the quality of the rock. Logs of the trenches are included in Appendix G. Mapping of the trenches generally showed from one to three feet of colluvial debris composed of thin slabs of white sandstone in a matrix of dark silty sand. The top few feet of the sandstone bedrock were weathered to a white sand, below which the rock mass was more resistant with a high density of stress relief joints parallel to the valley decreasing in frequency with depth. Sound, lightly jointed sandstone, impenetrable to the backhoe, was generally reached at a depth of about eight feet in all the trenches. At the downhill end of the trench near the original spillway alignment the top of sound rock dropped off abruptly to a depth in excess of 14 feet, which was the limit of the backhoe.

**Geotechnical Analysis**

Based on the geologic mapping and the bore hole and trenching information, it appears feasible to excavate a side channel spillway double cut in rock on either abutment. The right abutment cut would be about 400 feet long with a maximum depth of about 50 feet. The left abutment channel would be about 200 feet long but with a maximum depth of about 80 feet. The total volume of excavation appears to be roughly equal. Excavation slopes would be approximately 1H to 1V in the overburden colluvium and weathered rock and vertical 30-foot high cuts with 10-foot benches in sound rock.

The invert and side walls of the right abutment channel would intercept the coal seam and shale zone discussed in the section on geologic mapping, and may require some treatment with dental concrete, shotcrete, and rock bolts. The invert of the left side would be in sound massive sandstone.

The 1986 revised cost estimate was based on quantities taken off a layout on the right abutment. The ultimate decision on spillway location, however, would hinge on the utilization of the material derived from the excavation. The right abutment excavation would produce colluvium and weathered, fractured
sandstone in the upper ten feet. Below that, the cut would be in massive, sound gray (white) and tan (buff) sandstone overlying about a 10-foot zone of shale and coal near the channel invert. Core from borehole DH-33 showed the rock to be massive but poorly cemented and friable. Its use as riprap, therefore, is somewhat in question. The bulk of the excavation for the left abutment would be in the shaley/coal seam zone above elevation 6425. The lower part of the excavation, however, would be in the tan (buff) sandstone that drill hole DH-23 showed to be the soundest on the project and the best local source for riprap.

MATERIALS EXPLORATION

Purpose
The purpose of the materials exploration program was to locate and define potential borrow areas, and to obtain samples for laboratory testing. Soils suitable for impervious core, shell, filter, and drain zones of the dam were sought in the vicinity of the reservoir area.

Following a review of the 1985 geotechnical investigation results and local geology, a preliminary exploration plan was developed. The majority of the program was to be implemented using backhoe excavated test pits, augmented by truck-mounted auger drill holes where necessary. Materials were to be logged and photographed in the field, with disturbed samples taken for laboratory testing.

Prior to initiating the exploration program, surficial geology was examined with regard to the probable locations of potential construction materials. Generally, it was anticipated that soils suitable for impervious core could be found in areas of weathered in-place Cody Shale, or colluvium of intermixed Cody Shale and Mesa Verde Sandstone. Alluvial deposits were to be investigated for potential shell, filter, and drain materials. A sufficient number of test pits would be excavated in order to obtain representative coverage of the areas in question and define the approximate boundaries of those areas.
Core Materials Exploration

During the week of July 28, 1986, two rubber-tired backhoes were brought on-site for the MKE-supervised investigation program. The search for soils suitable for the core of the dam began on the north side of the proposed reservoir, immediately below exposed outcrops of the Cody Shale formation. Test pits in this area revealed colluvium comprised of silty sands with gravel and clay lenses, to a depth of approximately 12 feet, under which were alluvial gravels. The colluvial soils were highly stratified and heterogeneous, with low percentages of clay and silt. The deposits represented soils derived from the Cody Shale and Mesa Verde Sandstone cliffs above the area. These soils were considered unsuitable for use as core material. A visual inspection of the surficial deposits along the north side of the reservoir was undertaken, with emphasis on locating residual or transported materials, primarily of Cody Shale origin. No significant deposits of this type were located, except at the base of the cliffs in the Cody Shale formation itself. The material was in a relatively thin unit and practically inaccessible, so the focus of the investigation switched to the south side of the reservoir area.

One area near the west boundary of Section 36 revealed highly-weathered Cody Shale outcropping on sloping terrain near the tree line. The exploration program on the south side of the reservoir focused on this general area. Eleven test pits were excavated, logged, sampled, and photographed. The basic soil type encountered is classified as a lean clay of low plasticity (CL), using the Unified Soil Classification System (USCS). This material was considered a potential source for the dam core. Generally, the soils were either residual or slightly transported shales. Physical characteristics, such as particle size distribution, plasticity, and in-situ moisture content were fairly uniform. However, it was apparent the deposit was limited to a small area as several test pits revealed dissimilar materials (silty and clayey sands) or apparent boundaries between the geologic formations forming the shales and sandstones. It was decided more exploration would be necessary to determine the approximate limits of the proposed borrow area. Samples gathered were delivered to Northern Engineering and Testing, Inc. (NET) in Billings, Montana for laboratory testing.
Nine additional test pits were excavated in or near the proposed core borrow area. Additional samples were obtained and sufficient information gained for a reasonable estimate of available materials quantities.

The surface area of the proposed borrow was determined to be approximately 21.5 acres. The majority of the test pits were a minimum of 12 feet deep, the effective reach of the backhoes, and did not reach the full depth of the deposit. A conservative depth estimate of 10 feet of available material would produce a total core borrow area volume of 348,000 yd$^3$.

Test pit locations and approximate borrow area limits are shown on Figure 3. Test Pit logs and laboratory test results are included in Appendix G.

**Alluvial Materials Exploration**

The 1985 field investigation included five test pits and seven auger holes in the floodplain alluvium, upstream of the damsite. The results of the investigation showed the materials were variable, with considerable silty sand overburden. The 1986 exploration program was designed to provide more information on the alluvial deposits with regard to potential construction material uses and available quantities.

A total of 13 test pits were excavated in the alluvium from approximately 500 to 6,000 feet upstream of the proposed dam axis. The locations were selected to provide full coverage of the area in conjunction with the 1985 investigation. The test pits were logged, sampled, and photographed by MKE personnel. Occasional in-situ moisture density readings were taken.

The alluvial materials consisted of a mixture of geologically young sedimentary rocks from the local formations, and older metamorphic and igneous rocks from formations well upstream of the site. The sedimentary rocks were generally angular and sub-angular with a high percentage of weak and friable particles. The metamorphic and igneous rocks were generally sub-rounded, strong, and durable.

