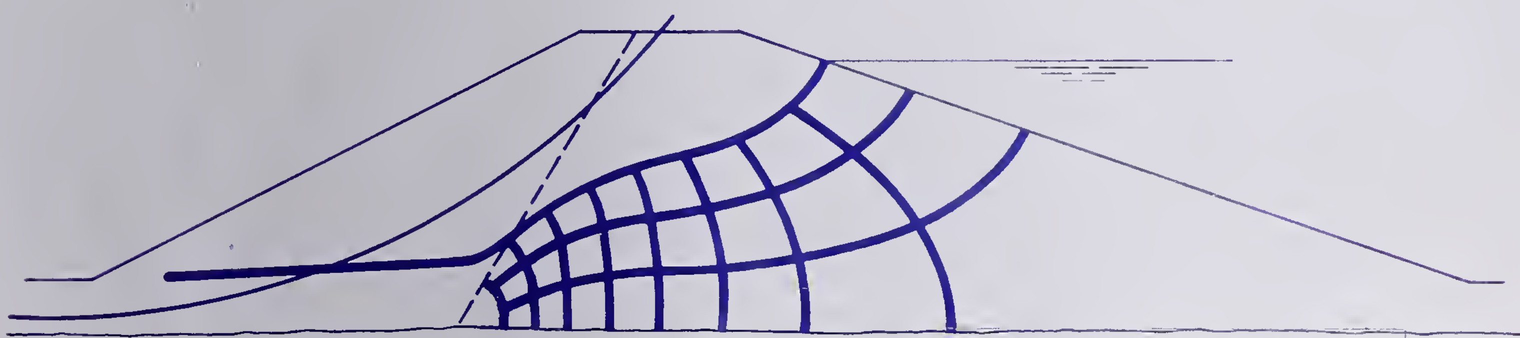
MIDDLE CREEK DAM

GEOTECHNICAL INVESTIGATION



Prepared for:

MONTANA DEPARTMENT
OF
NATURAL RESOURCES AND CONSERVATION

HKM ASSOCIATES

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April 1984

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MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION

Prepared for:
Montana Department of Natural Resources
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I. EXECUTIVE SUMMARY

A. PURPOSE OF STUDY

HKM Associates performed a geotechnical investigation for Middle Creek Dam in partial fulfillment of our contract, with the Montana Department of Natural Resources and Conservation (DNRC), No. WE-HKM-145. Middle Creek Dam is a zoned earth fill dam approximately 111 feet high with a low level outlet conduit and a concrete chute spillway with an ogee crest.

The dam is considered unsafe because the facility can only handle a portion of the design flood (PMF) without overtopping and causing failure, and the spillway is in bad repair (Ref. 11). The purpose of this geotechnical investigation is to provide an understanding of the subsoils conditions on which to evaluate rehabilitation alternatives for this facility.

"As built" drawings of the project are not available. However, design drawings have been obtained for this study and are on file at the DNRC in Helena, Montana (Ref. 22). There is no stability analysis of the embankment on file. No instrumentation, such as piezometers and survey control points, had been established at the facility prior to this study.

B. GENERAL APPROACH

The general approach began with a subsurface soils investigation to determine the physical and engineering properties of the materials. Subsequently, representative soil samples were selected for laboratory testing and engineering analyses were performed. The findings of these field and laboratory investigations and the engineering analyses were used as the basis for rehabilitation alternative selection and evaluation.

The field investigation included making 15 exploration drill holes and 14 test pits to determine soil conditions and obtain samples for laboratory testing. Instrumentation, to monitor embankment movements and changes in ground water levels, was installed during the field investigation. A geologic investigation was made to assist in evaluating the soils and the impact of a seismic event on the structure. The engineering analyses included seepage and stability studies. Both static and dynamic stability analyses were also performed.

Rehabilitation alternatives were conceptualized, however, design details have not been developed at the time of this report. Therefore, additional geotechnical analyses of the selected alternative will be made as the design develops. This contract will be completed in December 1984.

C. FINDINGS

The locations of the facility and the plan of the dam are shown on Sheet Nos. 1 and 2. The configuration of the profile and cross section of the embankment is presented on Sheet No. 3 in Appendix A. The findings during the field and laboratory programs are summarized on Sheet Nos. 4, 5 and 6.

Seepage through the left abutment is high but does not create a stability problem with the existing embankment. Presently, this seepage is being at least partially controlled by the existing trench drains. The condition of the foundation drain is unknown. An increase in the embankment crest elevation will require preserving and lengthening the existing trench drain system.

The impervious blanket on the left abutment upstream of the embankment was perforated by test pit exploration holes during the field investigation. This blanket should be repaired to its original function and size.

Seepage through the embankment and its foundation and through the right abutment does not appear to be a problem. Ground water levels and seepage throughout the downstream area should continue to be monitored. If the embankment and pool elevations are increased, a monitoring system should be constructed to record the trench drain discharge. It is recommended that the seepage from both abutments be monitored.

The results of the static stability analysis indicate the factors of safety (shown in Table 13) for the existing structure are within the Recommended Guidelines (Ref. 10). In order to maintain adequate static stability for the Reinforced Earth alternative, it will be necessary to hold the phreatic surface at an elevation near the existing phreatic surface. This may be accomplished by constructing an impervious barrier extending 6 to 8 feet below the bottom of the Reinforced Earth foundation. A trench drain system placed at the bottom of the impervious barrier trench will provide additional control of the phreatic surface. Stability using the earth fill alternative is not expected to be a problem.

The results of the dynamic stability analysis indicate that the minimum factor of safety against liquefaction is 1.5. These results are based on a maximum credible earthquake (design earthquake) of 6.5 (Richter Magnitude) at a distance of 25 kilometers from the dam with a horizontal acceleration of 0.22g at the base of the structure.

Deformation resulting from the design earthquake is calculated to range from 0.4 to 1.0 feet. This amount is relatively small compared to the available freeboard and is not considered a problem.

The two general rehabilitation alternatives conceptualized include raising the embankment crest by placing an earth fill on the crest and over the downstream face of the dam, or by constructing a cap on the crest, about 28 feet wide, with vertical walls retained by Reinforced Earth or an equivalent. These alternatives are represented on Sheet No. 3 of Appendix A.

Preliminary studies, not part of the geotechnical investigation, indicate that it is desirable to raise the embankment elevation 5 to 20 feet in order to increase the storage volume and the flood surcharge volume. Rehabilitation alternatives reported herein are limited to increasing the crest elevation approximately 10 feet.

The preferred rehabilitation alternative is the Reinforced Earth cap. The primary advantages of this system is that the cap adds less weight to the embankment, construction time is estimated to be much less, the existing outlet conduit does not have to be extended and less borrow is required for construction.

This report was prepared by HKM Associates' professional engineering staff in the Billings, Montana office.

Engineers having primary responsibility are presented below.

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II. INTRODUCTION

A. GENERAL

The purpose of this report is to present the geotechnical investigation for the Middle Creek Rehabilitation Feasibility Study. This report is submitted in fulfillment of a task (Task No. 3) requirement under contract (No. WE-HKM-145) with the Montana Department of Natural Resources and Conservation (DNRC).

This study consisted of a field drilling program, a geologic investigation, a laboratory investigation of soil properties, engineering analyses, and seepage monitoring. The engineering analyses included studying the existing seepage and stability questions as well as the anticipated questions associated with construction of the rehabilitation alternatives.

HKM Associates retained the firm of Rollins, Brown and Gunnell, Inc. (RBG), Provo, Utah for field drilling and testing and for professional consulting services. In addition, RBG performed the engineering properties tests during the laboratory investigation.

The Middle Creek Dam is an earthfill embankment located on Hyalite Creek (Middle Creek) in Gallatin County. The dam is located about 15 miles south of Bozeman, Montana on public land administered by the U.S. Forest Service. Middle Creek Reservoir (Hyalite Reservoir) has a storage capacity of 7,780 acre-feet (AF), at spillway crest and 10,230 AF at dam crest. This reservoir is presently used primarily for irrigation. Secondary uses are recreation and water supply for municipal water for the City of Bozeman and Montana State University.

Middle Creek Dam has a structural height of about 111 feet based on findings during this investigation and a crest length of about 1300 feet. The impoundment structures also include an 8-foot high earthfill dike located on the left abutment (direction determined by facing downstream) in the area of the existing Blackmore Campground. A 60-inch diameter low level outlet conduit is located near the center of the main embankment. A 40-foot wide concrete chute spillway with an ogee crest is located on the right abutment.

B. PROBLEM IDENTIFICATION

This project is classified, in accordance with the Corps of Engineers (COE) Phase I Inspection Report, National Dam Safety Program (Ref. 11), as having a high downstream hazard potential. Applying the appropriate design criteria (Ref. 10) for this hazard potential indicates that there are design inadequacies in the Middle Creek Dam project. Some of these inadequacies are summarized by the COE Phase I Inspection Report (Ref. 11) as follows:

> Inspection criteria recommends that a large-sized project with a high downstream hazard potential be capable of safely handling the probable maximum flood (PMF). The PMF is the flood expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region.
>
> Routing (studies) indicates that the dam is overtopped during the PMF when approximately 29 percent of the total flood volume enters the reservoir. Consequently, the project's spillway is considered seriously inadequate. The dam is constructed of materials that would quickly erode when overtopped by floodwaters.

The relatively flat embankment slopes, the embankment zoning, and the possibility of low water levels in the embankment, suggest that stability may conform with the recommended guidelines. Verification of this finding is required.

The spillway concrete is badly deteriorated, and the reservoir is operated to minimize spillway use.

On the basis of the field inspection and preliminary hydrologic analysis, Middle Creek Dam does not now conform to inspection guidelines with respect to discharge and/or storage capacities to safely handle the recommended spillway design flood, thus leading to potential for loss of life and property destruction. Because the project cannot safely handle one-half the recommended PMF, it is considered unsafe until the recommended actions are accomplished.

This consultant has identified additional problems with the spillway and the abutments. Specifically, the concrete in the spillway is in very deteriorated condition. The basin walls, end sill, and baffle piers show extensive cracking and concrete spalling. Reinforcement steel is exposed in several places. The presence of large volumes of seepage from the left abutment area suggest that the seepage volume may be excessive for the existing seepage control system, particularly, if the pool elevation is increased.

Sink holes have been observed on the upstream side of the crest on the left abutment. The left abutment area has been blanketed with bentonite in the past. However, during the field investigation it was determined that this blanket has holes in it. The locations of these holes coincide with the locations of the sink holes observed at the surface.

There is no information in the DNRC files on strength parameters of either the foundation soils or embankment materials. A design stability analysis is not available. Some stability calculations were made during construction based on soil strengths interpolated from the landslide which occurred on the right abutment during construction. While these stability calculations are of value, they are inadequate and incomplete, based on the COE guidelines (Ref. 10).

C. SCOPE OF WORK

The scope of this investigation and report includes studying the stability and seepage questions for the existing structure and for the rehabilitation alternatives which have been conceptualized up to this time. It is not the intent of this report to present the hydrology or hydraulic analyses nor to respond to all of the recommendations presented in the COE Phase I Inspection Report (Ref. 11). A complete scope of work for this investigation was described in our report entitled, Technical Approach and Business Proposal Middle Creek Dam Rehabilitation Feasibility Study, Task 3, dated June 30, 1983.

It is also anticipated that additional repair alternatives may be generated as the entire project develops. Geotechnical analyses will be performed for these alternatives as appropriate.

D. REPORT FORM

The report which follows summarizes the methods of investigation, the findings, and the engineering analyses. It progresses in the same sequence in which the investigation was performed, beginning with a research of the existing information, moving then to the field investigation, followed by the laboratory investigation, and then the engineering analyses.

The engineering analyses included evaluating the field and laboratory data, then the seepage monitoring in preparation for seepage calculations. The seepage analysis was completed prior to performing stability analyses as the impact of the ground water is a major consideration in stability. After the static and dynamic stability was completed for the existing structure, rehabilitation alternatives were selected and analyzed. This report concludes with the geotechnical recommendations. The report is followed by a reference section arranged in alphabetical order.

III. DESIGN AND CONSTRUCTION HISTORY

Middle Creek Dam was designed by the Montana State Water Conservation Board (renamed, Montana Department of Natural Resources and Conservation). The design was supplemented with recommendations from a private consultant, Mr. Ralph E. Proctor. The Department retained Mr. R.P. England as the construction contractor. Work on the dam began in 1939 and stopped in June of 1941. Work resumed again in August 1946 and continued until the facility was completed in 1951. A record of the construction progress was kept and is on file with the DNRC (Ref. 22). The purpose of this section is to present the results of our review of the design and construction records relating to the identified inadequacies in the facility. This review was made prior to performing the field investigation to assist in the organization of overall study.

A. FOUNDATION TREATMENT

The records indicate that at least 32 exploration holes and 19 grout holes were made in the embankment area to explore and treat the foundation. Apparently, field permeability tests were not performed during the design investigation, however, the character of the foundation was identified.

A cutoff trench was planned to control underseepage. A portion of the cutoff trench was excavated, then inspected by an engineer from the U.S. Forest Service prior to backfilling. It is not known if the complete cutoff trench shown in the design drawings (Ref. 22) was constructed. Records indicate that the cutoff trench was completed on the left abutment at an average depth of about 10 feet. Apparently, the cutoff trench extended to a depth of near 50 feet at one location.

During the foundation preparation, a large pocket of loose fine sand (a glacio-fluvial deposit) was discovered under the proposed upstream slope on the right half (looking downstream) of the valley floor. About 19 of the exploration holes were specifically located in the area of this sand pocket to identify its boundaries. Instructions were given to the Contractor to excavate the sand to "glacial material". The horizontal limits and depth of this excavation are unknown. Approximately 47,000 cubic yards of sand was removed by hydraulic excavation and pumping. During this excavation, it was discovered that the fine sand deposit was overlain by the glacial till (recessional moraine) on the right abutment. Excavation of this deposit continued until the right abutment till slope had been partially undercut. During December 1939, after work stopped for the winter, a large land slide (about 70 feet high) occurred on the right abutment as a result of undercutting at the toe.

Mr. Ralph R. Proctor, a noted consulting engineer from California, was retained to review the impact of the slide on the embankment and to review the entire foundation conditions. The slide was apparently stabilized by excavating and removing portions of the slide; also, by cutting off the crown of the slide which caused the driving forces and replacing it as fill near the toe to increase the resisting forces. Apparently, much of the material involved in the slide was left in place and the embankment was built over it. The approximate limits of the slide excavation and corrective fill is indicated on Sheet No. 2 in Appendix A. In 1940, it was determined to move the centerline of the embankment downstream 38 feet to reduce the impact of the slide on the structure. According to correspondence on file (Ref. 22), Mr. Proctor recommended that the cutoff trench be constructed shallower than originally designed in the area of the valley floor and the right abutment. The final location and size of the cutoff trench in the valley floor and right abutment is unknown.

The information in the design file indicates that a grout curtain was constructed in the area of the valley floor on the upstream side of the embankment. The grout holes were apparently located in a double row on 20-25 foot centers. The rows were approximately 10 feet apart, creating an effective spacing of about 14 to 16 feet.

The curtain was apparently located along the proposed cutoff trench alignment. The holes varied from 40 to 140 feet deep and 8580 sacks of cement and 453 sacks of bentonite were pumped into this area. The effectiveness of this grout curtain is unknown.

B. DRAINAGE

Foundation drainage was constructed as shown on Sheet No. 2 in Appendix A. It is unknown if these drain pipes had been installed prior to the decision to move the centerline of the embankment downstream 38 feet.

Remedial work was accomplished in 1956 to control seepage in the left abutment. This consisted of a trench drain system located on the downstream side of the embankment and an impervious blanket on the upstream side of the embankment. These items are shown on Sheet No. 2 in Appendix A.

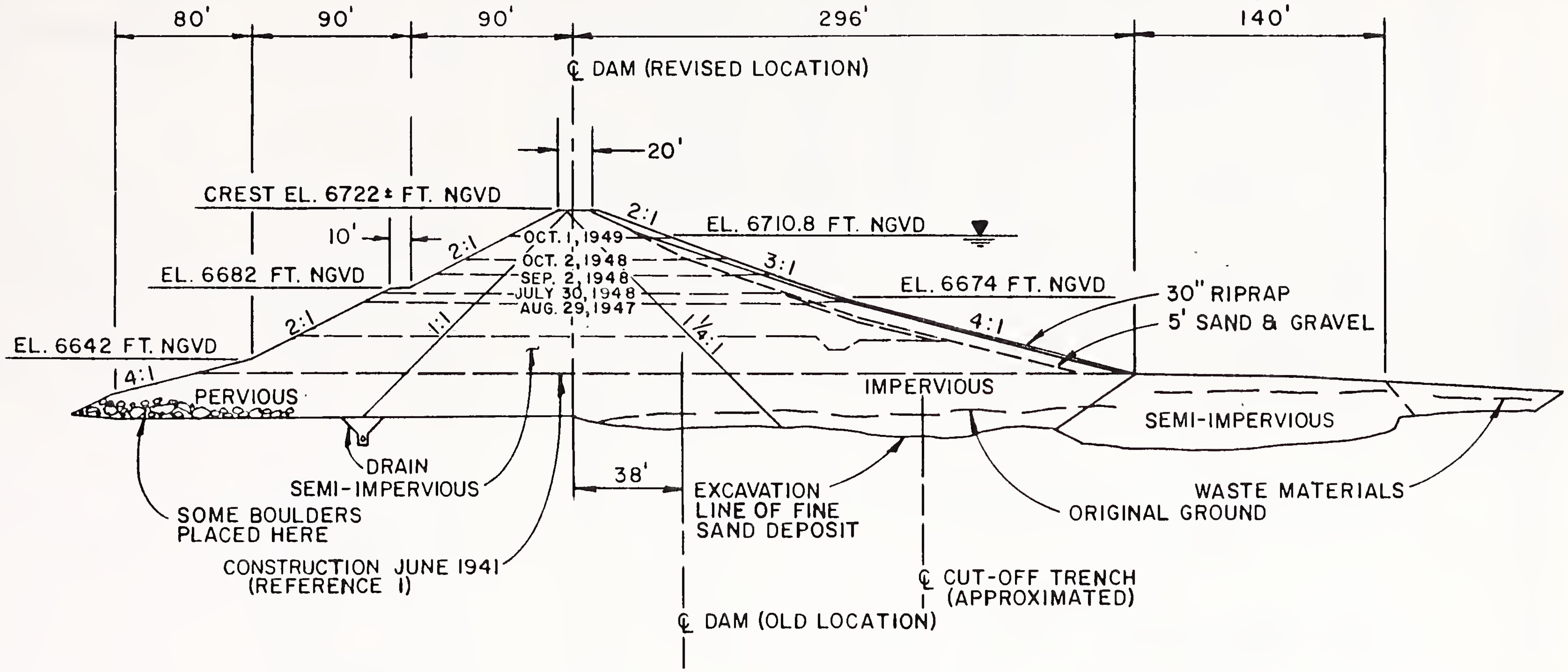
C. EMBANKMENT CONSTRUCTION

Figure 1 summarizes the construction progress by date and embankment configuration. The embankment was raised in horizontal lifts and compacted using sheepsfoot rollers.

Apparently, soil density tests were not performed during construction as no records of such tests are available. Soil moisture tests on the embankment are available in the design file (Ref. 22). A cross section of the embankment is represented on Sheet No. 3 in Appendix A.

Information from the Construction Progress Reports in the design files (Ref. 22) suggest that a rock fill was placed in the lower elevations of the previous fill section as depicted on Figure 1.

Four embankment profiles were surveyed for the purpose of establishing a cross section and to compare the existing embankment with the design drawings. Figure 2 shows the measured profile superimposed on the configuration presented on the design drawings. The existing embankment generally conforms to the design drawings. The measured crest is wider than the design crest. This appears to be a change from the design drawings. Because of this wider crest, it appears that the upstream slope was moved upstream just slightly. It is also possible that this is an overbuild. The crest appears slightly wider on the downstream side due to traffic and road maintenance grading.



NGVD : National Geodetic Vertical Datum

Source : Reference 22

MIDDLE CREEK DAM

DAM EMBANKMENT CONSTRUCTION HISTORY

(CUTOFF CONFIGURATION IS UNKNOWN)

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FIGURE 1

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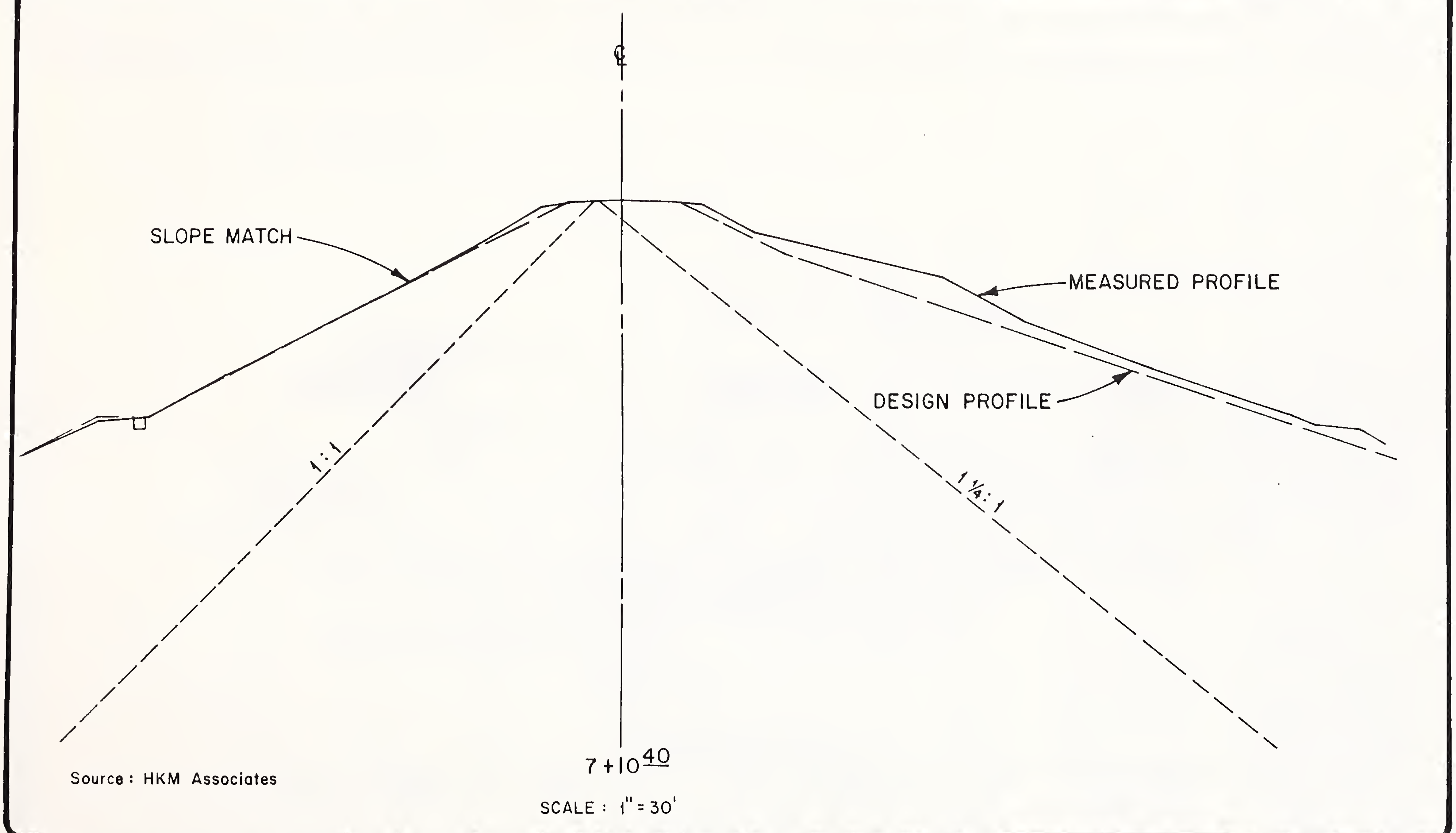

MIDDLE CREEK DAM

MEASURED VS. DESIGN PROFILE

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FIGURE 2

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IV. DESCRIPTION OF FIELD INVESTIGATION

A. INTRODUCTION

A field exploration program was performed during the summer and fall of 1983 for the purpose of identifying the existing soil profile and characteristics. This investigation consisted of exploration borings using a truck mounted drill rig, a geologic investigation, and a borrow investigation using a backhoe. In addition, concrete cores were cut out of the spillway floor. This section of the report summarizes the work performed during these field investigations. The findings during these field investigations are summarized in the subsequent sections.

B. DRILLING PROGRAM

1. Exploration Borings

The field drilling program included 15 exploration borings and 14 exploration test pits located as shown on Sheet Nos. 1 and 2 in Appendix A of this report. The locations and elevations of the borings were surveyed by HKM Associates. The test pit locations are approximated.

The drilling was performed by Rollins, Brown & Gunnell, Inc., Provo, Utah under the direction of an HKM professional engineer and professional geologist. The drill rig used was a truck mounted CME-55. The borings were advanced using an NX (2.97 inches O.D.) size rock bit and core barrel.

The borings were extended to depths of from 32.0 to 140 feet. The test pits were dug to depths of between 5 and 10 feet. Ground water levels were measured and soil samples were taken for field classification during the field drilling. Continuous logs of the holes were made and are detailed on the Logs of

Drill Holes, Sheet Nos. 4 and 5 in Appendix A and on the Logs of Test Pits, Sheet No. 6 in Appendix A.

Standard penetration resistance tests (N values on logs) were conducted in each of the exploration borings to obtain soil samples and to provide an indication of the relative density and strength of the subsoils. Several disturbed samples were also recovered from the drill cuttings. Undisturbed 3-inch diameter, thin-walled tube samples were recovered from the existing embankment fill and NX core samples were obtained in the foundation and abutment bedrock materials as the drilling progressed. The samples were carefully sealed in plastic to preserve their natural moisture content. The undisturbed tube samples were sealed and carefully boxed for transportation.

All samples recovered during the exploration program were then taken to HKM laboratories and inventoried. A few samples, selected for engineering strength tests, were sent to RBG laboratories in Provo, Utah for testing. The remainder of the samples were analyzed for physical characteristics at the HKM laboratory. Photos of the NX core samples are included in the Photo Booklet which was presented to the DNRC under a separate cover.

The informational goal of each drill hole is summarized in Table 1.

Table 1

Informational Goals of Exploration Borings

Drill Hole No.	Purpose
1	To investigate the foundation of the existing spillway and determine ease of excavation
2	To investigate the foundation of spillway and the character of the abutment
3	To investigate the right abutment
4	To determine the depth to bedrock and obtain undisturbed samples of the foundation; critical for the dynamic analysis
5	To investigate the pervious section of the dam and the foundation; critical for dynamic analysis
6	To investigate the semi-pervious section of the embankment and identify the phreatic surface; critical for the dynamic analysis
7	To investigate the impervious section of the embankment; critical for the static and dynamic stability analyses
8	To investigate the left abutment
9	To investigate the seepage in the left abutment
10	To investigate the left abutment
11	To investigate the potential auxiliary spillway area
12	To investigate soils in area of the potential auxiliary spillway
13	To investigate the old landslide in the right abutment
13A	To redrill DH-13 as it was lost due to difficult drilling
14	To investigate the right abutment at the contact with the downstream slope

Source: HKM Associates

2. Field Permeability Tests

During the subsurface investigation, in-place permeability tests were performed in the embankment soils, the overburden soils and in the bedrock. The purpose of these tests was to determine the consistency of the soil permeability rates and to establish criteria for seepage analyses. These tests were performed in accordance with procedures used by the U.S. Bureau of Reclamation Designation E-18 (Ref. 6). Water losses were measured, in gallons per minute, using constant pressure heads. The results of the permeability tests are presented on the Log of Drill Holes shown on Sheet Nos. 4 and 5 of Appendix A. The results of this information was used to calculate seepage quantities and hydraulic forces for the internal stress analysis.

3. Monitoring Tubes and Piezometers

Monitoring tubes were placed in each of the 15 drill holes to monitor piezometric ground water surfaces. The tubes were of 2 basic types; a standard slotted 3/4-inch diameter PVC pipe and a small diameter PVC tube with flow-through Casagrande piezometer tips. Each of the tubes were capped with screw-on caps. The monitoring tubes were placed in accordance with the method prescribed by the Bureau of Reclamation Designation E-28 (Ref. 6). Some of the drill holes have staged monitoring tubes which are installed to different depths. Monitoring of the water level in each of these tubes is being done on a regular basis, as this study progresses.

4. Slope Stability Monitoring Tube

A 2.75-inch diameter ribbed slope monitoring tube, 59 feet deep was installed in Drill Hole No. 5. The purpose of this tube is to monitor potential horizontal movements in the embankment.

Although horizontal movements are not a problem now, it is anticipated that future rehabilitation construction may necessitate monitoring slope movements.

The tube was obtained from The Slope Indicator Co., Seattle, Washington and installed as per their specifications. The tube was located near the center of the downstream slope. It is designed to be used with electronic slope indicator equipment. Results of the initial baseline reading for the slope inclinometer is included in Appendix B.

A secondary function of this tube was to measure the ground water levels. The lower end of the tube was left open to allow ground water monitoring. The accuracy of ground water measurements using this tube is questionable. Possible errors using this tube for water level measurements is discussed later in Section VIII.A.1.

5. Drill Pad Construction

Two drill pads were constructed for the purpose of access by the drill rig to the upstream and downstream slopes of the embankment.

One drilling pad was constructed on the upstream slope of the embankment for Drill Hole Nos. 7, 13, and 13A. This pad was constructed primarily of loose rock dumped on the existing slope. Some silty and sandy gravel was mixed with this loose rock to provide stability. This loose rock was purchased from Meridian Land and Mineral Company. The borrow location is NW1/4 of Section 25, Township 4 South, Range 6 East, Gallatin County, Montana.

The other drilling pad was constructed on the downstream slope for Drill Hole No. 4. This pad was cut out of the existing embankment with a dozer. Because the pad is located primarily on the portion of the downstream face with a 4:1 (horizontal to vertical) slope, the cut and fill zones of the pad are shallow, generally less than 2 feet.

Early in the summer of 1984, both drill pads will be reclaimed. The loose rock on the upstream face will be left on the upstream slope but it will be scattered up and down slope from the pad. This loose rock will provide additional riprap protection. The gravel portion of the pad will wash into the voids in the loose rock and disappear from view.

The pad on the downstream face will be regraded, using track mounted equipment, to conform with the original contour of the slope. This area will be reseeded in accordance with Forest Service recommendations.

6. Completion of Drill Holes and Monitoring Tubes

The depth of the drill holes and installed monitoring tubes are summarized in Table 2. This information is also summarized on the Logs of Drill Holes Sheet Nos. 4 and 5 of Appendix A.

7. Drill Hole Caps

Caps have been constructed for each of the drill holes. The method of construction is approximated in the sketch on Figure No. 3. The caps are buried below ground to reduce vandalism. The metal pin buried with the cap will enable location by metal detector if required. A steel post was also placed in the ground adjacent to the drill holes at distances varying from about 5 to 6 feet to facilitate locating the holes. An exception to this procedure was warranted at Drill Hole Nos. 7, 13 and 13A where fence posts were not placed to avoid accumulations of floating debris.

Table 2

Drill Hole Completion

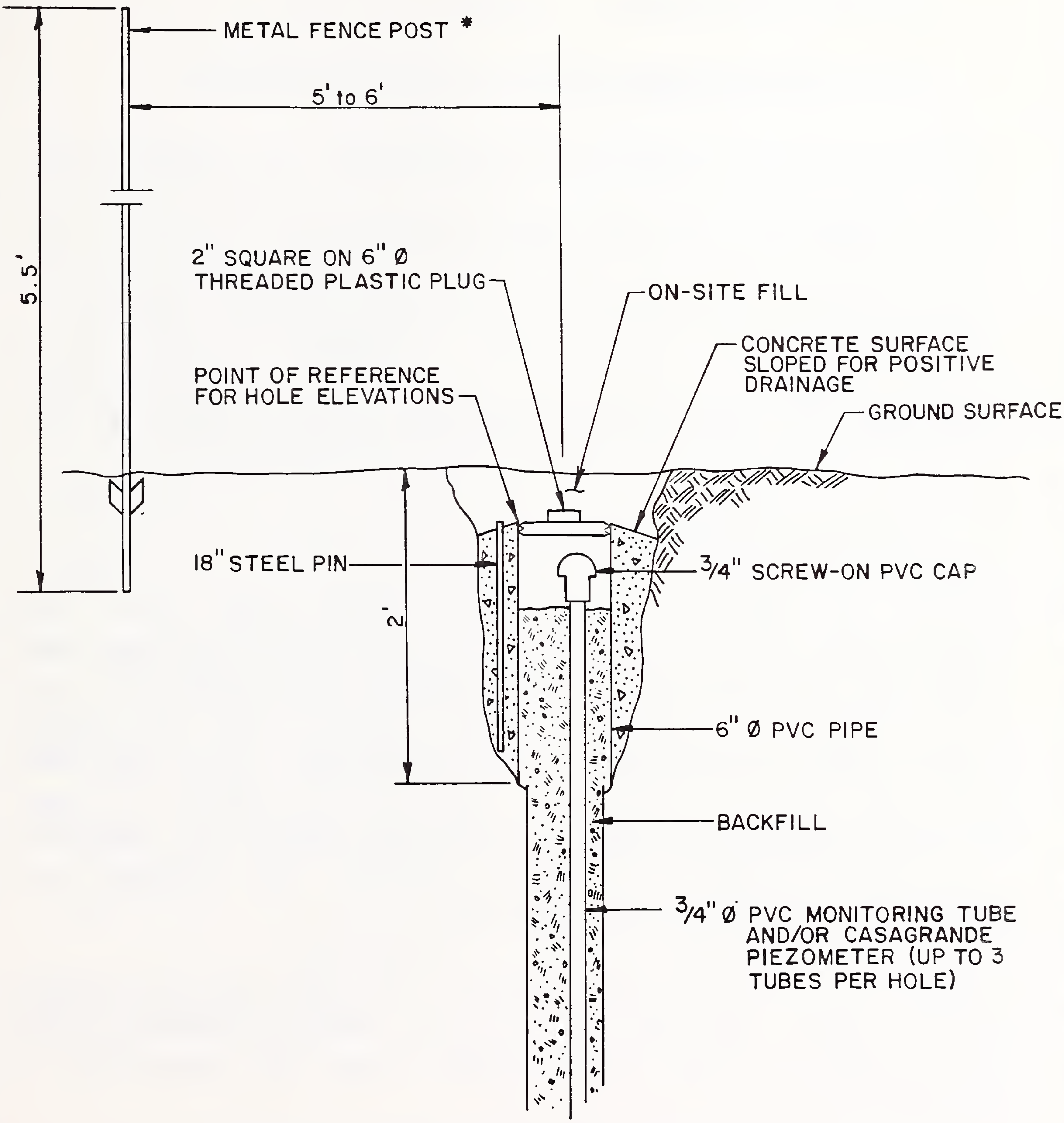
Drill Hole	Hole Depth (ft)	Perforated PVC Monitoring Tubing (ft)	Casagrande Piezometer (ft)	Description
1	32	29		Perforated below 19'
2	42	42		Perforated below 22' and backfilled with pea gravel.
3	85.5	37 & 78		Perforations at 27 to 37' and 68 to 78', respectively. Backfilled from 2 to 30' with drill cutting 30 to 38' with pea gravel, 38 to 40' bentonite plug, 40 to 62' with drilling cuttings, 62 to 66' bentonite plug, and 66 to 78' with pea gravel.
4	140	15.5, 57 & 140		Perforations at 10.5 to 15.5', 47 to 57', and 120 to 140', respectively. Bentonite plugs are located at 16 to 18', 60 to 64' and 82 to 85'. All other areas are backfilled with pea gravel.
5	88.5			A slope inclinometer tube is placed to a depth of 59'. The hole is backfilled to 60' with pea gravel, at 60 to 65' with a bentonite plug and with hole cave below 65'.
6	140	125	82.5	The monitoring tubing is perforated in the bottom 30'. The hole is backfilled from 2 to 80' with the drill cuttings, 80 to 85' with pea gravel, 85 to 90' with a bentonite plug and below 90' with drill cuttings.
7	136	136	60	The monitoring tubing is perforated below 116'. The hole is backfilled with drill cutting to 58', 58 to 62' with pea gravel, 62 to 65' with bentonite, 65 to 110' with drill cuttings, 110 to 136' with pea gravel. Both tubes were pulled apart at a joint near 40 feet during installation then repaired. They appear to be functioning properly.