Test pits located near the stream in recent meanders contained little or no silty sand overburden. The meandering effects of the river were evident by
HEREBY CERTIFY THAT THE PROPOSED DAM LAYOUT AND CHARACTERISTICS SHOWN WERE MADE UNDER MY DIRECTION DURING OCTOBER-DECEMBER 1980 AND CORRECTLY REPRESENT THE PROPOSED DAM LOCATION AND CAPACITY, AS DESCRIBED IN THE ACCOMPANYING APPLICATION.

STATE OF WYOMING
COUNTY OF PARK
I, RALPH E. WASHINGTON OF DENVER, COLORADO, HEREBY CERTIFY THAT THE PROPOSED DAM LAYOUT AND CHARACTERISTICS SHOWN WERE MADE UNDER MY DIRECTION DURING OCTOBER-DECEMBER 1980 AND CORRECTLY REPRESENT THE PROPOSED DAM LOCATION AND CAPACITY, AS DESCRIBED IN THE ACCOMPANYING APPLICATION.

STATE OF WYOMING
COUNTY OF PARK
I, RALPH E. WASHINGTON, AMERICAN WATER DEVELOPMENT CORPORATION, HEREBY CERTIFY THAT THE PROPOSED DAM LAYOUT AND CHARACTERISTICS SHOWN WERE MADE UNDER MY DIRECTION DURING OCTOBER-DECEMBER 1980 AND CORRECTLY REPRESENT THE PROPOSED DAM LOCATION AND CAPACITY, AS DESCRIBED IN THE ACCOMPANYING APPLICATION.

TEST PT LOCATIONS

FIGURE 3
the gradational stratifications in the test pits. The materials were
generally classified as clean to silty, poorly-graded gravels (GP-GM).

Test pits located on the low terraces away from the stream contained two to
six feet of silty sands overlying the alluvial gravels. These soils were
considered detrimental to the alluvial borrow as a source of shell zone
materials, because the presence of the silty sands would decrease both
strength and permeability of the shell zone. However, the quantity of
overburden made stripping and wasting impractical, so the decision was made to
consider the borrow area as a whole. As a result, full-face composite samples
were obtained from all test pits to reflect the variations in materials that
could be expected when the entire alluvial borrow area was excavated. The
composite samples were generally classified as poorly-graded silty sand with
gravel (SM-GM).

Generally, the alluvial materials would be suitable for the dam shell zones.
The durability and strength of the materials make them doubtful for use as
concrete aggregates, and questionable for use as drain and filter zones in the
dam.

Ground water was encountered in all the alluvial borrow area test pits.
Average depth to the water table was approximately eight feet. Using a depth
of eight feet to calculate the readily accessible alluvial materials, the
total available volume was 990,000 yd$^3$.

These test pit locations are also shown on Figure 3.

LABORATORY ANALYSIS

Purpose
Soil and rock core samples, obtained during the field exploration program,
were sent to NET for laboratory testing. The purpose of the laboratory
analyses was to classify the various site materials using physical property
tests, and determine the range of engineering properties for shear strength,
permeability, compressibility, and compaction characteristics of the
predominant construction and dam foundation materials.
NET and MKE jointly developed a laboratory testing program that would accomplish this goal. However, the full schedule of laboratory testing was not completed due to termination of the project. Complete details of the laboratory testing program are contained in NET's Report of Geotechnical Investigation, included in Appendix G. The intent of the discussion which follows is to highlight specific test results and material properties as they relate to the dam and foundation design.

The laboratory program focused on testing samples from four specific locations at and near the proposed damsite. These four were:

1. The impervious core material borrow located on the south side of the proposed reservoir, approximately three quarters of a mile upstream of the proposed dam axis.

2. The alluvial material borrow area located in the proposed reservoir.

3. The alluvial and colluvial soils located within the "footprint" of the proposed dam.

4. The various rock units in the foundation beneath the proposed dam.

Results

Physical property testing of the potential impervious core borrow material classified the soil as a lean clay (CL). The materials sampled exhibited plasticity indices ranging from 6 to 15 percent, with an average of 10. The samples typically contained approximately 20 percent sand and 80 percent fines, of which about 35 to 40 percent were clay-sized particles. A remolded permeability test on the potential core material resulted in a permeability value of $1.6 \times 10^{-6}$ cm/sec. A consolidated-undrained triaxial compression shear test performed on the sample which exhibited the highest plasticity index produced an effective angle of internal friction ($\phi$) of $32^\circ$, with no effective cohesion ($c'$). Based on laboratory test results for a limited
number of samples, it appears the lean clay material would provide a strong and impervious embankment dam core zone.

The deposits in the proposed reservoir alluvial borrow area were found to be variable, both in depth and areal extent. Gradation analyses performed on samples of the deposits near the existing creek channel revealed the material to be a clean-to-silty, poorly-graded gravel (GP-GM). Away from the creek to the north, test pits excavated in old river terraces encountered silty sand (SM) layers, up to 6.2 feet thick, overlying the clean-to-silty, poorly-graded gravel. Occasional lenses of finer-grained sandy silt (ML) were encountered. Laboratory tests classified composite (mixed-face) samples from these test pits as poorly-graded silty sand with gravel (SM-GM). Shear strength and permeability tests were planned for these materials, but subsequently cancelled. Using past experience on the behavior of these typical alluvial soils with similar particle size distributions, the material was judged to possess a high effective angle of internal friction, suitable for use as supporting shell zones in the proposed dam.

The overburden soils beneath the "footprint" of the proposed embankment consist of extremely variable, poorly-consolidated alluvial and colluvial deposits. Alluvial soils encountered ranged from fat clays (CH) to clean, poorly-graded gravels (GP). Samples of colluvial material from the abutments and higher elevations of the floodplain were classified as silty sand with gravel (SM). A consolidated-undrained triaxial compression shear test conducted on the colluvium, indicated shear strength parameters of \( \phi' = 33^\circ \), with \( c' = 500 \) psf. Even though the sample contained approximately 20 percent gravel and 40 percent sand, it had about 40 percent silt fines which appear to have governed the shear strength. The only other laboratory analyses completed for the foundation soils were physical property tests on selected samples of lean to fat clay, sandy silt, silty sand, and poorly-graded gravel. Results of gradation analyses and Atterberg limits tests suggest that the shear strength and permeability of foundation overburden materials could span a wide range.