Table 2 (continued)

Drill Hole Completion

Drill Hole	Hole Depth (ft)	Perforated PVC Monitoring Tubing (ft)	Casagrande Piezometer (ft)	Description
8	80	78		The monitoring tube is perforated below 68'. The hole is backfilled with drill cutting down to 15', then 15 to 20' with bentonite, 20 to 68' with drill cutting, and 68 to 78' with pea gravel.
9	60	15 & 48		The monitoring tubes are perforated from 10 to 15' and 20 to 48', respectively. The hole is backfilled from 2 to 15' with pea gravel, 15 to 17' with a bentonite plug and 17 to 48' with pea gravel.
10	111	29 & 93		The monitoring tubes are perforated at 19 to 29' and 73 to 93', respectively. The hole was backfilled from 2 to 30' with pea gravel, 30 to 32' with a bentonite plug and 32 to 93' with pea gravel.
11	37	30		The tubing is perforated below 20'. The hole is backfilled with drill cutting.
12	35.5	34		The tubing is perforated below 24'. The hole is backfilled with drill cuttings.
13	67	55		The tubing is perforated below 50'. The hole is backfilled with pea gravel from 0 to 46', 46 to 48' with a bentonite plug and 48 to 56' with pea gravel.
13A	82	75		The tubing is perforated below 55'. The hole is backfilled from 2 to 50' with pea gravel, 50 to 55' with a bentonite plug, and 55 to 75' with pea gravel.
14	81	70	38	The monitoring tube is perforated from 50 to 70'. The hole is backfilled from 2 to 38' with pea gravel, 38 to 41' with a bentonite plug an 41 to 70' with pea gravel.

Source: HKM Associates. Also see Sheet Nos. 4 and 5 in Appendix A.



* The metal fence posts are located south of the monitoring tubes except at Drill Hole No. 2 where it is east of the tube and at Drill Hole Nos. 13 and 13A where no fence posts were installed.

Source: HKM Associates

MIDDLE CREEK DAM

DRILL HOLE CAP CONSTRUCTION

8M087.113 FEBRUARY 1984

FIGURE 3

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C. TEST PIT EXPLORATION

1. Materials Investigation

A materials investigation, using a backhoe, was performed after the field drilling program. As previously mentioned, fourteen (14) test pits were made at the location indicated on Sheet Nos. 1 and 2 of Appendix A. A primary purpose of this investigation was to identify potential borrow sources for repair work on the embankments. Test Pits were also used to observe embankment materials, the sink hole area on the left abutment, and to obtain soil samples. The informational goal of each exploration test pit is summarized in Table 3.

2. Sink Hole and Drain Trench

During the field investigation, several sink holes were observed on the left abutment. An investigation of these sink holes was made by test pit excavation and by pumping water into the sink area to attempt to connect them to the high seepage area on the downstream side of the embankment. Four large test pits were made in the area as approximately located on Sheet No. 2 in Appendix A. Only one test pit (TP-104) was logged as each hole revealed a similar lithology. The findings are described later in Section VIII.B.1.

Test Pit 114 (TP-114) was made in the area of the existing trench drain. This was done in an attempt to determine the effectiveness of the trench drain which was described in Section III.B.

Table 3

Informational Goals of Exploration Test Pits

Test Pits	Purpose
101	To investigate the ease of excavation of the principal spillway
102	To obtain a bulk sample of the pervious section of the embankment and to observe seepage
103	To obtain a bulk sample of the impervious section of the embankment and to investigate the old landslide area
104	To investigate the upstream impervious blanket on the left abutment
105	To investigate the ease of excavation of the potential auxiliary spillway area
106, 107 & 111	To obtain a bulk sample of the borrow from the potential auxiliary spillway excavation and observe seepage
108-110	To obtain samples of the borrow areas on the east side of the reservoir
112-113	To investigate the potential borrow area on the mountain side above the right abutment
114	To investigate the area of the trench drain on the left abutment downstream from the embankment

Source: HKM Associates

D. SPILLWAY FLOOR INVESTIGATION

Three concrete cores were cut out of the existing spillway for the purpose of evaluating the integrity of the concrete. Soundings, using a large impact hammer, were made prior to coring the concrete to search for voids, thin or low quality concrete. The results of the soundings did not indicate any relative differences in the concrete condition. Therefore, random locations were selected for the cores. These locations were limited to the spillway floor.

The concrete was tested in compression. Results indicate a compressive strength of 5930 pounds per square inch which is indicative of high quality concrete. The concrete thickness and reinforcing steel appears to be as depicted in the design drawings (Ref. 22).

Exploration borings using a hand auger were attempted in each of the holes made in the spillway floor. The foundation soil is a well compacted clayey gravel. The hand auger holes could not penetrate this material more than a few inches. No voids were encountered under the floor slab.

V. ENGINEERING GEOLOGY AND SEISMICITY

A. SETTING

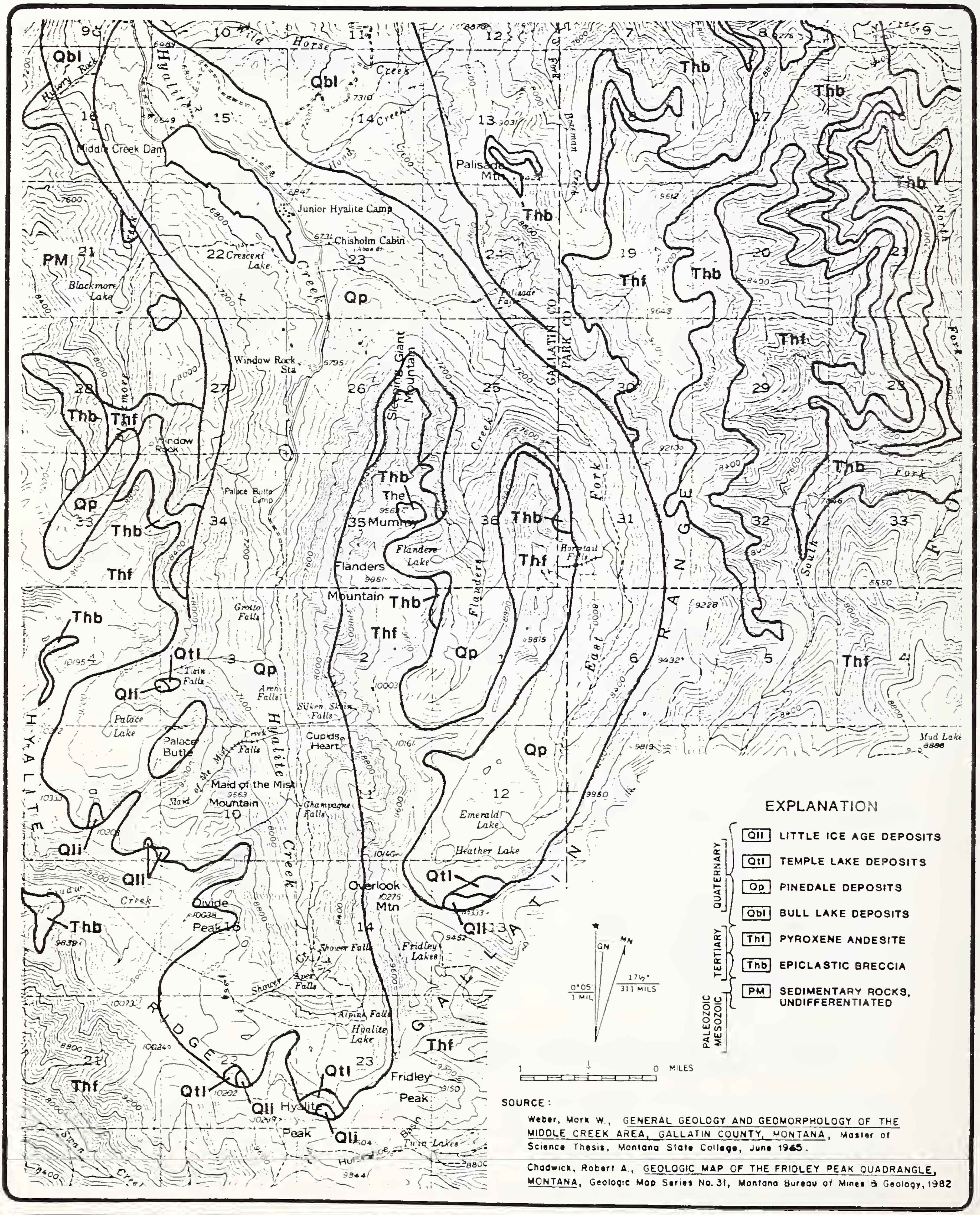
Middle Creek drains a portion of the northern end of the Gallatin Range, flowing generally north to its confluence with the Gallatin River. The mountainous portion of the Middle Creek drainage, the area in which the dam and reservoir are located, consists of a rugged, tree covered topography ranging in elevation from 5440 feet at the range front to 10,300 feet along the crest of the Gallatin Range. Middle Creek Dam and Reservoir is located in the lower portion of the glaciated section of the Middle Creek drainage.

The Gallatin Range, together with the Madison Range to the west, forms a continuous structural block in which Precambrian metamorphic basement rocks are overlain in places by infolded Paleozoic and Mesozoic sedimentary rocks. Eocene volcanic rocks unconformably overlie these older rocks in the Gallatin Range portion of the block and dip gently eastward toward the Yellowstone Valley (Ref. 7).

The general structural pattern of the area is that of south-dipping Paleozoic and Mesozoic sedimentary rocks deformed by northwest and northeast trending faults and folds (Ref. 32). No major faults are noted in the dam and reservoir area (Ref. 20). There are, however, numerous major faults within the region, the nature of which will be discussed later in Section V.C.

B. LOCAL GEOLOGY AND STRATIGRAPHY

Figure 4 is a geologic and geomorphic map of Middle Creek Reservoir and surrounding area that was adapted from the available geologic literature (Refs. 7 and 32). The topography



MIDDLE CREEK DAM

GEOMORPHIC AND GEOLOGIC MAP

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FIGURE 4

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of the area is the result of the actions of water and wind, intensely modified by the effects of Pliestocene and Neoglacial glaciation (Ref. 32). Assuming the volcanic sequence covered the Middle Creek area (as evidenced by the presence of volcanic outliers), at least 4000 feet of erosion in the drainage has occurred since Eocene time. The stratigraphy in the reservoir area is typified by shallow Quaternary glacial, glacio-fluvial and recent slopewash, colluvial and alluvial deposits overlying northeast dipping Eocene and Mesozoic sedimentary rocks. The upper valley walls are formed of about 1200 feet of northeast dipping Paleozoic rocks capped by up to 2000 feet of Tertiary volcanics and volcanic breccias. Toward the head of the Middle Creek/Hyalite drainage, the total depth of the volcanic pile approaches 4000 feet (Ref. 20).

Glacial, glacio-fluvial, and recent alluvial, slopewash and colluvial deposits form the valley floor and foundation in the dam and reservoir area. These deposits in turn overlie a northeast dipping, soft, white to tan, thin-bedded, micaceous and carbonaceous siltstone and/or claystone. Several similar outcrops of gray to brown, thin-bedded, micaceous, carbonaceous siltstone were mapped in the Garnet Mountain Quadrangle (Ref. 20). These siltstones were tentatively assigned a late early Eocene age on the basis of spore and pollen analyses. An outcrop of a similar white to tan siltstone and/or claystone immediately underlying the volcanic sequence was noted on Antelope Butte south of Livingston, Montana. This outcrop is believed to correlate with the siltstone and/or claystone noted in the Garnet Mountain Quadrangle and underlying the Middle Creek Dam.

The glacial deposits form most of the area underneath and along the sides of the dam and reservoir and consist mainly of terminal, lateral and ground moraines varying from a few feet to more than 100 feet in thickness. The glacial deposits in the valley floor are the result of two Wisconsin glacial advances. There is evidence of an extensive pre-Wisconsin, Marble Point, glaciation in the Middle Creek area (Ref. 32). However, no evidence of this glaciation is present in the area of interest. In fact, it appears that the valley floor may have been up to 600 feet higher than its present level during Marble Point time (Ref. 32). Of the two Wisconsin glaciations, the oldest, Bull Lake, was the most extensive. The later Wisconsin glaciation, Pinedale, was not as extensive as Bull Lake in the Middle Creek area and the limits of Pinedale deposits are totally within the limits of Bull Lake deposits. There is evidence for Neoglaciation in the upper Middle Creek/Hyalite drainage (Ref. 32). However, any post-Pinedale glaciation did not advance beyond the cirques in the uppermost portions of the drainage. The glacial moraine deposits are characterized by their heterogeneous, unsorted, and unstratified nature. In the dam and reservoir area, the glacial moraine deposits are of Pinedale age and are a compact gravelly clay to clayey gravel with scattered large erratics.

Glacio-fluvial deposits consist of ice-contact and outwash sediments. The ice contact deposits consist of a gravelly sand to clayey sand and are interfingered with the till as encountered in Drill Hole Nos. 7, 13 and 13A. During construction of the dam, saturated ice-contact deposits at this location washed out and caused collapse of the overlying till, as discussed in Section III.A. The outwash deposits consist of a coarse sandy, gravelly, cobble conglomerate and underlie and overlie the till; in turn overlaying the siltstone/claystone. The outwash deposits were encountered during this investigation in nearly every drill hole and are represented as a conglomerate in Sheet No. 2 or 7 in Appendix A.

The recent alluvial deposits are present in the center of Middle Creek valley, as well as in the bottom of adjoining tributaries, and are characteristically a sandy gravel and cobble. The recent alluvial deposits are gradational upward from the outwash deposits and, during this study, differentiation of the two deposits was arbitrarily made based upon coarseness. The underlying outwash deposits were determined as being the coarser of the two fractions.

Slopewash and colluvial deposits consist of heterogeneous deposits formed along the valley walls as a result of downslope movement, chiefly due to the action of gravity but assisted by water. The slopewash and colluvial deposits are present along the sides and center of the valley. These deposits are generally well developed throughout the area below the dam and are not distinguished separately from the parent materials on the geology and geomorphic map in Figure 4. A slope stability investigation of the reservoir area was completed as a part of this investigation and is discussed later in Section V.D.

C. SEISMICITY

The Middle Creek Dam is located on the eastern border of the Three Forks and Madison-Hebgen seismic regions within the Intermountain Seismic Belt (Ref. 24). The dam is within Zone 3 (major damage potential) and on the edge of Zone 4 (great damage potential) on the Seismic Risks Map of the U.S.A. presented on Sheet 7 of 7 in Appendix A (Ref. 12). The seismicity of the area was investigated by researching available geologic information to determine fault activity and obtaining a history of earthquakes in the region. A maximum credible earthquake (MCE) for the site was developed.

1. Fault Activity

Assuming a MCE magnitude of 7.0, a near field radius of 25 miles was determined for the investigation (Ref. 18). For the far field motion, the Hebgen earthquake, originating on the

Hebgen fault located about 48 miles south of the site, was chosen. The Hebgen event was a magnitude 7.1 earthquake and the most severe recorded earthquake in Montana.

Table 4 presents a tabulation of significant faults or fault systems within 30 miles of Middle Creek Dam and Reservoir. Recent activity, as detailed in the various sources referenced, is also tabulated. The only fault within the near field, which exhibits definite evidence of activity within the Halocene Period, is the Deep Creek-Luccock Park Fault complex (also named Emigrant Fault in older sources). The Central Park Fault may have moved in the Halocene Period, however, no conclusive evidence is present. The Deep Creek-Luccock Park Fault complex is considered as the fault most critical to the Middle Creek Dam due to both distance and fault magnitude.

2. Historic Seismicity

A history of earthquakes in the region has been obtained from the National Geophysical and Solar Terrestrial Data Center in Boulder, Colorado. A history of 839 events (which have occurred within about 200 miles of the reservoir) have been computer tabulated. In addition, the October 28, 1983 Challis, Idaho earthquake with a magnitude of 6.9 on the Richter Scale and the three main aftershocks have been added. A summary of these events is presented on Sheet No. 7 of 7 in Appendix A. Several of the recorded earthquakes overlap each other and cannot be counted individually by visual observation of the sheet. It was determined that of the 843 events only two had magnitudes on the Richter Scale in the range of 7 - 8. Thirteen (13) had magnitudes in the range of 6 - 7, and 99 had magnitudes in the range of 4.5 - 6. Figure 5 is presented for clarification of the approximate relationship between earthquake intensity, acceleration, and magnitude.

Table 4

Significant Faults Within 30 Miles of Middle Creek Dam and Reservoir

Fault Name	Distance From Dam (mi(km))	Approx. Fault Length (mi(km))	Activity
Bridger	4(7)	40(67)	No evidence of post-Pliocene movement
Gallatin Range	8(13)	20(33)	Major movement ceased in late Eocene (Ref. 25)
Squaw Creek	6(10)	25(42)	No activity since mid-Eocene (Ref. 20)
Central Park	28(47)	15(25)	Evidence of early to middle Pliestocene movement (Ref. 14)
Spanish Peaks	18(30)	90(150)	No activity since mid-Eocene (Ref. 20)
Deep Creek-Luccock Park	15(25)	35(58)	There is evidence of post-Pinedale movement along the Deep Creek Fault (Ref. 21). There is also evidence of post-Bull Lake pre-Pinedale movement along the Luccock Park Fault (Ref. 16)

Sources: HKM Associates and references tabulated above

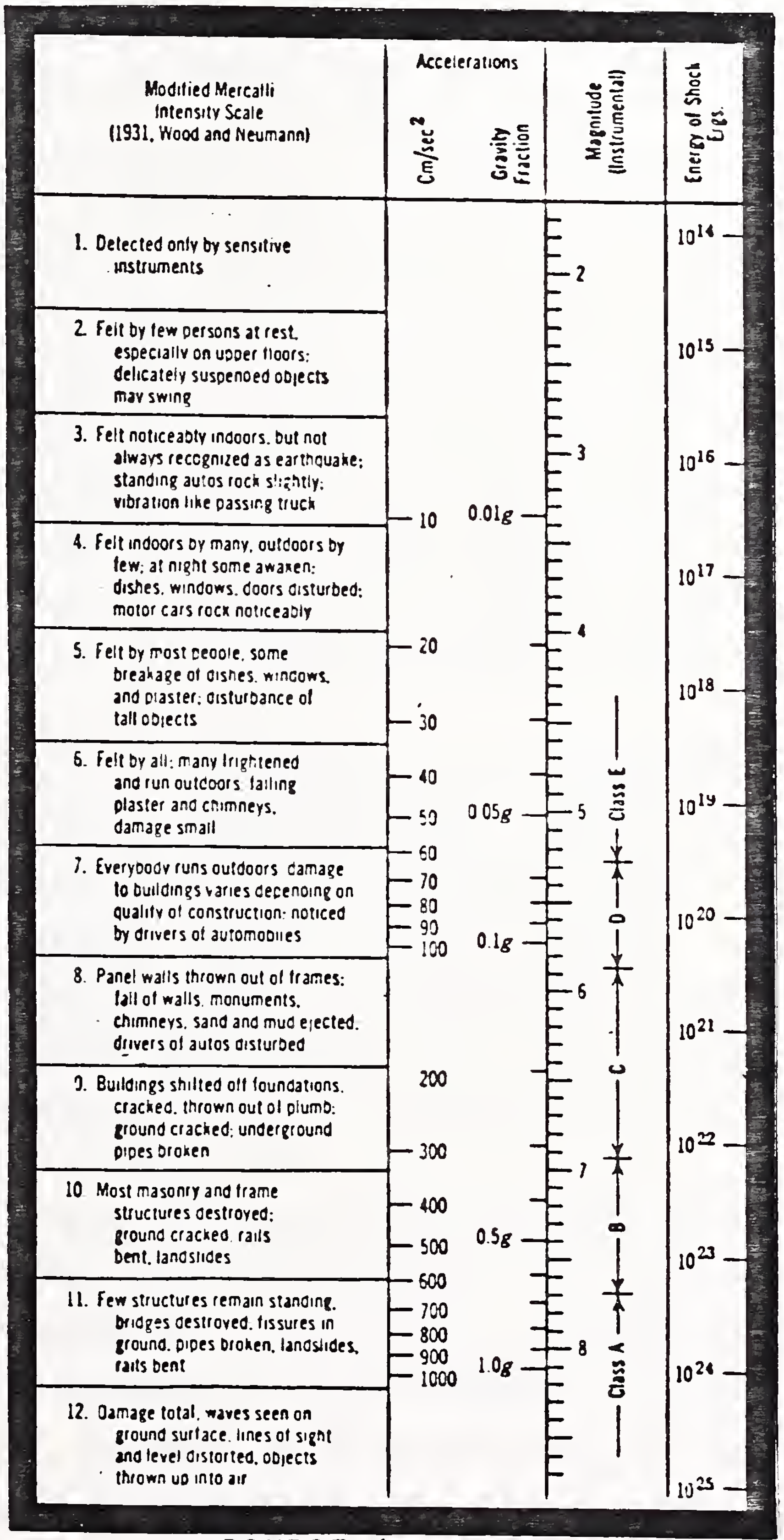
Middle Creek Reservoir is located in a relatively active seismic zone with the majority of the seismic events occurring in the southwestern Montana-Yellowstone Park area. Since 1825, Montana has experienced five shocks that reached the intensity of 8 or greater (Modified Mercalli Scale). The epicenters occurred in the Three Forks-Clarkston Basin area, approximately 40 miles northwest of the site, and in the Madison-Hebgen area, approximately 48 miles to the south. Numerous other shocks of intensity 4 and greater have been reported within a 150-mile radius of the site.

3. Design Earthquake

The Hebgen earthquake of magnitude 7.1 at a distance of 48 miles from the site was selected as the far field motion. The Deep Creek-Luccock Park Fault complex is potentially active and was selected as the near field motion. Earthquake event parameters were developed for the Deep Creek-Luccock Park Fault as follows:

1. Selected Design Earthquake Magnitude = 6.5. Applicable earthquake magnitudes range from 6.0 to 7.0. Upper and lower magnitude values were obtained based on the methods proposed by Bonnilla (Ref. 3) assuming fault rupture along the entire fault, which is extremely unlikely, and fault rupture along one-half the length of the fault. During the Hebgen earthquake of 1959, fault rupture occurred along about one-half of the Hebgen Fault. Fault displacement values were based upon fault type and length. A median magnitude value of 6.5 was selected as the design value for the Deep Creek-Luccock Park Fault complex.

2. Intensity IX. Intensity was developed in relation to magnitude as proposed in Krinitesky and Marcuson (Ref. 18) and as correlated in Figure 5.



SOURCE: (REF. 29)

MIDDLE CREEK DAM
APPROXIMATE RELATIONSHIP, EARTHQUAKE INTENSITY, ACCELERATION AND MAGNITUDE

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FIGURE 5

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3. Acceleration Design Value = 0.22g. Acceleration values ranged from 0.09 to 0.21g. Upper and lower limits for acceleration were developed using curves for earthquake motion developed by Krinitesky and Marcuson (Ref. 18) based on about 600 strong motion records. The lower limit was chosen based upon the mean data curve, while the upper limit was chosen based on the mean plus one standard deviation curve, or 84 percentile curve. Both upper and lower values were chosen for a near field hard site. The 0.22g acceleration value for the site area was developed by Algermissen and Perkins (Ref. 1) as the 90 percent probability value of not being exceeded in 50 years. As this value is higher than the calculated value, it was selected as the design acceleration value for the Deep Creek-Luccock Park Fault complex.

4. Selected Duration = 16 Seconds. The range for the earthquake duration was determined to be 3 to 16 seconds based upon curves developed by Krinitesky and Marcuson (Ref. 18). The upper limit value was developed conservatively based upon a near field soft site. The duration value of 16 seconds seems reasonable as Montana earthquakes tend to attenuate quickly (Ref. 24).

A magnitude 7.1 earthquake at the Hebgen fault would develop acceleration values of 0.06g to 0.10g with a significant duration of 5 seconds to 16 seconds, based upon the projection methods used above. These values compare favorably to the 0.05g seismograph value obtained at Bozeman during the Hebgen event. As a result, the projected Deep Creek-Luccock Park event was selected as the MCE or design earthquake for Middle Creek Dam.

D. RESERVOIR BANK STABILITY

A slope stability investigation of the reservoir area was undertaken to evaluate areas of possible unstable banks. The study consisted of a reconnaissance of the reservoir slopes, to determine if any current instability exists, and the measurement of slope angles, to determine areas of oversteepened slopes. Slope angles were measured using a Brunton compass. Sheet No. 1 of Appendix A presents the results of the study. At present, the slopes surrounding the reservoir area are stable. Unstable slopes are in evidence outside the reservoir area in the unconsolidated glacial deposits. The unstable areas outside the reservoir area are generally saturated and undrained. No saturated and undrained areas were evident along the reservoir walls at the time of this investigation.

The reservoir slopes were divided into three categories: slopes steeper than 25 degrees; slopes between 15 degrees and 25 degrees; and slopes less than 15 degrees. Based on visual observations, Brunton compass measurements, and the results of the determination of the angle of internal friction of the soils by laboratory tests, the natural angle of repose of the glacial materials forming the reservoir walls is estimated in the range of 25 degrees to 30 degrees. Consequently, slopes in excess of 25 degrees are estimated to be near or exceeding the natural angle of repose. Presently, stability is provided by vegetation; assisted by a coarse gravel matrix, and low soil moisture contents. Any changes in the present conditions of the slope may cause instability.

Slopes in the range of 15 degrees to 25 degrees are less than or approach the natural angle of repose. A triggering mechanism, such as an earthquake or rapid dewatering, may induce slope movements.

Slopes less than 15 degrees are at less than the natural angle of repose. There is minimal chance of instability on these slopes.

In conclusion, the reservoir walls appear to be stable in their present condition. Future instability is not expected to occur unless the slopes are denuded of vegetation, the soil moisture is increased significantly, and/or a triggering mechanism occurs. It should be noted that no reservoir bank instability was observed during the dewatering of the reservoir in Fall, 1983.

VI. LABORATORY ANALYSIS

Representative field samples were selected for laboratory testing after careful visual examination of the soil and consideration of the design criteria. The physical soils property tests were performed in the HKM laboratory and the engineering property tests were performed by RBG in Provo, Utah.

Test	Purpose of Test
Natural Moisture Content (ASTM D-2216)	To determine the natural (in situ) water content and to correlate the moisture contents with the phreatic surface.
Atterberg Limits (ASTM D-423 & D-424)	To provide an indicator of the shear strength and compressibility of the soil.
Particle-Size Distribution (ASTM D-422)	To determine the grain sizes of the soils for classification and identification of physical characteristics.
Natural Unit Density (ASTM D-2937)	To determine the (in situ) dry unit weight of the soil.
Unconfined Compression (ASTM D-2938)	To determine the shear strength parameters of the bedrock.
Moisture - Density Curve (ASTM D-698)	To determine the relationship of water to the density of soil during a compaction or remolding process.

Test	Purpose of Test
Direct Shear (ASTM D-3080)	To determine the consolidated drained shear strength of the soils.
Triaxial Shear	To determine the consolidated undrained and drained shear strength of soils.
Consolidation (ASTM 2435)	To define the compressibility characteristics of the soil.
Concrete Compression	To determine the compressive strength of concrete.

The (ASTM C-39) laboratory tests were performed in strict accordance with applicable ASTM procedures. The triaxial tests are not specified by ASTM, however, these tests were performed in accordance with the current state-of-the-art (Ref. 4). A Summary of the Laboratory Test Results is presented on Sheet Nos. 4, 5 and 6 of Appendix A. Additional test data for individual tests is detailed on Plate Nos. 1-27 of Appendix A.

VII. SOIL CHARACTERISTICS

A. EMBANKMENT SOILS

The embankment configuration consists of an upstream impervious zone (soil type 1), a center semi-pervious zone (soil type 2) and a downstream pervious zone (soil type 3). Each zone composes approximately one-third of the total cross section. A cross section of the embankment is shown on Sheet No. 3 of Appendix A.

Detailed descriptions of the soils are also presented on the logs presented on Sheet Nos. 4, 5 and 6 of Appendix A. The results of the field and laboratory tests are plotted adjacent to the drill hole and test pit logs corresponding to the depth to which the test results are applicable. Two sets of standard penetration resistance (N) values have been presented on the logs, N and N'. N values referred to the actual values obtained during the field tests. N' is the corrected standard penetration resistance value. The correction is for effective overburden pressure (Ref. 28) and spoon size. A spoon size reduction factor of .78 was used where a large diameter drive spoon (2.5-inch inside diameter) was substituted for the standard (2-inch inside diameter) spoon in order to obtain larger cohesionless soil samples.

A reference of the laboratory soils test results for the soil types encountered at the embankment, potential spillway and borrow areas is presented on Table 5.

Table 5

Reference of Laboratory Soils Test Results

Soil Type	Zone/Material	Plate Nos. (in Appendix A)
1	Impervious	3, 6, 13, 17, 25
2	Semi-pervious	4, 25
3	Pervious	5, 14, 16
4	Sand Foundation	7, 8
5	Foundation	9,26
6	Bedrock	2
-	Right Abutment	1, 9, 15, 27
-	Left Abutment	10, 11, 18
-	Potential Auxiliary	
-	Spillway	12, 19, 20, 24
-	Borrow Areas	12, 21, 22, 23

Source: HKM Associates

1. Impervious Zone

The material in the impervious zone is typically a clayey gravel (GC) with discontinuous lifts of sand interbedded. Corrected standard penetration resistance (N') values ranged from 22 to over 100 blows per foot with a weighted average of 34. The average was weighted by rejecting all tests with less than 1 foot of penetration. The dry unit weight averages about 110 pounds per cubic foot with about 17 percent moisture when fully saturated. The consolidated undrained internal angle of friction is 33.5 degrees with a cohesion value of 3.5 pounds per square inch. Field permeability tests in this material averaged about 37 feet per year.

The following classifications are generally used, particularly by the U.S. Bureau of Reclamation (Ref 5), to describe the permeability of soils:

Permeability in Feet Per Year	Description
less than one	impervious
1-100	semi-pervious
greater than 100	pervious

These values may be used to evaluate the field results shown on the logs on Sheet Nos. 4 and 5. However, by these standards, the impervious zone at Middle Creek is actually a semi-pervious zone. For the purpose of this report the word description "impervious" will continue to be used in reference to the upstream zone of the embankment.

2. Semi-Pervious Zone

This zone is a gravel material similar to the impervious zone. The gravel is clayey nearer the crest of the embankment and becomes silty with depth. The clay content also decreases with depth. N' values averaged about 33 blows per foot and the field permeability tests averaged 34 feet per year. While these engineering characteristics are similar to the impervious zone, the material is actually more granular than the impervious zone. It has a higher internal angle of friction with little or no cohesive strength.

3. Pervious Zone

This zone is sandy gravel material with N' values ranging from 33 to over 100 blows per foot. The average N' value is 56 blows per foot. The average is weighed by averaging only N' values with a full 1 foot of penetration. This material has an estimated relative density of 72 percent. Field permeability values ranged from 47 to 820 feet per year and averaged about 270.

Test Pit No. 102 was made in this material for the purpose of in-place visual observation and density testing. A field density test performed using a nuclear densometer indicated the in-place dry density is about 114 pounds per cubic foot.

4. Riprap

The riprap on the face of the dam is a well indurated andesite and basalt. It is loose rock with an estimated average size of 8 inches in diameter. It is approximately 2.5 feet thick and it is underlain by a sandy and silty gravel riprap bedding or drain fill. The design drawings indicate that the bedding is 18 inches thick.

The riprap has slid down the slope, away from the crest, in some places, however, the riprap protection appears adequate. At some locations the riprap appears considerably thicker than 2.5 feet. A general profile of the riprap face is shown on Sheet 3 in Appendix A and on Figure 2.