Unconfined compression tests were performed on rock core samples representing the three major geologic units in the proposed dam foundation. These are the
sandstone, interbedded sandstone and shale, and claystone shale. Compressive strengths of the sandstone ranged from 1282 to 6736 psi, with an average value of 2578 psi. The interbedded sandstone and shale rock core yielded compressive strength values ranging from 1258 to 2817 psi, with an average of 1786 psi. The claystone shale strengths were the most variable, exhibiting a range of 1120 to 9432 psi and an average of 4981 psi. The above rock core compressive strengths, even though on the lower bound of the range of typical strengths for these types of rock, would be adequate to support the proposed embankment dam.
III. PRELIMINARY DESIGN

DAM FOUNDATION

Criteria
Embankment dam foundation design involves determining suitable methods of treatment for the foundation and abutments to insure stability of the dam, eliminate or reduce the potential for deformations in the embankment, reduce and/or control seepage through the foundation and abutments, and implement measures to safeguard against internal erosion of embankment material into the dam foundation. Foundation treatment methods could include partial-to-complete excavation of overburden soils, excavation of weathered bedrock, shaping of rock cliffs and overhangs on abutments, deep curtain grouting of foundation rock, shallow blanket grouting within the vicinity of the impervious core and foundation contact, placing dental concrete in irregularities on the rock surface, and slush grouting exposed cracks in the foundation bedrock. The foundation conditions at the proposed damsite would require implementing each of these treatment techniques to a certain degree.

Results
The variability of the alluvial and colluvial overburden soil deposits in the floodplain introduces numerous uncertainties about the proposed dam foundation. Of major concern are the low-density silt and sandy silt lenses encountered in the alluvial deposits during the geotechnical investigations performed this year. In-situ density and moisture content determinations suggest the material would be susceptible to collapse when loaded with the embankment, and subsequently inundated during reservoir impoundment. The low standard penetration test (SPT) blow counts for this material encountered in the boreholes also support this hypothesis.

Another material of concern in the overburden soils is the lean-to-fat clay lenses which were occasionally encountered. The particle size distribution and Atterberg limits indicate this material would exhibit low shear strength. The numerous meanders of Gooseberry Creek and the depositional nature of the floodplain deposits preclude identification and mapping of these low density and weak lenses by any practical exploration techniques.
NET states that the foundation soils above the gravel will require densification in order to support the proposed embankment and minimize settlement. The effectiveness of dynamic compaction techniques to densify soils, especially partially-saturated soils at depth, is questionable. Because of the many uncertainties associated with the overburden deposits and the questionable effectiveness of dynamic compaction, densification techniques were not considered in the preliminary foundation design. Work being performed at Jackson Lake Dam in Wyoming will help to provide valuable data on this potentially viable foundation treatment alternative.

The preliminary foundation design calls for excavating the foundation soils beneath the proposed embankment, down to the silty gravel (GM) layer overlying the bedrock. Beneath the impervious core zone, the excavation would continue through the gravels into sound bedrock, thus providing a barrier against under-seepage. A typical foundation excavation section is shown in Figure 4.

This foundation excavation scheme assumes that the silty gravel layer overlying the bedrock would remain in place, except beneath the core zone. Because of the depth, only bore holes were used to explore these gravels. A typical description of this deposit reads as follows:

Silty gravel with sand; medium dense to very dense; small to large gravel with fine to coarse sand; low to non-plastic fines; saturated; occasional clayey lenses.

SPT blow counts in the gravels were very high, typically in excess of 40. Even though this value is indicative of dense to very dense materials, SPT values for gravelly soils are not particularly reliable. If during construction the gravel deposit was found to be relatively loose and the clayey lenses appeared to be continuous, excavation of this material beneath the entire proposed embankment may be required.

Bore holes which penetrated the bedrock revealed a surficial zone of weathered and highly fractured rock that was typically about 5 feet in depth. In some locations, particularly on the right abutment, the depth of this zone
EMBANKMENT EXPLANATION

1. <last day derived from weathered Cady Shale>
2. Sands and gravels from alluvial borrow area
3. Fine-grained excavation and tailings form required
4. Compensate and filter from alluvial borrow area or off
5. Processed sand and gravel from alluvial borrow area or off
6. Processed sand and gravel from alluvial borrow area
7. Sanitave rock fragments from sanitary excavation and
drainage foundation shaping

PHASE II - CONCEPTUAL DESIGN

PHASE I - FEASIBILITY DESIGN

UPPER GOOSEBERRY DAM
GOOSEBERRY LEVEL III PROJECT
CROSS SECTIONS
WYOMING WATER DEVELOPMENT
COMMISSION

FIGURE 4
approached 10 to 15 feet. This material would be excavated below the core zone down to a rock surface which is treatable using dental concrete and slush grout.

Table 1 presents approximate required foundation excavation quantities for the various design floods considered. The numbers reflect the assumption that the silty gravels overlying the bedrock would remain in place.

<table>
<thead>
<tr>
<th>INFLOW DESIGN FLOOD</th>
<th>FOUNDATION EXCAVATION (yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10,000 yr</td>
<td>540,000</td>
</tr>
<tr>
<td>PMF/4</td>
<td>550,000</td>
</tr>
<tr>
<td>PMF/2</td>
<td>565,000</td>
</tr>
<tr>
<td>PMF</td>
<td>572,000</td>
</tr>
</tbody>
</table>

The sandstone cliff exposed on the proposed lower right abutment near the creek would require careful shaping to minimize the potential for differential settlement of the embankment.

Water pressure tests performed in the exploratory bore holes indicated the bedrock comprising the abutments is typically more pervious than that in the valley floor. Using these test results in conjunction with rock quality designations (RQD) and core recovery, a preliminary grouting program was developed for the proposed foundation. Curtain grouting, approximately 30 feet deep in the valley floor and 50 to 90 feet deep in the abutments, would be performed to minimize seepage losses from the reservoir. Shallow blanket grout holes, approximately 10 feet deep, would be required in localized areas of the foundation to "tighten" joints in near-surface rock.
EMBANKMENT ZONING AND MATERIALS UTILIZATION

Procedure
The preliminary embankment zoning shown in Figure 4 was developed based largely on results of the 1986 field exploration program. Many components of the preliminary design were established using judgment and assumed soil parameters that were to be verified by the laboratory testing program. Refinement of the preliminary design was to proceed using the laboratory results and detailed analyses. However, upon notice of project termination, laboratory tests which were planned but not in progress were subsequently cancelled. Unfortunately, tests performed in the latter phase of a soils program are typically the most important for analysis and design. Initial tests usually focus on determining physical properties of the soils on which selection of the more detailed shear strength, permeability, and consolidation tests are based.