B. NATURAL IN-PLACE MATERIALS

Laboratory soils tests on the in-place materials at the site were referenced previously in Table 5 in Section VII.A. The natural soils in the abutment, foundation and borrow soils will be described in this section.

1. Abutments

A generalized geologic profile section of the embankments and abutments is presented on Sheet No. 2 of Appendix A. The abutments are composed of glacial till with a variety of soil classifications and engineering characteristics. Descriptions of these soils are given on the logs of the drill holes referenced below and shown on Sheet Nos. 5 and 6 in Appendix A.

Abutment	Drill Hole (DH)
Left	10 (below 14 feet), 8 & 9
Right	1 (below 10 feet)
	2 (below 4 feet)
	3 (below 20 feet)
	13 (below 16 feet)
	13A (below 12 feet)
	14 (below 21 feet)

Permeability rates in the left abutment are very high. Values range from 13 to 82,000 feet per year and average 8085. This average is weighted by rejecting the highest value.

The permeability of the right abutment is much lower than the left abutment. The permeability of the right abutment averages 1012 feet per year. Both abutments appear to be stable.

2. Sand Deposit in Foundation

A glacio-fluvial deposit of predominately sand with some silt and clay was encountered underlying the embankment under the upstream slope. This deposit was discovered during the construction of the dam and apparently most of this material was excavated at that time. A discussion of the foundation treatment in this area is included in Section III.A.

Several standard penetration tests (N values) were made in this deposit in Drill Hole No. 7 at a depth of 71.0 to 90.0 feet. The N' values ranged from 29 to 38 blows per foot and averaged 34. Test results for a silt from this deposit at DH-7 indicates a dry unit weight of 87 pounds per square foot and saturated moisture content of 32.8 percent. This sample was also determined to have an internal friction angle of 28.3° and a cohesion value of 5 pounds per square inch. The amount of this material passing the No. 200 sieve ranges from 19% in the sand to 98% in the silt.

3. Foundation

The foundation soils, other than the sand deposit previously described, consist of a thin granular deposit underlain by well consolidated glacial till. Typically, the till is classified as a clayey gravel. N' values in this material ranged from 25 to 122 blows per foot and averaged about 47 blows per foot. This till has high shear strength and moderate to low compressibility.

4. Bedrock

Bedrock was encountered in Drill Hole Nos. 7 and 4. It is typically a thin layer (4 to 6 feet) of sandstone underlain by interbedded claystone and siltstone shale. This material is highly fractured to a depth of about 120 feet. Field permeability rates varied from 3 to 700 feet per year and averaged 147. The rock quality designation (RQD) is fair. The permeability rate, the RQD and the core recovery percentage are shown on the logs on Sheet Nos. 5 and 6 in Appendix A.

The RQD is a method of estimating the in situ rock quality. This relationship is presented in Table 6.

Table 6
Rock Quality

RQD (%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

Source: See Reference 5

5. Proposed Spillway Area

Test Pit Nos. 105-107 and 111 were made in the area of the proposed auxiliary spillway. It was determined from this investigation that the area can be excavated by conventional method. The material in the area is a gravel with sand and silt seams. Thick organic deposits are evident in the area of Test Pit 107.

VIII. SEEPAGE

A. MONITORING

1. Ground Water Monitoring in Drill Holes

A short term monitoring program has been conducted during this investigation to provide a better understanding of ground water levels in the foundation and abutments and the phreatic surface through the embankment. An attempt was made to check the ground water levels in each of the drill holes upon completion of the hole. After the drilling was completed, routine measurement of the water levels was continued. Results of these readings are presented in Table 7.

During the period of time over which the monitoring was performed, the elevation of the pool in Middle Creek Reservoir fluctuated as listed in Table 8.

In general, the water level drops with the drop in the reservoir elevation but at a much slower rate. Pore pressures are not excessive in any of the holes but do appear to be higher in the foundation soil than in the embankment on the downstream side of the embankment.

Significant findings among the measurements in Table 7 are recorded in Drill Hole (DH) Nos. 4, 5, 8, 9 and 10 and are explained in the following paragraphs.

Typically, the phreatic surface through the embankment will drop with the drop in pool elevation. As recorded in Tube No. 6P, the phreatic water level in the embankment dropped 28.5 feet while the pool dropped 36.5 feet during the same period (Aug 3 to Oct 18). However, the amount of drop recorded in

Table 7

Summary of Ground Water Measurements

in Feet NGVD

Tube Number	Tube Depth(ft)	Top of Hole	Aug 3, 1983	Aug 16, 1983	Oct 18, 1983	Jan 8, 1984
1	30.0	6723.47	6712.8	6708.1	6708.6	--
2	42.0	6722.98	6710.8	6707.0	6705.8	--
3A	36.8	6721.58	6690.6	6689.2	6686.9	6687.5
3B	79.1	6721.58	6689.1	6687.3	6685.7	6686.7
4A	16.7	6638.95	--	dry	6634.1	6623.4
4B	57.0	6638.95	--	6604.3	6596.6	6613.4
4C	140.0	6638.95	--	6618.3	6606.7	6595.0
5	79.8	6681.66	6657.7	6655.7	6654.6	6651.2
6P	87.4	6721.44	6681.4	6652.9	6652.9	6652.1
6	125.0	6721.44	--	6626.2	6619.4	6621.4
7P	60.0	6680.79	--	--	6680.7	--
7	130.0	6680.79	--	--	6663.8	--
8	78.1	6681.94	6671.8	6674.0	6654.9	< 6603.8**
9A	17.9	6662.10	6652.5	6651.3	6645.7	<6644.2**
9B	47.4	6662.10	6652.5	6650.8	6035.8	6630.3
10A	30.7	6721.71	--	6692.2	6691.8	<6691.0**
10B	92.0	6721.71	6680.6	6673.3	6652.3	6653.6
11	30.0	6715.76	--	--	6695.4	--
12	34.0	6725.13	--	--	6702.5	--
13A*	75.0	6680.76	--	--	6678.8	--
13B*	56.0	6680.76	--	--	6678.8	--
14P	36.1	6683.10	--	(not stabilized)	6658.3	6656.2
14	68.0	6683.10	--	6615.2	6615.6	<6615.1**

EXPLANATION:

P	Casagrande Piezometer	
A, B & C	Identification of monitoring tubes when there is more than one in a hole, A being the shallowest.	
*	13A & B are in two separate holes, see Sheet 3 of 7.	
**	<6691.0	Levels dropped below the tube.

Source: HKM Associates

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Table 8

Middle Creek Reservoir Pool Elevations

Date	Pool Elevation (feet, NGVD)
July 25, 1983	6702.7
Aug 4	6701.0
Aug 25	6702.5
Sept 1	6685.7
Sept 15	6667.9
Oct 15	6664.5
Nov 15	6670.0
Dec 15	6674.8
Jan 15	6673.9
Feb 1	6673.6

Source: See Reference 31

These water levels were provided by the dam tender and adjusted by HKM Associates to reflect NGVD (National Geodetic Vertical Datum) elevations.

Tube No. 5 is only 3.1 feet during this same period (Aug 3 to Oct 18). The total drop in Tube No. 5 from Aug 3, 1983 to Jan 8, 1984 is only 6.5 feet. These readings seem inconsistent relative to each other as Drill Hole Nos. 5 and 6 are both near the maximum section of the embankment and in a plane approximately perpendicular to the axis of the dam. A possible explanation for this inconsistency is that Tube No. 5 is not recording the phreatic surface level.

Tube No. 5 is a slope inclinometer tube. The drill hole (DH-5) was extended into the foundation to a depth of 88.5 feet. The hole was backfilled to 65 feet and then a bentonite plug was placed in the hole at 60 to 65 feet to seal off the foundation. One foot of pea gravel was placed over the bentonite at 59 to 60 feet. Then a 2.75-inch diameter open end slope inclinometer casing was placed to 59 feet and backfilled with pea gravel. It is suspected that the bentonite plug is not sufficient to seal off the hole below the bottom of the tube. There are higher pressures in the foundation material which have more of an influence on the water level in Tube No. 5 than the phreatic surface.

It is possible that the phreatic surface is lower than is recorded in Tube No. 5. To be conservative, HKM has assumed that the water level in Tube No. 5 is correct. During construction of the rehabilitation repairs, an additional hole should be drilled in the area of DH-5 in order to install a piezometer to monitor the phreatic surface. Additional monitoring holes are described later in Sections XI and XII.

The readings in Tube Nos. 4B and 4C support the findings that pore pressures are high in the bedrock foundation. Tube No. 4C is recording pore pressures in the bedrock while Tube No. 4B represents the foundation conglomerate. Water levels are 10 to 18 feet higher in Tube No. 4C.

The monitoring program also assisted in evaluating the grouting program discussed previously in Section III.A. Based on the findings in the drill holes heretofore reported in this section and the construction history, it appears that the affectiveness of the grout curtain in controlling seepage is questionable. Because of the nature of the foundation and abutment, it is not feasible to expand or improve the grout curtain.

The water levels in Tube Nos. 8, 9B and 10B represent seepage through the left abutment. When the pool elevation varied 36.5 and 27.0 feet between August 3 to October 16 and August 3 to January 8, 1984, respectively, water levels in the tubes fluctuated as summarized in Table 9.

Table 9

Water Level Fluctuation In the Left Abutment

	Feet of Drop	
Tube No.	Aug 3 to Oct 18, 1983	Aug 3 to Jan 8, 1984
8	16.9	68
9B	17.6	22.2
10B	28.3	27.0

Source: HKM Associates

These readings indicate that seepage in the abutment is very responsive to pool fluctuations. Based on these measurements, the volume of seepage through the abutment is expected to be high.

Measurements in Tube Nos. 8 and 9A during high pool elevations suggest that the trench drain system, shown on Sheet No. 2 in

Appendix A and described in Section III.B, is intercepting seepage through the abutment. This seepage is further described in the next section.

2. Surface Seepage

The term surface seepage has been used here to refer to any seepage that can be observed visually whether it is at the ground surface or a drain pipe outlet.

Seepage quantities were estimated primarily by visual observation. A more accurate method of measuring seepage quantities is not presently available at the site.

Visual seepage from the left abutment was observed from two drain pipes. One outlet is located near Drill Hole 9 and had an estimated maximum flow of between 100 to 300 gallons per minute (gpm). The flow is greatest when the pool is at maximum elevation. This flow drops suddenly to almost nothing, less than 1 gpm, when the pool elevation drops below about 6695 feet. The reason for the sudden drop in flow is because the phreatic surface no longer intersects the drain trench.

A trace of free water is surfacing at the groin area of the right abutment and the downstream slope. Based on the measurement reported in Table 7, it is apparent that this may be perched water from the abutment. Tube Nos. 14A and 14B indicates that perched water is accumulated on the plastic silty clay deposit on the abutment. The source of the perched water is probably precipitation. This water appears to be forced to the surface in this groin area near the break in slope at elevation 6642 feet. This seepage does not appear to be a problem.

Seepage was observed from the collection drain system downstream of the embankment on the left abutment. This system was constructed in 1956 as a rehabilitation measure, as described previously in Section III.B. The location of this system is shown on Sheet No. 2 in Appendix A. The estimated seepage volume collected and discharged during of high pool is 40 to 80 gallons per minute. The volume decreases significantly during low pool.

The drain system in the embankment foundation consists of a 12-inch diameter concrete pipe drain, located as shown on Sheet No. 2 in Appendix A. The point of discharge is in the stream below the embankment. Although the discharge point has been located, a discharge volume could not be estimated as it is below the water surface in the stream. The effectiveness of this drain system is unknown. It is anticipated that a discharge volume will be estimated in the summer of 1984.

Seepage volumes do not appear to be increasing and piping does not appear to be a problem. However, field measurements should be performed using some form of monitoring device to verify these visual observations.

B. SEEPAGE ANALYSIS

1. Sink Hole Exploration

During the field investigation, several sink holes were discovered on the left abutment as discussed previously in Section IV.C.2. A meeting was held on October 12, 1983 at the site with personnel from DNRC and HKM, and with Mr. Ralph Rollins of Rollins, Brown and Gunnell, Inc. to observe the sink holes and determine the need for additional investigation.

Several holes were dug in the area during the meeting. Because each hole was similar, only Test Pit 104 was logged and is presented on Sheet 6 in Appendix A.

The test pits penetrated the impervious blanket located on the left abutment, as shown on Sheet No. 2 in Appendix A. The impervious blanket is 9-inch thick (design thickness) bentonite layer covered by about 3 feet of protective clayey gravel fill. The thickness of the impervious blanket actually varied from 1 to about 8 inches with some isolated holes extending all the way through the blanket. It was observed that the holes in the blanket coincide with the sink holes. The sink holes were actually caused by high water velocities through holes in the bentonite blanket.

While the test pits were open in the area of the sink hole, DNRC personnel pumped water into the open pit in an attempt to correlate the seepage in this area with the seepage being collected by the toe drains on the downstream side of the embankment. No seepage was observed at the drain outlet resulting from the pump-in testing. It was determined that no additional testing was needed at the time of this sink hole investigation.

2. <u>Left Abutment Seepage</u>

Seepage through the left abutment is moving under gravity-flow at a gradient as low as .04. The seepage is apparently partially controlled by the trench drain system at the downstream toe of the dam. It is not known if the foundation drain system is operating.

The sudden change in the collected seepage discharged at the drain outlet, as the elevation of the pool passes elevation 6695 (approximate), is apparently the result of the phreatic surface intersecting the trench drain system. Based on the findings during the drilling, monitoring and the sink hole exploration, there is no apparent isolated water bearing seam through the abutment which is passing a large volume of water.

The impervious blanket on the left abutment does not appear to be protecting a water bearing seam. However, the blanket is decreasing the entrance area for seepage into the abutment. As constructed on Sheet No. 2 in Appendix A, it is probably decreasing seepage to a small degree. The trench drain is a much more effective seepage control system.

If the pool elevation remains as operated historically, repair of the blanket in the area of the sink holes and test pits appears prudent in order to maintain the consistency of the existing system. However, this repair is not expected to significantly decrease the seepage volume. If the pool surface is increased, lengthening of the trench drain will be needed in addition to repairing the impervious blanket. Seepage control is discussed further in the Rehabilitation Alternatives, Section XI.

3. Calculations

An analysis was performed to determine seepage quantities through the embankments, foundations and abutments. The analysis was performed using hand calculations and a computerized (HEC) finite element program (Ref. 9). Seepage quantities were estimated by plotting profiles through the embankment, foundation and abutments, and determining the unit seepage in each. Darcy's Law (Q=KiA) was used:

Example:

K = 5500 feet per year (weighted average for left abutment)

$$i = \frac{\text{change in head}}{\text{length of flow path}}$$ (the average hydraulic gradient for the profile)

A = cross sectional area (units)

The horizontal permeability rate used in the computerized calculations for the embankment and foundation was assumed to be five times the vertical (ratio 5:1) permeability. This ratio was selected based on the knowledge that the materials are primarily gravel which has been compacted by a roller. A summary of the soil permeability parameters used in the calculations are presented in Table 10. Results of the computerized calculations are plotted on Figure 6 showing contours of equal hydraulic head. The hydraulic gradients in the vertical and horizontal direction are shown in Figures 7 and 8. A summary of the seepage volume calculations are presented in Table 11.