The preliminary design of the proposed Upper Gooseberry Dam contains seven different material zones as follows:

1. Core
2. Shells
3. Filter
4. Drain
5. Transition
6. Bulk Fill
7. Slope Protection

Each zone serves a specific purpose within the embankment. This purpose, along with the potential material source for each zone, will be discussed in the following paragraphs.

Table 2 presents the total proposed embankment volumes for the inflow design floods considered.
TABLE 2
TOTAL VOLUME AND CREST ELEVATION OF PROPOSED EMBANKMENT DAM

<table>
<thead>
<tr>
<th>INFLOW DESIGN FLOOD</th>
<th>CREST ELEVATION (ft)</th>
<th>TOTAL VOLUME (yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10,000 yr.</td>
<td>6416</td>
<td>991,500</td>
</tr>
<tr>
<td>PMF/4</td>
<td>6419</td>
<td>1,050,000</td>
</tr>
<tr>
<td>PMF/2</td>
<td>6423</td>
<td>1,125,000</td>
</tr>
<tr>
<td>PMF</td>
<td>6425</td>
<td>1,165,000</td>
</tr>
</tbody>
</table>

For the remainder of this section of the report, approximate volumes presented for each zone are associated with the PMF/4 design.

Core
The primary function of the core zone is to retard the flow of water through the embankment; therefore, the essential property for the material comprising this zone is that of low permeability. Strength is of a secondary nature only.

The preliminary design indicates that approximately 146,000 yd³ of material would be required to construct the core. Based on calculations presented previously for the total available volume of core material, it appears sufficient quantities of satisfactory material exist for this zone.

As shown in Figure 4, the core has been designed as an upstream sloping zone. There are two principal reasons for this configuration:

1. The maximum height of fine-grained material subject to consolidation is reduced with this configuration as compared to a centrally located core. This should result in less embankment settlement.

2. Shifting the core upstream provides space for placement of a bulk fill zone in the downstream shell without affecting the outer slope. As will be discussed later, this bulk fill zone allows for more efficient use of construction materials at the site.
Conservative design practice dictates that the minimum effective width of the core should not be less than 30 to 50 percent of the normal reservoir head at any point.

**Shells**
The upstream and downstream shells support the core zone and provide stability to the embankment. Therefore, the essential material property is that of strength. If possible, it is desirable for the upstream shell to be constructed using pervious material so that pore water can drain during reservoir drawdown. Permeability is of secondary nature for the downstream shell.

The outer slopes of the dam are governed primarily by the shear strength of the shell zones, but also by strengths of internal zones and foundation material through which potential failure surfaces could develop. The preliminary design slopes were set at 3H to 1V upstream and 2.5H to 1V downstream using judgment and knowledge of past behavior of similar materials. Using these outer slopes, the volume of shell material required in the proposed embankment was computed to be approximately 660,000 yd³.

The alluvial deposits located in the proposed reservoir borrow area would be utilized for the shell material. As discussed previously, the deposit is primarily a clean to silty, poorly-graded gravel near the existing creek channel. Away from the channel on the terraces, the cleaner gravels are overlain by a silty sand, with occasional lenses of sandy silt. The clean, poorly-graded gravels would be a superior material for the shell zones of an embankment dam. If the shells were composed entirely of this type of material, it is possible that the slopes could be steepened slightly. However, it is unlikely that the shells would be composed entirely of this desirable material because of the variability of the alluvial borrow, the high water table, and the fairly limited quantity of this material in the borrow.

Detailed slope stability analyses were to be performed using laboratory shear strength tests results to optimize the proposed embankment slopes.
**Bulk Fill**

In order to efficiently utilize the various materials that may be available for construction at the proposed damsite, zones of different materials can be defined within the overall limits of the shells. For the preliminary design, a zone of heavily compacted silts, silty sands, and sandy silts was incorporated into the downstream shell beneath the upstream sloping core. Because of the abundance of these materials in the required foundation excavation and overlying the desirable alluvial gravels in the proposed reservoir borrow, optimizing the size of this bulk fill zone would be beneficial in terms of embankment cost.

The preliminary configuration of this zone is shown in Figure 4. The material quantity associated with this configuration is approximately 126,000 yd$^3$. As this zone increases in size, the shear strengths of the various materials comprising the bulk fill would begin to govern the downstream slope. Since the bulk fill would contain finer-grained materials, its shear strength would be less than that of the predominantly gravel shells. Thus, as the volume of the bulk fill zone increases, it would be necessary to flatten the downstream slope. Further detailed stability and cost studies would be required to achieve the most cost effective balance between bulk fill and downstream shell quantities.

**Filter**

The purpose of the filter zone in the preliminary design is to protect against internal erosion of the core material into the downstream shell and/or foundation under the action of seepage. The filter material must meet rigid gradation and durability requirements.

The filter zone would be placed immediately downstream of the core zone beginning at the normal reservoir water surface elevation and extending to the foundation bedrock surface. All exposed bedrock in the cut-off trench downstream of the core zone would be blanketed with filter material to prevent migration of foundation materials into the drain and/or downstream shell zones.
The width of the filter zone was set at 8 feet. This was established considering a practical width that could be placed using conventional construction equipment. The quantity of filter material associated with the preliminary embankment design is approximately 34,000 yd$^3$.

At this stage, it was assumed that filter material would be processed from the alluvial deposits in the proposed reservoir borrow area. The presence of friable sandstone particles within the borrow raised questions regarding the quality of this possible filter material source. Aggregate durability tests to determine the suitability of this material were planned, but not completed.

**Drain**

The purpose of the drain zone in the preliminary design is to transmit the seepage that passes through the core and foundation out of the embankment in a controlled manner, and prevent the phreatic surface from entering the downstream shell. The drain material must meet rigid gradation and durability requirements.

The drain zone would be placed immediately downstream of the filter zone, beginning 10 feet below the normal reservoir water surface elevation and continuing down to the filter material overlying the bedrock foundation surface. For a distance of approximately 200 feet across the maximum section of the valley, a 4-foot thick horizontal drainage blanket would be placed beneath the downstream shell to collect seepage and transmit it to the downstream toe of the embankment. At the toe, drain tubing would be installed to provide for controlled discharge and measurement of the seepage.

The horizontal drainage blanket would be placed on top of a 2-foot thick zone of filter material, which would be placed on the excavated foundation surface. This would prevent migration of materials from the dam foundation into the gravel drain. Similarly, a 2-foot thick filter blanket would be installed between the bulk fill zone and drainage blanket to prevent contamination of the drain by finer materials from the bulk fill zone.

The width of the chimney portion of the drain zone was set at 8 feet. This was established again considering a practical width for using conventional
construction equipment. The horizontal drainage blanket within the maximum section of the valley was sized to conduct the anticipated embankment and foundation seepage.

Based on the preliminary design, approximately 33,000 yd\(^3\) of drain material would be required. At this stage, it was assumed that drain material would be processed from the alluvial borrow deposits. However, the friable sandstone fragments within the borrow raised questions concerning durability of the drain material. Breakdown of gravel-sized sandstone fragments, with time, could reduce the hydraulic capacity of the drain. Aggregate durability tests to determine the suitability of the alluvial borrow deposits for use as drain material were planned, but not completed.

**Transition**

The upstream transition zone fulfills a dual purpose. First, the zone acts as a filter media between the upstream shell and core zones for drainage from the core during reservoir drawdown. Second, in the event that cracks develop in the core, material would be transported into these cracks from the transition thereby reducing the amount of seepage that would otherwise occur.

The downstream transition zone acts as a buffer between the drain zone and finer-grained materials in the bulk fill zone to prevent contamination of the drain.

The width of both transition zones was set at 8 feet again based on a practical width for conventional construction equipment. The material quantities required for the upstream and downstream transition zones are approximately 31,000 and 22,000 yd\(^3\), respectively.

While the gradation requirements of the transition need not be as stringent as that of the filter zone, the transition material, like the filter material, must not have the capability to support open cracks that could develop as a result of deformations or displacements in the embankment or foundation. For the preliminary design, it was assumed that this zone would be obtained through minimal processing of the alluvial materials in the proposed reservoir.
borrow area. Another potential source was revealed by test pit B-4 located north of the proposed reservoir borrow. The material encountered in this test pit was a poorly-graded fine to medium sand, containing 30 percent silt fines (SM). Further investigation of this area would be required to determine if this is a viable source.

**Slope Protection**
This zone provides erosion protection to the upstream shell zone against wave action in the reservoir. The zone is typically made up of cobble and boulder size (6 to 24 inch) material, known as riprap, capable of withstanding forces exerted by waves without displacement. The specific size distribution of the riprap is a function of the wave action that a design wind and reservoir combination is capable of producing. The material comprising this zone should be reasonably sound and durable.

A single layer of riprap, 2-feet thick measured normal to the slope, was incorporated into the preliminary embankment design. The riprap would extend from the dam crest down to an elevation 5 feet below the reservoir dead pool elevation. Using this configuration, approximately $12,000 \text{ yd}^3$ of good quality riprap would be required.

Because the sandstone at the site appears to be of poor quality for use as riprap, it may be necessary to provide a thicker slope protection zone and accept deterioration of the rock fragments with time. Rock excavation from the foundation cutoff, abutment shaping, and spillway excavation could be utilized in an upstream rock fragment zone. This would also reduce costs slightly, since this material must be excavated regardless of whether it is used or wasted. Thus, the quantity of gravel shell material could be reduced.

The downstream face of the embankment would be mulched and seeded to provide grass cover erosion protection.

**EMERGENCY SPILLWAY - DAM BREACH ANALYSIS**

**Procedure**
The size of the emergency spillway was approximated for four levels of design
floods. The floods were routed through the reservoir under two conditions—one with the design flood, and another with a dam breach. The spillway width was established by using a constant unit discharge and rounding the width to the nearest five feet. The maximum reservoir level was determined during the routing and at least one foot of freeboard was added to establish the dam crest. The intention was to optimize the spillway width and surcharge combination by comparing costs. However, the dam design effort was terminated before this activity was completed.

Computation of various design flood levels was described in the Phase I report. These floods were routed through the reservoir to determine flood stages at selected locations below the reservoir. Cross-sections, at several critical locations below the dam, were surveyed for use in the flood routing. The location of these sections, shown on Figure 5, were at ranch headquarters, bridges, and farmsteads.

Design flood selection criteria is a dynamic process which is constantly changing. New meteorological approaches and dam safety hazards are two areas producing changes. No criteria have been established for Wyoming and each site is analyzed independently. Thus, the reason for evaluating four design flood levels.

The four design floods were routed through the reservoir with the results shown on Figure 6. This indicates the flood stage at various selected points below the reservoir. Routing of the floods without a reservoir was also accomplished to establish a baseline under existing conditions.

Dam safety guidelines require a breach analysis to be made of the dam. Unfortunately, there is a lack of agreement on the dam breach method to be used, and on data from actual dam breach failures. There is support for using a dam breach flood peak which just exceeds the design flood. This is the minimum flood which will cause the dam to fail due to overtopping and breaching. However, there is also support for using the PMF as the breach flood, although the probability of occurrence is much less.
STREAM REACH USED IN DESIGN FLOOD ANALYSIS
GOOSEBERRY PROJECT, WYOMING

FIGURE 5
Several factors influence the breach computation, the most critical of which are the time it takes the breach to occur and the shape of the breach opening. A model has been developed which incorporates physical factors into a breach computation. These factors include material properties of the dam such as median grain size, unit weight, friction angle, dam height, top width, dam slopes, reservoir capacity, and outlet capacity. The model computes the rate of breach formation, geometry of the breach opening, and resulting outflow hydrograph.

**Results**

The results of the breach model were plotted on the profile of those critical cross-sections below the dam. This provided an indication of the significance of the dam breach at the various flood levels. Although some cross-sections were surveyed, other values were interpolated, and the accuracy relative to flood stage and property locations is only approximate. Because of the assumptions used in the design flood computation and the breach development, the results likely represent a "high side" case. The results for the three critical cross sections are shown in Figures 7, 8, and 9.

Interest in evaluating incremental damages for different level floods was expressed by the Wyoming State Engineer. An initial approach was made at analyzing this condition. The results are shown in Table 3 and indicate the design flood level is between the 10,000 year frequency and the 1/4 PMF. A complete discussion of the flood routing and dam breach analysis appears in Appendix H.

**OUTLET WORKS AND PRINCIPAL SPILLWAY**

No changes were made in the outlet works and principal spillway size and location from that shown in the Phase I report. The outlet works was sized to discharge the maximum monthly requirement at minimum reservoir water surface elevation 6337. The outlet capacity increases to 215 cfs at top of inactive pool elevation 6365.