Table 10

Summary of Soil Permeability(K) Parameters

Soil Type	Zone Material	Horizontal K (ft/yr)	Vertical K (ft/yr)
1	Impervious	37	7
2	Semi-pervious	34	7
3	Pervious	270	54
4	Sand Deposit	37	7
5	Foundation Till	1600	320
6	Bedrock	0 (assumed)	0
	Riprap & Filter	1000	1000

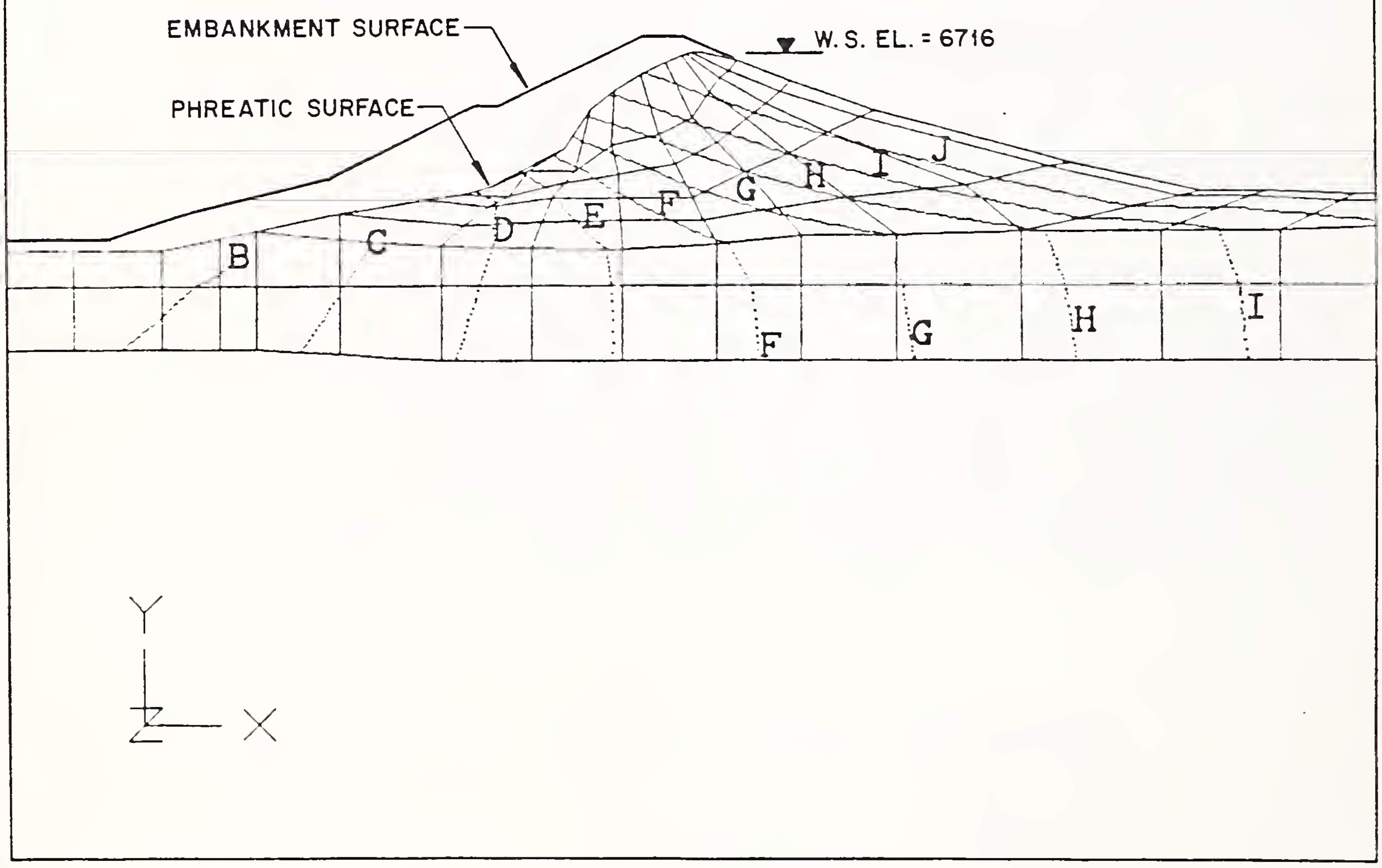
Source: HKM Associates

Table 11

Calculated Seepage Volumes

Seepage	Volume Gallons per Minute
Through-the-embankment	9 (Range 8-20)
Foundation	40 (Range 30-60)
Right abutment	25 (Range 20-50)
Left abutment	580 (Range 500-800)

Source: HKM Associates



CONTOUR	HEAD (in Ft. N.G.V.D.)
B	6617
C	6628
D	6639
E	6650
F	6661
G	6672
H	6683
I	6694
J	6705

Source: Computer generated, see reference 9

MIDDLE CREEK DAM

CONTOURS OF EQUAL HYDRAULIC HEAD

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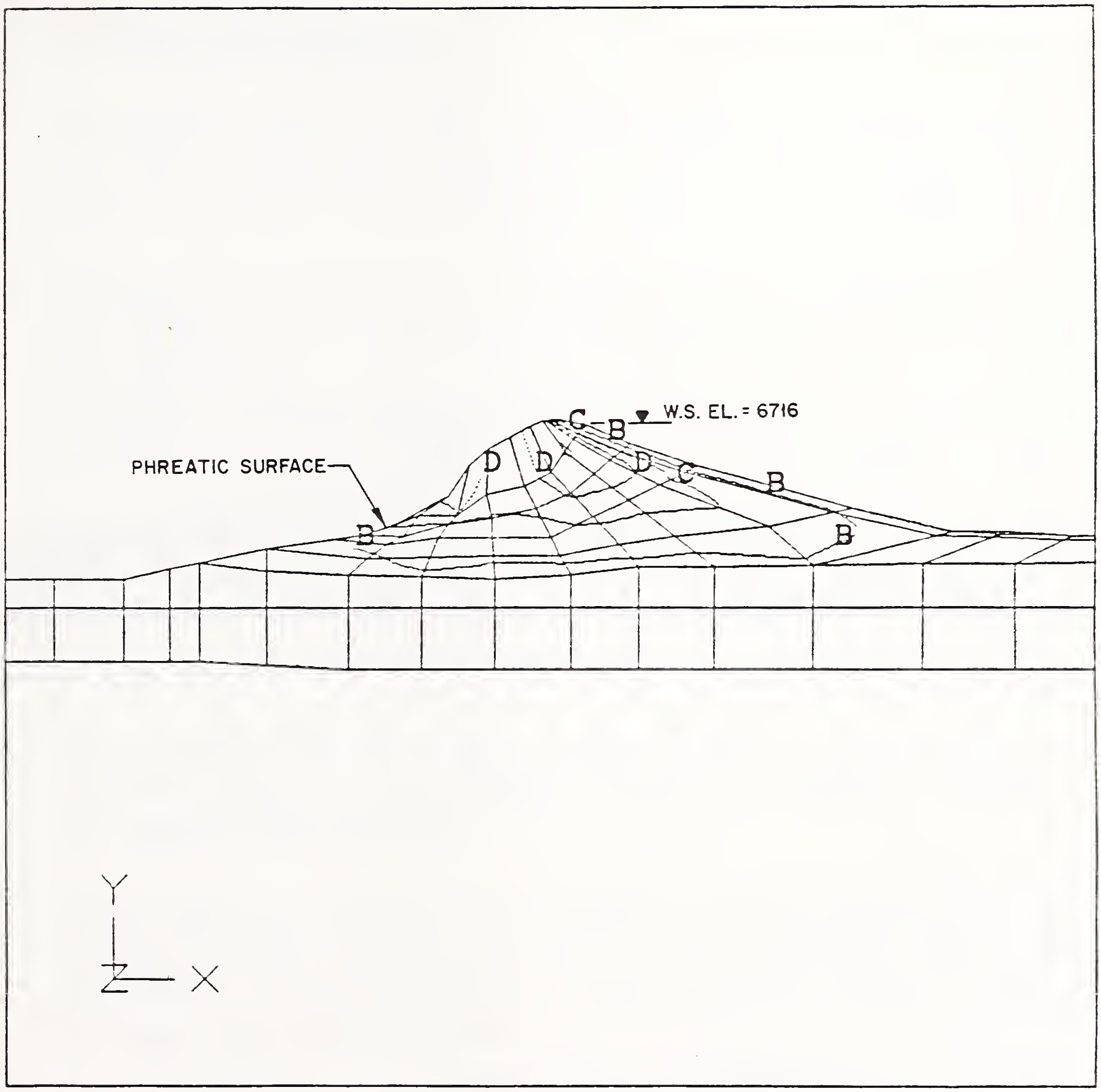
FEBRUARY 1984

FIGURE 6

HKM ASSOCIATES
ENGINEERS — PLANNERS



CONTOUR	HYDRAULIC GRADIENT
A	0.0
B	0.2
C	0.4
D	0.6
E	0.8

Source : Computer generated, see reference 9

MIDDLE CREEK DAM
CONTOURS OF EQUAL HYDRAULIC GRADIENT
IN THE HORIZONTAL (X) DIRECTION

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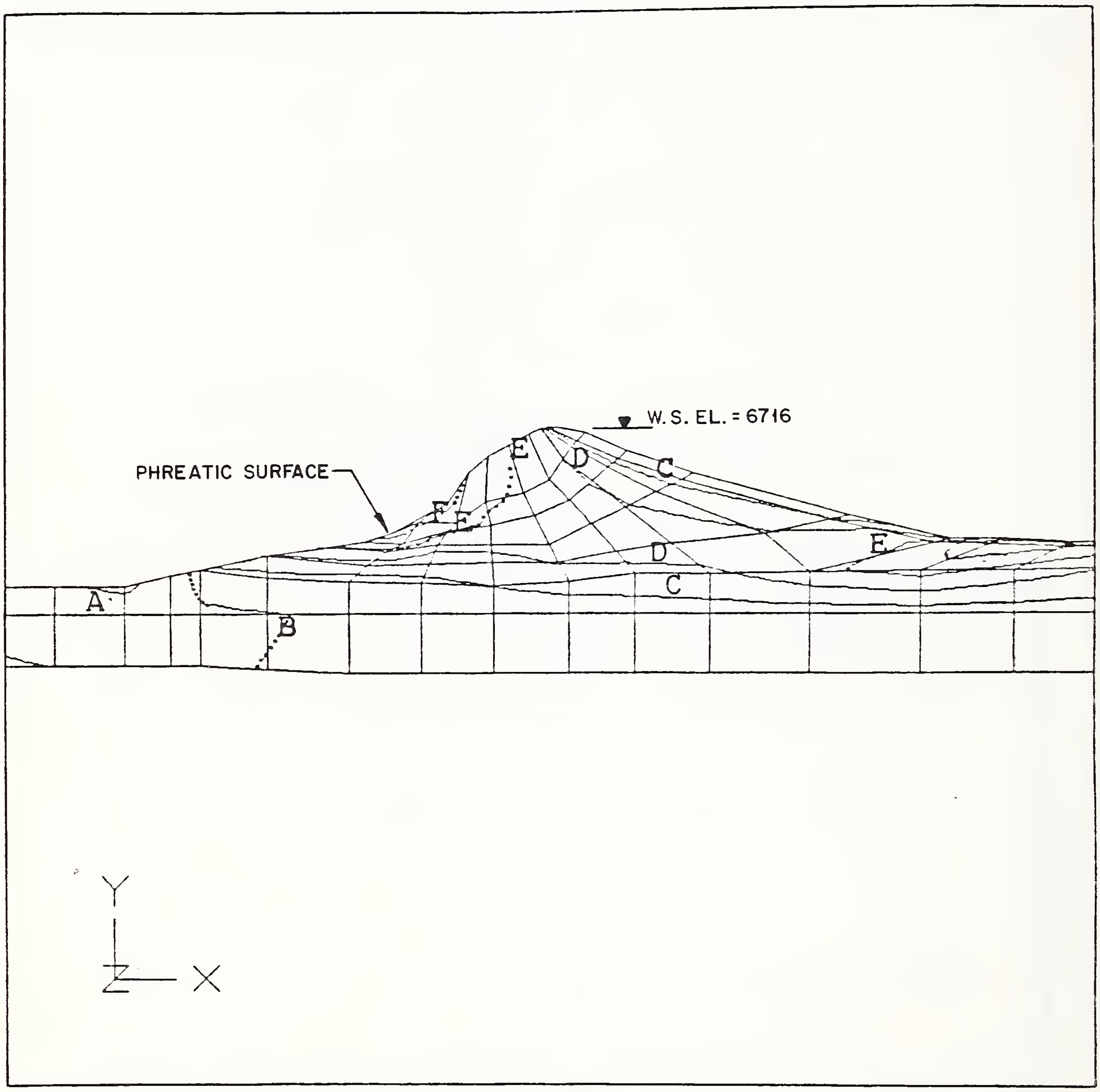
FIGURE 7

HKM ASSOCIATES
ENGINEERS — PLANNERS



CONTOUR	HYDRAULIC GRADIENT
A	-0.30
B	-0.083
C	0.133
D	0.350
E	0.57
F	0.78
G	1.0

Source: Computer generated, see reference 9

MIDDLE CREEK DAM
CONTOURS OF EQUAL HYDRAULIC GRADIENT
IN THE VERTICAL (Y) DIRECTION

8M087.113 FEBRUARY 1984

FIGURE 8

HKM ASSOCIATES
ENGINEERS — PLANNERS

C. SEEPAGE CONCLUSIONS

1. The water levels in the abutments and embankments respond to increases and decreases in the pool elevation as anticipated but at significantly lower rates of change.

2. Hydrostatic pressures in the foundation and embankment do not appear to be excessive. The hydrostatic pressures in the foundation in the area of Drill Hole Nos. 4 and 5 (downstream) are higher than the embankment creating an artesian effect.

3. The change in the phreatic surface through the embankment is slow relative to the potential change in pool elevation suggesting that the sudden drawdown case is critical in assessing the embankment stability.

4. The effectiveness of the grout curtain in controlling seepage appears questionable. It is not feasible to expand or improve the curtain.

5. The volume of seepage at the dam does not appear to be increasing with respect to past volume estimates. A metered monitoring system should be installed to verify seepage volumes from both abutments.

6. Piping does not appear to be a problem. However, a turbidity test should be performed prior to rehabilitation construction to establish a basis for future analysis. After construction, turbidity should be monitored for at least one year.

7. Seepage in the groin area on the right abutment does not appear to be a problem but should be monitored.

8. Seepage through the left abutment is controlled by the drain systems on the downstream side of the dam.

9. The effectiveness of the foundation drains is unknown.

10. The impervious blanket on the left abutment should be repaired to maintain the consistency of the existing seepage control system.

IX. STATIC STABILITY ANALYSIS

A. STABILITY CONSIDERATIONS

The static stability analysis performed to evaluate the slope stability included the following primary considerations:

- The embankment configuration
- The soils' physical characteristics
- The soil strength parameters
- The location of the phreatic surface through the embankment and abutments
- The loading conditions to be analyzed

Each item will be reviewed with regard to stability in the following subsections. The potential for liquefaction of the foundation soils and deformation resulting from a seismic event will be discussed in Section X.

1. Embankment Configuration

HKM Associates prepared 4 surface profile drawings of the embankment during the field investigation. Additionally, a topographic drawing, (Sheet No. 2 in Appendix A) with 1 foot contour intervals, was prepared using aerial photography. Using this data and the configuration shown in the design drawings, an embankment profile was selected for the stability analysis. The cross section is shown on Sheet No. 3 of Appendix A.

As discussed previously in Section II.C., Figure 2 has been prepared to show the difference between the measured profile of the embankment and the design drawings. These findings indicate that crest width is about 6-8 feet wider than the design drawing and some overbuild appears on the upstream slope. Apparently, there was a small runoff collection trench constructed on the berm on the downstream slope (See Figure 2). Since construction, the trench has filled with slough. There are no physical features of this trench visable today.

The internal zones of the embankment are represented as detailed in the design drawings (Sheet 3 in Appendix A and Reference 22). The characteristics of the material in the embankment were determined using test results of soils sampled during the drilling.

The zones within the embankment are also indicated on Sheet No. 3. Findings during the drilling resulted in a general agreement with the cross section indicated on the design drawings. However, the location of the boundaries between soil types 1, 2, and 3 are not well defined.

2. Physical Characteristics of the Embankment

The physical characteristics of the embankment have been described previously in Section VII. As a summary, these characteristics are referred to below by soil type.

Soil Type*	See Plate Nos. (in Appendix A)
1 (impervious zone)	3, 6, 11, 17
2 (semi-pervious zone)	4, 5
3 (pervious zone)	14, 16
4 (sand layer)	7, 8, 9
5 (foundation)	10, 13
6 (bedrock)	2

* Reference soil type with cross section shown on Sheet No. 3 in Appendix A.

3. Strength Parameters

The embankment cross section used in this analysis is shown on Sheet No. 3 in Appendix A. The parameters assigned to the various soil types were selected based on the laboratory soils strength tests presented in Plate Nos. 25-27 in Appendix A, the field tests, and our experience with soils of similar physical characteristics. It was extremely difficult to obtain undisturbed samples at the site. Many attempts were made and failed because of the coarseness of the material. Where undisturbed samples were obtained, the material was slightly finer. Therefore, the laboratory strength tests likely resulted in conservative strength values.

The strength parameters used with the static and dynamic computer programs are presented in Table 12. Dynamic stability will be discussed in Section X.

Table 12

Strength Parameters

Soil Type	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Cohesion Value (psf)	Internal Friction Angle
1 (impervious)	120	125	200	32.5 (steady state)
1 (impervious)	120	125	200	26.0 (sudden drawdown)
2 (semi-pervious)	125	130	0	34.0 (steady state)
	125	130	0	30.0 (sudden drawdown)
3 (pervious)	125	130	0	34.0
4 (sand layer)	105	110	0	28.0
5 (foundation)	130	130	0	34.0
6 (bedrock)	135	135	1000	30.0

Source: HKM Associates

Two different internal angles of friction were used for soil types 1 and 2. The lower values represent the total stress parameters used for the sudden drawdown analyses. The higher values were used for the steady state seepage stability analysis and for the dynamic analysis which will be described in Section X. Plate No. 25 in Appendix A represents the strength parameters selected for soil type 1. A cohesion value of about 1/2 the tested value was selected. The strength of the sand foundation is represented on Plate No. 26. A cohesion value of 0 was used because the physical properties tests indicated that much of this sand material is granular non-plastic.

4. Phreatic Surface

The phreatic surface used in the stability analyses is delineated on the embankment cross section shown on Sheet No. 3 in Appendix A. The actual measured water levels in the monitoring tubes are also presented. The computerized flow analysis (Ref. 9) described in the Seepage Analysis, Section VIII. indicated a higher phreatic surface. Therefore, a conservatively higher phreatic surface was selected in the area of Drill Hole 6 for the analysis.

The phreatic surface measured in Drill Hole 5 is 20 to 30 feet higher than that calculated by the finite element computer analysis (Ref. 9). It is possible that the computer analysis is correct because the computer drawn phreatic surface is based on an average of many field permeability tests while the elevation of the measured phreatic surface may not be representative because of the characteristics of the drill hole. The hole was drilled into the foundation soil where higher pore pressure may exist, as described previously in

Section VIII.A.1. An attempt was made to plug the hole with bentonite during the drilling to seal off any differential pressures. However, the quality of the bentonite seal is questionable and an artesian effect is apparent. The higher, measured, phreatic surface was used in the analysis to be conservative.

B. SLOPE STABILITY COMPUTATIONS

The embankment slope stability was calculated by computer using the Corps of Engineers program "Slip Circle Slope Stability with Side Forces". This program utilized Taylor's modified Swedish method with a circular arc failure plane (Ref. 13).

Three loading conditions were analyzed: sudden drawdown, steady state seepage, and seismic loading. Each of these conditions are discussed in this section.

1. Sudden Drawdown

Middle Creek Dam is used for recreation, municipal water supply and for irrigation. Late in the summer when the irrigation demand is high, the pool level is drawn down relatively quickly creating sudden drawdown conditions. It was assumed for the loading condition analyzed that the drawdown would be from the normal pool elevation to a minimum pool elevation of 6637 feet. This elevation is estimated to be about 35 feet lower than the typical minimum drawdown elevation (Ref. 31). This minimum pool elevation is considered conservatively low.

In the calculations the resisting friction forces were determined using moist or saturated unit weights above the phreatic surface, at normal (full) pool, and submerged weights below this level. The driving forces were determined using saturated weights above the lowered pool, saturated weights within the drawdown zone, and submerged (buoyant) weights below the drawdown zone. The total strength soil parameters were used in the computer calculations.

2. Steady Seepage with Normal Pool

The condition of steady state seepage is developed when the normal (maximum) water level is maintained for a period of time sufficient to develop a steady (non-transient) phreatic line through the embankment. A flow net was drawn from water levels measured in the drill holes and checked by computer (Ref. 9) to estimate the configuration of the phreatic surface.

3. Seismic Loading

Seismic effects were first evaluated using the pseudostatic method of analysis. This analysis assumes that the earthquake imparts an additional horizontal force into the embankment acting in the direction of the potential failure (approximately parallel to embankment surface). The critical failure arc (lowest factor of safety) is used with this added driving force to determine a new factor of safety for the seismic loading (Ref. 8).

4. Results

The minimum calculated factors of safety are presented in Table 13. The recommended minimum factors of safety, in accordance with the Corp of Engineers Recommended Guidelines (Ref. 10), are also presented in Table 13 for comparison.

Table 13

Results of the Static Stability Analysis

Loading Condition	Minimum Factors of Safety Calculated	Recommended
Sudden Drawdown	1.26	1.2
Steady State Seepage	1.67	1.5
Seismic Loading on Downstream Slope	1.09	1.0

Source: HKM Associates and Reference 10

It was assumed in the calculations that the direction of the side forces is parallel to the average surface profile adjacent to the slice interface. Computer printouts of each of these minimum factors of safety are included in Appendix B. The Recommended Guidelines (Ref. 10) do not require an earthquake loading calculation for the sudden drawdown condition.

C. EMBANKMENT STABILITY CONCLUSIONS

The existing Middle Creek Dam embankment is in compliance with the Recommended Guidelines. There are no apparent static stability problems with the existing embankment. Factors of safety for an increased crest elevation are presented later in Section XI.

X. DYNAMIC STABILITY ANALYSES

A. NEED FOR DYNAMIC ANALYSES

The COE Recommended Guidelines (Ref. 10) recommends that a seismic stability investigation be performed for high hazard dams located in Seismic Zone 4. Middle Creek Dam is located in seismic Zone 3 near the borderline between Seismic Zones 3 and 4, as shown on Sheet No. 7 of Appendix A. Because of its proximity to Zone 4 dynamic analyses were considered prudent.

The "state-of-the-art" in seismic stability analyses includes three general methodologies; a comparative analysis, a liquefaction analysis and a deformation analysis. The comparative analysis is simply a comparison with other dam embankments, soil types and seismic events which have been associated with liquefaction. This comparison is always the first approach or step for a dynamic stability analysis. This comparison procedure is also used in conjunction with computerized dynamic analysis when evaluating the cyclic strength of embankments based on penetration test data.

The liquefaction and deformation analyses are more sophisticated analytical procedures involving field and laboratory testing, and computerized calculations. The liquefaction analysis is performed when the comparison procedure indicates that liquefaction is a possibility and/or when it is known that the embankment and foundation soils suffer a considerable loss in strength under cyclic loading conditions.

Deformation should not be a problem for a dam and foundation not subject to liquefaction. However, if the pseudostatic analysis indicates a factor of safety less than 1.0, using a seismic coefficient of one third (1/3) the amplified peak acceleration, a deformation analysis is warranted. All three of these procedures were warranted for Middle Creek Dam. Each of these analyses are summarized in the following sections.

B. COMPARISON PROCEDURE

A comparison procedure is presented in Exhibits 6 and 7 in Appendix B. Exhibit 6, titled Bureau of Reclamation Seismic Reevaluation of Embankment Dams Criteria, provides an evaluation of the liquefaction potential based on an earthquake magnitude and the epicentral distance. Using the Deep Creek Fault, which is located about 25 kilometers from Middle Creek Dam, and a design earthquake with a Richter magnitude of 6.5, the chart suggests that the potential for liquefaction is possible.

The other comparison procedure, Exhibit 7, titled California's Seismic Re-evaluation of Dams Criteria, requires estimating the relative density of the material susceptible to liquefaction, classifying the level of seismic acceleration and determining the soils classification. The material encountered, in Drill Hole Nos. 7 and 13, underlying the embankment fill and abutment till, respectively, is a silt and sand mix. Typically, these type materials are susceptible to liquefaction and deformation when the relative density is moderate to low. Standard penetration resistance tests made in Drill Hole 7 indicate that the relative density of this material is about 71% (N'=29) which is considered dense. The seismic design acceleration is 0.22g, as discussed previously in Section V. The predicted behavior of the foundation soils based on this California procedure suggests that there is not a liquefaction problem.

C. LIQUEFACTION ANALYSIS

1. Procedure

The dynamic analysis procedure used for Middle Creek Dam is summarized in this section. The general procedure was presented at the American Society of Civil Engineers National Convention in May 1983 (Ref. 17). The basic steps are listed as follows:

1. Determine the cross section of the dam to be used for analysis.
2. Determine the maximum credible time history of accelerations to which the dam and its foundation might be subjected.
3. Determine, as accurately as possible, the stresses existing in the embankment before the earthquake. This is best accomplished using finite element techniques.
4. Determine the dynamic properties of the soils comprising the dam, such as shear modulus, damping characteristics, and Poisson's ratio which determine the embankments response to dynamic excitation. These properties are non-linear so it is necessary to determine how these properties vary with strain.
5. Compute, using an appropriate dynamic finite element analysis procedure, the stresses induced in the embankment by the selected base excitation.
6. Calculate the cyclic strength of the liquefiable soils in the embankment based on the standard penetration resistance values, the effective normal vertical stress, and the ratio of shear stress to vertical stress. Use a recent state-of-the-art procedure for determining the cyclic strength (Ref. 17).
7. Evaluate the factor of safety against failure in the liquefiable areas by comparing the cyclic strength with the cyclic stress induced by the time history of accelerations.

8. If liquefaction does not occur, determine the deformations which are likely to occur in the embankment due to the time history of accelerations.

These steps are further described in the following subsections. Step 8 is described in the next section, D. Deformation Analysis.

2. Step 1

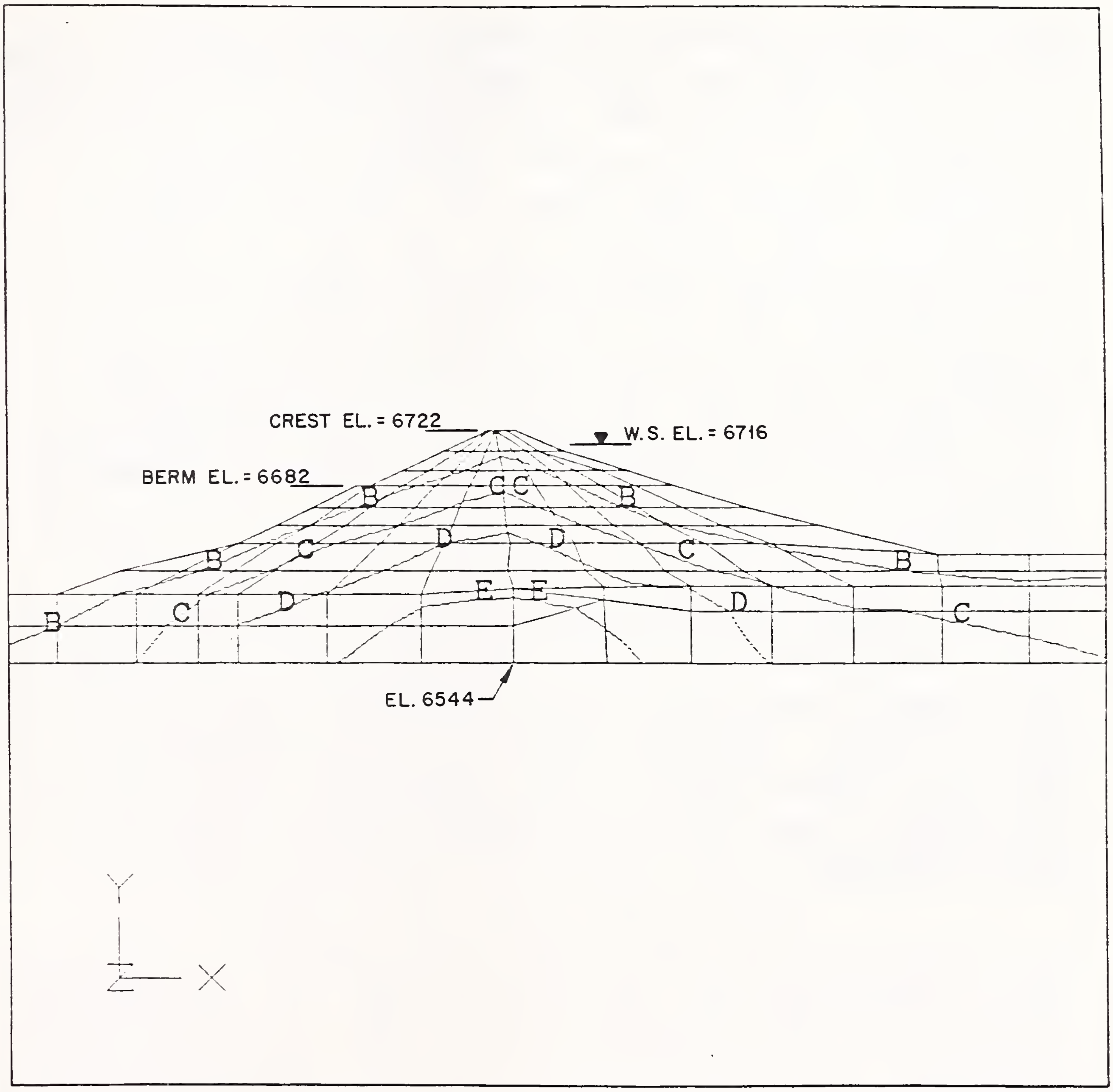
The cross section used in this analysis is the maximum embankment section shown on Sheet No. 3 in Appendix A. The surface profile was prepared using a ground survey made in the field. The internal boundaries were estimated using the boring logs and the design drawings. The material strength properties are previously presented in Table 12.

The embankment was divided into the finite element grid shown in Figure No. 9. Each element in the grid is assigned the appropriate soil parameters.

3. Step 2

This step involves the determination of the maximum likely time history of accelerations to which the dam will be subjected during its design life. The critical parameters for the maximum credible earthquake (MCE) are Richter magnitude, maximum acceleration, duration of shaking, and fundamental period. These parameters were described previously in Section V.

The closest fault to the Middle Creek Dam site is the Deep Creek-Luccock Park Fault located approximately 25 kilometers southwest of the dam site. The U.S.G.S. publication, Probabilistic Estimates of Maximum Acceleration and Velocity in



CONTOUR	EFFECTIVE VERTICAL STRESS, in Tons/Ft.^{2}
A	0
B	1.1
C	2.1
D	3.3
E	4.4
F	5.5

Source: Computer generated, see reference 23

MIDDLE CREEK DAM

CONTOURS OF EFFECTIVE VERTICAL STRESS

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FIGURE 9

HKM ASSOCIATES

Rock in the Continguous United States (Ref. 1) gives estimates of accelerations with a 90% probability of not being exceeded in 50 years. For our site the maximum value is 0.22 g. This corresponds to a 6.5 Richter magnitude earthquake at the Deep-Creek Luccock Park Fault, as discussed previously in Section V. This magnitude, 6.5, may also be selected from attenuation curves prepared by Seed and Idriss (Ref. 26). In this same paper (Ref. 26), the predominant period is shown as a function of Richter magnitude and distance from the fault. The predominant period for our site is approximately 0.28 seconds. For a magnitude 6.5 earthquake, the duration of significant shaking would be approximately 16 seconds as derived previously in Section V.

It would seem logical to use the Hebgen Lake record as the design earthquake, however, it was recorded at a seismograph 48 miles from the epicenter and the maximum recorded acceleration was only 0.05g. The accelerogram has been damped out such that the true representation of the earthquake is questionable. Furthermore, there is no corrected, digitized record of the quake available.

The Helena quake of 1935 (6.0 M) was recorded at a seismograph only 7 kilometers from the epicenter (Ref. 2). The specteral content has been preserved due to the proximity of the seismograph. The maximum acceleration was approximately 0.14 g and this record has been corrected and digitized for computer use.

HKM Associates used the Helena East-West time history of accelerations since the fourier spectrum analysis of the components indicates that this component had more energy. The time history was scaled up to a maximum acceleration of 0.22 g and the fundamental period was changed from 0.33 sec to 0.28 sec by altering the time step. The duration was set at 16 seconds.

4. Step 3

This involves several intermediate steps. They are as follows:

1. Perform triaxial shear tests of the materials comprising the dam in order to establish the non-linear, stress dependent parameters for use in the static stress analysis.

2. Perform a seepage analysis to determine the phreatic surfaces and the hydraulic gradients in the x and y directions so that seepage forces can be calculated.

3. Calculate the effective stress state in the embankment and foundation using the finite element program ISBILD developed at Cal-Berkeley (Ref. 23). Use the buoyant weight below the phreatic surface and total weight above the surface and apply the seepage forces caused by seepage through the dam at the finite element nodes.

The results of the triaxial shear tests are presented on Plate No. 25.

The results of the seepage analysis have been presented previously in Section VIII. The results of the computerized finite element program for seepage analysis also included calculations of the hydraulic gradient in the x and y directions. Figures 7 and 8, presented in Section VIII, show the computer graphic printout of the hydraulic gradient. Knowing the value of the hydraulic gradient at each element, the seepage forces at the nodes were calculated.

The results of the static effective stress calculations from the ISBILD computer calculations are plotted on Figure 9. The results of the shear stress calculations are shown on Figure 10. It should be recognized that the results of the effective vertical stresses also represent the major principal stresses in the embankment.

5. Step 4

The computer program selected to perform the dynamic calculation for the Middle Creek Dam is QUAD 4 (Ref. 15). This program was developed at the University of California at Berkeley specifically for evaluating the seismic response of soil structures by variable damping finite element procedures. This program is further described in the next step. The purpose of this step is to describe the selection of the shear modulus and damping ratio used with QUAD 4.