The principal spillway will consist of an overflow weir discharging into a wet well structure which utilizes the same conduit as the outlet works. Three feet of surcharge storage above the spillway weir, to the invert of the
Section A

Design Flood Magnitude

10,000 Yr. 1/4 PMF 1/2 PMF PMF

Elevation, Ft.

6230 6220 6210 6200 6190

Station, Ft.

1+00 2+00 3+00 4+00 5+00 6+00 7+00 8+00 9+00 10+00

Approximate Structure Location

- Dam Breach, PMF Inflow
- Dam Breach, Design Flood Inflow +
- Design Flood, No Reservoir
- Reservoir Routed Design Flood, No Breach

FIGURE 7
Section 2

Design Flood Magnitude

10,000 Yr.  1/4 PMF  1/2 PMF  PMF

Dam Breach, PMF Inflow

Dam Breach, Design Flood Inflow

Design Flood, No Reservoir

Reservoir Routed Design Flood, No Breach

Approximate Structure Location

Elevation, Ft.

Station, Ft.
Section 4

Design Flood Magnitude

10,000 Yr. 1/4 PMF 1/2 PMF PMF

5340

5330

5320

5310

5300

5300 1+00 2+00 3+00 4+00 5+00 6+00 7+00 8+00 Station, Ft.

5340

5330

5320

5310

5300

5300

Approximate Structure Location

Dam Breach, PMF Inflow

Dam Breach, Design Inflow +

Design Flood, No Reservoir

Reservoir Design

Flood, No Breach
emergency spillway, will contain the 100-year flood volume. The overflow weir will discharge 100 cfs at three feet of head and automatically pass the 100-year flood volume.
<table>
<thead>
<tr>
<th>FLOOD LEVEL OR CONDITION</th>
<th>RESIDENTIAL DAMAGE</th>
<th>LOSS OF LIFE POTENTIAL</th>
<th>RESERVOIR COST</th>
<th>INCREMENTAL COST</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A 2 4</td>
<td>A 2 4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10,000 yr.</td>
<td>None None None</td>
<td>L L L</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10,000 yr. (Res)</td>
<td>None None None</td>
<td>L L L</td>
<td></td>
<td>7,750,000</td>
</tr>
<tr>
<td>10,000 yr. (Breach)</td>
<td>None 2' 6'</td>
<td>L M M</td>
<td></td>
<td>600,000</td>
</tr>
<tr>
<td>1/4 PMF</td>
<td>None None Slight</td>
<td>L L L</td>
<td></td>
<td>8,350,000</td>
</tr>
<tr>
<td>1/4 PMF (Res)</td>
<td>None None None</td>
<td>L L L</td>
<td></td>
<td>980,000</td>
</tr>
<tr>
<td>1/4 PMF (Breach)</td>
<td>6' 4' 8'</td>
<td>M M M</td>
<td></td>
<td>2,150,000</td>
</tr>
<tr>
<td>1/2 PMF</td>
<td>None None 3'</td>
<td>M L L</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/2 PMF (Res)</td>
<td>None None 18&quot;</td>
<td>L L L</td>
<td></td>
<td>9,330,000</td>
</tr>
<tr>
<td>1/2 PMF (Breach)</td>
<td>18&quot; 5' 10'</td>
<td>M M M</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PMF</td>
<td>None 12&quot; 7'</td>
<td>M M M</td>
<td></td>
<td>11,480,000</td>
</tr>
<tr>
<td>PMF (Res)</td>
<td>None None 6'</td>
<td>M M M</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(Agricultural damages were assumed to be constant. This would not always be true with regard to fences, haystacks, etc., but data were not available.)

/1 Approximate depth of water in residence. No attempt was made to assign monetary values because of a lack of knowledge regarding residence size, age, value, etc.

/2 Potential shown as L - Low, M - Moderate, H - High. Low generally indicates depths below 4 ft., except in cases where the flow depths and velocities in the adjacent floodplain were judged to present hazards, e.g., sections A and 2. High indicates depths over 6 ft.

/3 Costs based on the unit costs derived with adjustments for outlet works and spillway.
IV. COST ESTIMATES

The preliminary construction cost estimate was derived from a 37-bid item list representing quantities calculated for major work items necessary to construct the proposed project. Unit costs were assigned to each bid item based on recent bids for heavy construction work in the western United States and MKE's experience with work in the Rocky Mountain region. Table 4 is a list of the bid items, quantities, unit costs, and total cost per item. The total preliminary construction cost estimate is shown at the bottom of the table. Table 5 is a list of similar heavy construction projects for which cost abstracts were obtained and used in estimating the construction cost of the proposed project. Unit costs for many bid items can span a wide range. The unit costs presented in Table 4 were selected using judgment considering the quantities, project location, complexity of construction, and economic conditions of the region.

The bid item list reflects the quantities of the PMF/4 project design. Quantities associated with the embankment zones, foundation excavation, spillway excavation, and outlet works length change for the various inflow design floods considered. At this stage, the other items in the bid list were assumed to remain fairly constant in relation to the major quantity items mentioned above.

Figure 10 presents the total construction cost for each major component of the project, and demonstrates how these costs vary with the inflow design flood size. The embankment construction cost in this figure reflects not only each zone cost, but also the costs required for foundation excavation and all other miscellaneous bid items not included in the spillway and outlet works. As can be seen from the figure, the total embankment and outlet works construction costs do not vary greatly with the design flood levels. However, the spillway cost rises exponentially with increasing inflow design floods. The magnitude of each inflow design flood versus the total project construction cost is shown on Figure 11.
## TABLE 4
PRELIMINARY CONSTRUCTION COST ESTIMATE FOR PMF/4 PROJECT DESIGN
GOOSEBERRY LEVEL III PROJECT

<table>
<thead>
<tr>
<th>ITEM</th>
<th>DESCRIPTION</th>
<th>QUANTITY</th>
<th>UNIT</th>
<th>COST/UNIT</th>
<th>COST</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization and Demobilization</td>
<td>1</td>
<td>LS x 500,000</td>
<td>500,000</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Diversion and Care of Stream</td>
<td>1</td>
<td>LS x 50,000</td>
<td>50,000</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Foundation Dewatering</td>
<td>1</td>
<td>LS x 200,000</td>
<td>200,000</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Clearing and Grubbing</td>
<td>100</td>
<td>ACRE x 800.00</td>
<td>80,000</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Stripping Dam Site and Borrows</td>
<td>100,000</td>
<td>CY x 1.00</td>
<td>100,000</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Access Roads</td>
<td>1</td>
<td>LS x 52,000</td>
<td>52,000</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Guard Rails</td>
<td>500</td>
<td>LF x 12.00</td>
<td>6,000</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Instrumentation</td>
<td>1</td>
<td>LS x 40,000</td>
<td>40,000</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Seeding</td>
<td>25,000</td>
<td>SY x 0.10</td>
<td>2,500</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>Dam Foundation Excavation; Common</td>
<td>550,000</td>
<td>CY x 2.00</td>
<td>1,100,000</td>
<td></td>
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<tr>
<td>11</td>
<td>Dam Foundation Excavation; Rock</td>
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<td>CY x 10.00</td>
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<tr>
<td>12</td>
<td>Zone 1 Core; Excavate and Place</td>
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<td>CY x 4.00</td>
<td>584,000</td>
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<td>13</td>
<td>Zone 2 Shell; Excavate and Place</td>
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<td>14</td>
<td>Zone 3 Bulk Fill; Placement</td>
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<td>CY x 0.50</td>
<td>63,000</td>
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<td>15</td>
<td>Zone 4 Filter; Excavate, Process and Place</td>
<td>34,000</td>
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<td>340,000</td>
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<tr>
<td>16</td>
<td>Zone 5 Drain; Excavate, Process and Place</td>
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<tr>
<td>17</td>
<td>Zone 6 Transition; Excavate and Place</td>
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<td>159,000</td>
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</tr>
<tr>
<td>18</td>
<td>Zone 7 Slope Protection; Excavate and Place</td>
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<td>CY x 13.00</td>
<td>156,000</td>
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<tr>
<td>19</td>
<td>Excavation for Dental Concrete</td>
<td>150</td>
<td>CY x 50.00</td>
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<tr>
<td>20</td>
<td>Dental Concrete</td>
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<td>CY x 70.00</td>
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<tr>
<td>21</td>
<td>Slush Grouting</td>
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<td>CF x 5.00</td>
<td>50,000</td>
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</tr>
<tr>
<td>22</td>
<td>Foundation Cleanup</td>
<td>7,000</td>
<td>SY x 5.00</td>
<td>35,000</td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>12-Inch HDPE Perforated Drain Pipe</td>
<td>200</td>
<td>LF x 20.00</td>
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<td></td>
</tr>
<tr>
<td>ITEM</td>
<td>DESCRIPTION</td>
<td>QUANTITY</td>
<td>UNIT</td>
<td>COST/UNIT</td>
<td>COST</td>
</tr>
<tr>
<td>------</td>
<td>------------------------------------------------</td>
<td>----------</td>
<td>------</td>
<td>-----------</td>
<td>--------</td>
</tr>
<tr>
<td>24</td>
<td>Mobilization and Demobilization for Pressure Grouting</td>
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</tr>
<tr>
<td>25</td>
<td>Furnish and Place Metal Pipe</td>
<td>3,500</td>
<td>LBS x</td>
<td>1.00</td>
<td>3,500</td>
</tr>
<tr>
<td>26</td>
<td>Drill Set-Ups</td>
<td>330</td>
<td>EA x</td>
<td>25.00</td>
<td>8,250</td>
</tr>
<tr>
<td>27</td>
<td>Drilling Grout Holes</td>
<td>18,000</td>
<td>LF x</td>
<td>12.00</td>
<td>216,000</td>
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<tr>
<td>28</td>
<td>Coring NX-Size Check Holes</td>
<td>1,800</td>
<td>LF x</td>
<td>20.00</td>
<td>36,000</td>
</tr>
<tr>
<td>29</td>
<td>Casing Grout Holes</td>
<td>1,800</td>
<td>LF x</td>
<td>10.00</td>
<td>18,000</td>
</tr>
<tr>
<td>30</td>
<td>Hook-Ups to Grout Holes</td>
<td>1,100</td>
<td>EA x</td>
<td>25.00</td>
<td>27,500</td>
</tr>
<tr>
<td>31</td>
<td>Pressure Grouting Foundation</td>
<td>9,000</td>
<td>BAGS x</td>
<td>10.00</td>
<td>90,000</td>
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<tr>
<td>32</td>
<td>Furnish and Handle Cement for Pressure Grouting</td>
<td>9,500</td>
<td>BAGS x</td>
<td>5.00</td>
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<tr>
<td>33</td>
<td>Water Tests in Grout Holes</td>
<td>1,100</td>
<td>EA x</td>
<td>20.00</td>
<td>22,000</td>
</tr>
<tr>
<td>34</td>
<td>Spillway Excavation; Common</td>
<td>13,000</td>
<td>CY x</td>
<td>2.00</td>
<td>26,000</td>
</tr>
<tr>
<td>35</td>
<td>Spillway Excavation; Rock</td>
<td>28,000</td>
<td>CY x</td>
<td>10.00</td>
<td>280,000</td>
</tr>
<tr>
<td></td>
<td>(In excess of 15,000 used for riprap)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>Careful Blasting; Spillway</td>
<td>2,350</td>
<td>SY x</td>
<td>20.00</td>
<td>47,000</td>
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<tr>
<td>37</td>
<td>Outlet Works Structure</td>
<td>1</td>
<td>LS x</td>
<td>825,000</td>
<td>825,000</td>
</tr>
<tr>
<td></td>
<td>SUBTOTAL</td>
<td></td>
<td></td>
<td></td>
<td>7,586,250</td>
</tr>
<tr>
<td></td>
<td>+ 10% CONTINGENCY</td>
<td></td>
<td></td>
<td></td>
<td>758,625</td>
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<tr>
<td></td>
<td>TOTAL</td>
<td></td>
<td></td>
<td></td>
<td>8,344,875</td>
</tr>
</tbody>
</table>

TABLE 4 (Page 2)
PRELIMINARY CONSTRUCTION COST ESTIMATE FOR PMF/4 PROJECT DESIGN
GOOSEBERRY LEVEL III PROJECT
# TABLE 5
## LIST OF COST ABSTRACTS

<table>
<thead>
<tr>
<th>PROJECT</th>
<th>STATE</th>
<th>YEAR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strontia Springs Dam</td>
<td>Colorado</td>
<td>1979</td>
</tr>
<tr>
<td>Spinney Mountain Dam</td>
<td>Colorado</td>
<td>1980</td>
</tr>
<tr>
<td>Windy Gap Project</td>
<td>Colorado</td>
<td>1981</td>
</tr>
<tr>
<td>Willow Creek Dam</td>
<td>Oregon</td>
<td>1981</td>
</tr>
<tr>
<td>Sultan Dam</td>
<td>Washington</td>
<td>1982</td>
</tr>
<tr>
<td>Ridgway Dam</td>
<td>Colorado</td>
<td>1982</td>
</tr>
<tr>
<td>Winchester Reservoir</td>
<td>Kentucky</td>
<td>1983</td>
</tr>
<tr>
<td>Upper Stillwater Dam</td>
<td>Utah</td>
<td>1983</td>
</tr>
<tr>
<td>Middle Fork Dam</td>
<td>Colorado</td>
<td>1984</td>
</tr>
<tr>
<td>Galesville Dam</td>
<td>Oregon</td>
<td>1984</td>
</tr>
<tr>
<td>San Justo Dam</td>
<td>California</td>
<td>1984</td>
</tr>
<tr>
<td>Brantley Dam</td>
<td>New Mexico</td>
<td>1984</td>
</tr>
<tr>
<td>Colorado Department of Highways</td>
<td>Colorado</td>
<td>1985</td>
</tr>
<tr>
<td>Pactola Dam Modification</td>
<td>South Dakota</td>
<td>1985</td>
</tr>
<tr>
<td>Jackson Lake Dam Modification</td>
<td>Wyoming</td>
<td>1986</td>
</tr>
<tr>
<td>Spring Creek Dam Rehabilitation</td>
<td>Colorado</td>
<td>1986</td>
</tr>
</tbody>
</table>
UPPER GOOSEBERRY DAM
DESIGN FLOOD LEVEL vs. CONSTRUCTION COST

FIGURE 10
UPPER GOOSEBERRY DAM
INFLOW DESIGN FLOOD vs. CONSTRUCTION COST

CONSTRUCTION COST ($ Millions)

INFLOW DESIGN FLOOD (1000 CFS)

FIGURE 11
A contingency of 10 percent of the preliminary construction cost estimate was included in the total cost. This contingency represents unlisted bid items and unknown factors that might impact construction.
V. COMPARISON WITH FEASIBILITY ESTIMATE

Results of the geotechnical investigations described earlier in this report necessitated modifications to the feasibility design of the proposed Upper Gooseberry Dam. A major design change was the foundation preparation for the dam. Extensive low density silt layers of varying thickness scattered throughout the foundation would require removal prior to placing the embankment. The total volume of overburden excavation necessary to remove this material is 550,000 yd$^3$. The similar volume estimated for the feasibility design was 115,000 cu yds.

The geometry of the dam core was changed primarily because of a limited amount of impervious material. This decreased the amount of core material required, but increased the size of the shells. This did not have a significant impact on the cost because the unit cost of the core was slightly higher than the unit cost of the shell. Refinements in the filter and drain provisions of the latest design increased the cost slightly.

An item which increased the cost significantly was the need for foundation grouting. Geotechnical analysis during the Phase I drilling indicated the permeability of the bedrock was fairly low. Only a minor amount of grouting on the abutments and obvious discontinuities along the foundation was recommended. However, the presence of several stress relief joints was observed during Phase II exploration, indicating the need for a more extensive grouting program. A site visit was also made to Lower Sunshine Dam and Reservoir, where extensive seepage around or under the dam was observed. This site is similar geologically to the Upper Gooseberry site and the seepage amount dictates a need to try and salvage as much water as possible at the Gooseberry site.

Another item which increased the project cost was the emergency spillway. Earlier estimates and geologic assumptions during Phase I exploration placed the spillway invert in bedrock, just upslope from the right end of the dam crest. Additional geotechnical exploration indicated a much greater colluvium cover over the bedrock at this location. The result was the necessity to move
the spillway farther upslope so the channel could be founded in the bedrock. This would require additional excavation of rock and overburden.

A comparison of several parameters for the two design level dams is shown in Table 6. The feasibility estimate was for a dam having a design flood of 1/2 PMF, and thus the values are slightly larger than a 1/4 PMF design would have been. The increase in embankment volume shown for the latest design is mainly the result of the additional excavation for the foundation.

TABLE 6
COMPARISON OF PARAMETERS
GOOSEBERRY LEVEL III PROJECT

<table>
<thead>
<tr>
<th>DESIGN LEVEL</th>
<th>FOUNDATION EXC. (yd³)</th>
<th>TOTAL EMBANKMENT VOLUME (yd³)</th>
<th>COST ESTIMATE ($ x 10⁶)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feasibility</td>
<td>115,000</td>
<td>860,000</td>
<td>5.0</td>
</tr>
<tr>
<td>Preliminary Final</td>
<td>550,000</td>
<td>1,050,000</td>
<td>8.3</td>
</tr>
</tbody>
</table>

The cost increase is primarily the result of the three major items mentioned previously. These are:

1. The additional excavation and subsequent placement of embankment necessitated by the presence of extensive silt lenses in the dam foundation.

2. Grouting to minimize seepage losses through the abutments and foundation.

3. Relocation of the emergency spillway further upslope to intersect the bedrock in the right abutment.
VI. CONCLUSIONS

Foundation conditions at the damsite are not as conducive to construction as originally estimated. Extensive layers of low density silt necessitated design changes which substantially increased construction cost estimates.

Sufficient material is available within the vicinity of the proposed site to construct the dam, with the possible exception of good quality sands and gravels for filters and drains.

The design flood which does not aggravate the natural conditions, or when routed provides some improvement during flooding, is the flood having a magnitude of \( \frac{1}{4} \) PMF. Some damage during a possible dam breach is likely, but this is the case under all design flood scenarios, except the PMF. However, the PMF would create damage during routing and have a high potential for causing loss of life.

The estimated cost of construction of a dam at the Upper Gooseberry site is nearly $2000 per acre foot of storage. Considerable carryover storage is required and the annual yield is about one-third the storage volume, resulting in an annual cost per acre-foot of yield of over $240. This is beyond the present capability of agriculture to repay.

Topographically, the Upper Gooseberry Dam site is at least equal to any other site analyzed during the study. One site just upstream from the upper site, sometimes referred to as the Bureau site, has a crest length about 200 feet shorter than Upper Gooseberry. However, this site is above Red Creek and has a drainage area 5 percent less than the upper site.

The most obvious overall conclusion reached during the Gooseberry investigation is the need for an additional water supply. Some means of bringing water into the basin should be identified and evaluated.