The shear modulus (G) has been found to be fairly well approximated for sands by the following equation (Ref. 27).

$$G = 100\ K_2\ (\sigma'm)^{1/2}\ \text{psf}$$

where $\sigma'm = \dfrac{1 + 2Ko}{3}\ \sigma v'$

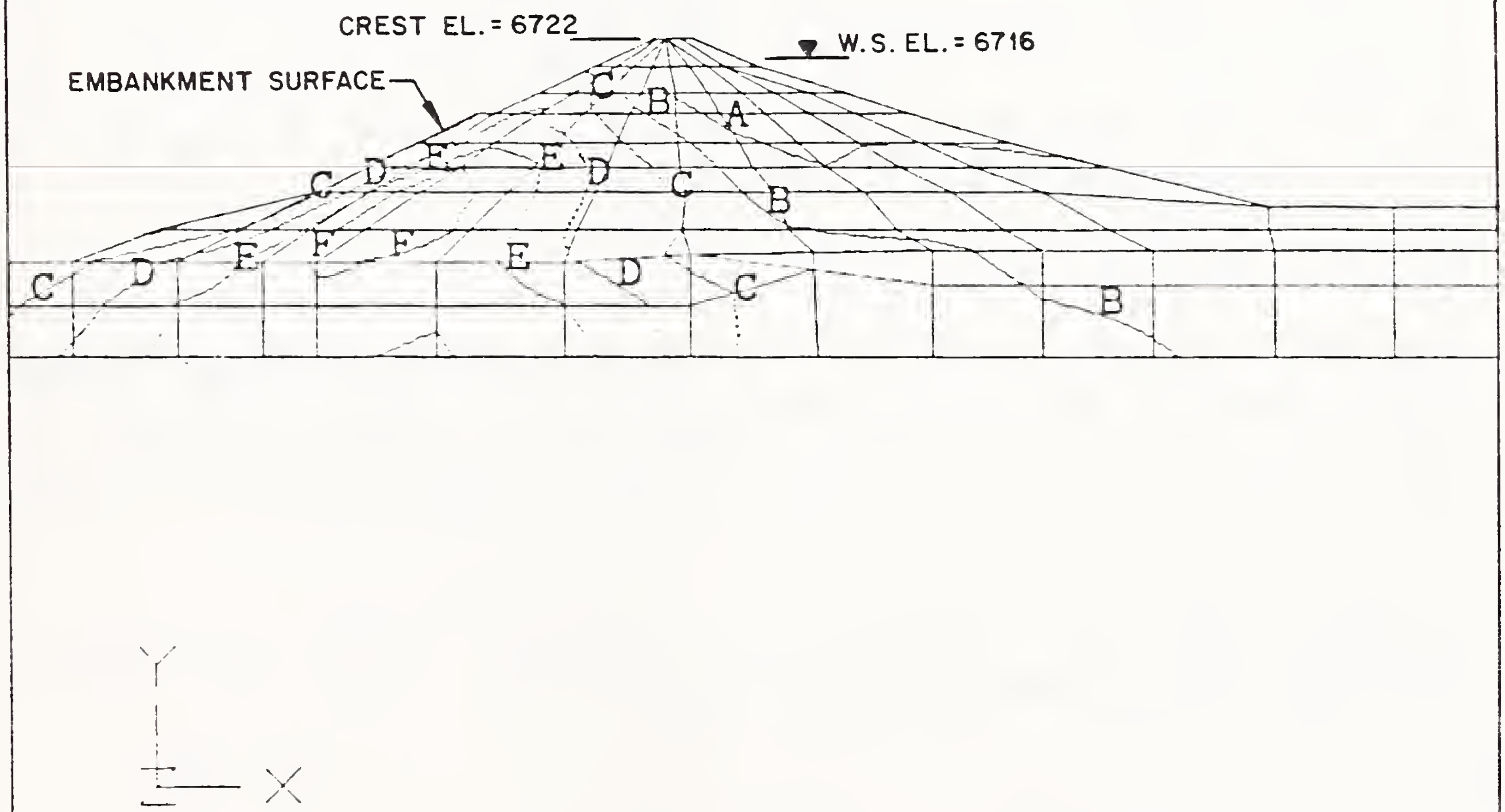
Ko = coefficient of lateral stress at rest

K_2 = non-dimensional parameter which reflects the influence of void ratio and strain amplitude

$\sigma'm$ = effective mean pressure

$\sigma'v$ = effective vertical normal pressure

K_2 varies non-linearly with shear strain but reaches a relatively constant maximum value at small strains (defined as 10^{-4}%). The QUAD 4 program requires a maximum value for G and an initial value for G and then iterates until the G used agrees with the G associated with the calculated strain. The relationship between G and the unit weight used in the program is that proposed by Seed and Idriss (Ref. 27).



CONTOUR	SHEAR STRESS τ_{xy}, in Tons/Ft.^{2}
A	0
B	0.2
C	0.4
D	0.6
E	0.8
F	1.0
G	1.2

Source: Computer generated, see reference 23

MIDDLE CREEK DAM

CONTOURS OF STATIC SHEAR STRESS

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FIGURE 10

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The damping ratio varies non-linearly from a high value of near 24% at 1% shear strain to a low value of 1% at 10^{-4}% shear strain. The QUAD 4 program requires only an initial guess for damping ratio and then it iterates until the damping is appropriate for each element.

6. Step 5

This step involves calculating the maximum shear stress in each element induced by applying the specified time history of accelerations at the base of the finite element grid. The seismic event selected is the Helena earthquake of October 31, 1935 scaled up to 0.22g.

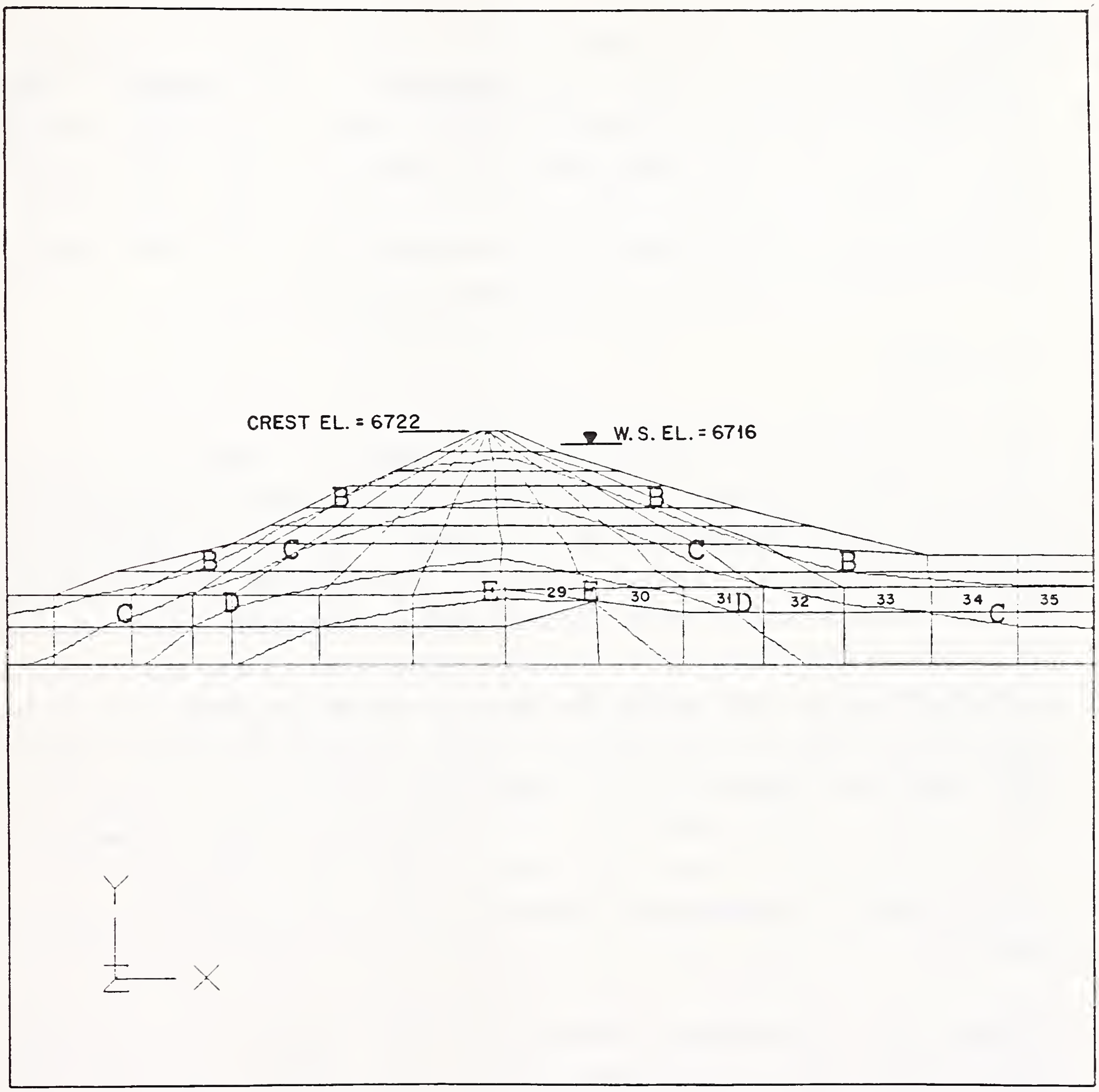
The results most pertinent to the analysis are the induced dynamic stresses in the elements representing the questionable sand deposit under the upstream slope. These elements are consecutively numbered from 29 to 35, as represented on Figure 11.

Results, for the elements in question from the computer printout are presented in Table 14.

Table 14
Results of Cyclic Stress Calculations

Element	Tc (max. induced, psf)	Cyclic Stress Tc X .65 (psf)
29	2024	1316
30	1899	1234
31	1680	1092
32	1083	704
33	1208	785
34	918	596
35	968	629

Source: Rollins, Brown and Gunnell Inc.



CONTOUR	MAXIMUM DYNAMIC SHEER STRESS τ_c, in pcf (induced by earthquake)
A	200
B	740
C	1280
D	1820
E	2360
F	2900

Elements 29 – 35 represent the sand deposit

Source: Computer generated, see reference 15

MIDDLE CREEK DAM

CONTOURS OF MAXIMUM DYNAMIC SHEAR STRESS

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FIGURE 11

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These stresses are calculated using direct step-by-step integration of the equations of motion. The program iterates so that the damping ratio and shear modulus for each element are compatible with the strain in the element. The maximum shear stress is multiplied by a factor of 0.65 (Ref. 30) to determine an equivalent cyclic stress which can be compared with the cyclic strength calculated in Step 6.

7. Step 6

The step includes determining the dynamic strength of the sand layer in the foundation. The most recent method being used to evaluate the dynamic strength of sands involves a correlation between standard penetration tests and field data of actual sand deposits which have failed due to liquefaction. The use of penetration test data in lieu of cyclic load test data is consistent with the state-of-the-art. This approach has become the more desirable because it avoids judgment on the part of the engineer concerning the representativeness of samples and for cyclic load testing. In addition, the strength is correlated with actual earthquakes and sand deposits in the field rather than some equivalent laboratory representation of shaking.

During the drilling operations, a number of standard penetration tests were performed on the sand, silty sand, and sandy silt which was found to exist beneath the upstream toe of the dam. The following is a summary of corrected (See Section VII.A.) N' values with the depth location:

DH#	Depth	Description	(Corrected Blows N' Per Foot)
7	66'-71'	Silty sand, compact, black	38
	73'-76'	Clayey Sand, 77% sand 19% silt and clay	34
	76'-78'	Clayey sand, 77% sand 19% silt and clay	29
	78'-80'	Clayey sand, 77% sand 19% silt and clay	35
	80'-83'	Clayey silt, 89% silt 9% clay (non-plastic)	35

The minimum value, N' = 29, was used for this analysis. The pre-earthquake static stresses were then used to calculate the cyclic shear strength.

The N' value of 29 is believed to be conservative for two reasons; first, because it is the minimum test value and second, because of the high silt and clay content. The average N' value is 34. N'=29 is the lowest penetration test value obtained in the sand deposit.

Studies show (Ref. 28) that the liquefaction potential for sands with an appreciable amount of fines decreases with the increase in the percentage of silt and clay. In fact, correlations between penetration resistance and liquefaction characteristic for sands are not applicable for silty sands unless they are modified to allow for the fine content of the silty sands. This is done by adding about 6 to the N' value. Based on the laboratory tests, it appears that there is some justification in increasing the N' to 35 for this analysis. However, because the lab tests represent the soil in only isolated areas (two drill holes), it appears prudent tc limit N' to 29.

8. Step 7

A summary of the results of this portion of the dynamic analysis is presented on Table 15.

Table 15

Factors of Safety Against Liquefaction

Element	Cyclic Strength	Cyclic Stress	Factor of Safety
29	2080	1316	1.58
30	3720	1234	3.01
31	2720	1092	2.50
32	1240	704	1.76
33	2280	785	2.90
34	1240	596	2.08
35	1720	629	2.73

Source: Rollins, Brown and Gunnell Inc.

In every element in question, the factor of safety against liquefaction is at least 1.5. The minimum accepted factor of safety is generally considered to be 1.0. The dam should not experience liquefaction for an earthquake with a magnitude of 6.5 and an epicenter as close as the Deep Creek-Luccock Park Fault and with a maximum horizontal ground acceleration induced at the embankment foundation of 0.22g.

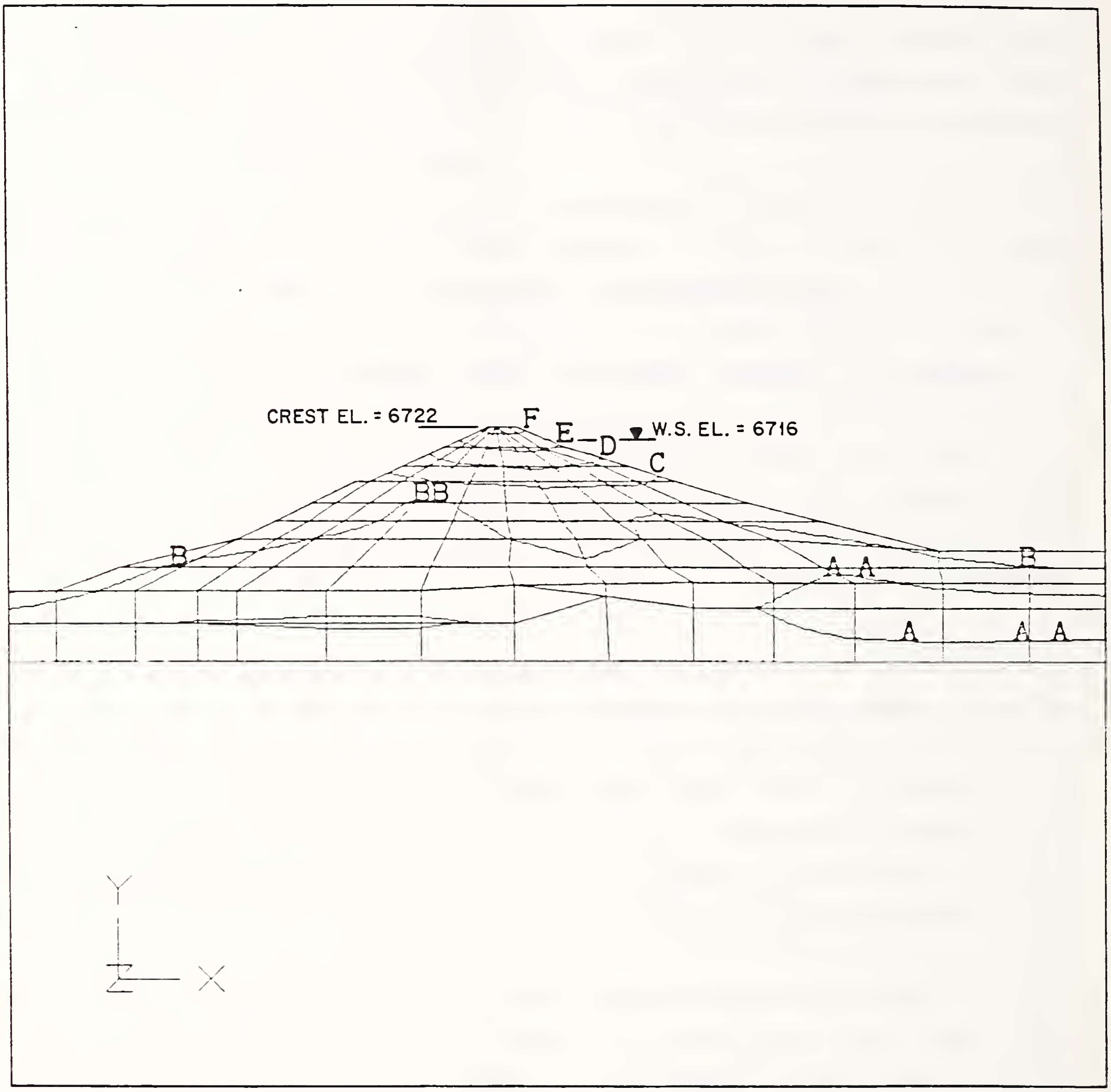
D. DEFORMATION ANALYSIS

Based on the Bureau of Reclamation Seismic Reevaluation of Embankment Dams Criteria, presented in Exhibit 6 in Appendix B, a deformation analysis is still warranted. An analysis was performed based on the Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformation, by F.I. Makdisi and H.B. Seed (Ref. 19).

In this analysis the permanent displacement of the embankment was estimated from a design curve which was derived from a number of deformation case studies. The use of this curve requires a knowledge of the yield acceleration and the time history of average induced accelerations for a potential sliding mass. The yield acceleration is that induced movement at which deformation may be initiated within an embankment. The design curve is based on averages of a range of results. Therefore, the deformation estimate provides a range of potential movement.

Contours of the maximum horizontal accelerations (g's) induced within the embankment by the design earthquake are presented in Figure 12. A maximum peak (at crest) acceleration of 0.47g was calculated by the Quad 4 computer program for this portion of the dynamic analysis. The natural period of the embankment due to the specified ground motion of 0.22g was also calculated to be 0.69 seconds.

The calculations indicated that the embankment deformations resulting from the design earthquake and a yield acceleration of 0.05 would be in the range of 0.4 to 1.0 feet. This amount of movement is small and does not create a freeboard problem as the freeboard above normal pool is expected to be greater than 8 feet. Because the sand deposit does not underlie the outlet conduit, settlements resulting from the design earthquake are not anticipated along the conduit.



CONTOUR	MAXIMUM HORIZONTAL ACCELERATION, (g's)
A	0.20
B	0.25
C	0.30
D	0.35
E	0.40
F	0.47

Horizontal accelerations were generated using the design earthquake with a magnitude of 6.5 and a specified horizontal acceleration at the base of .22 g.

Source: Computer generated, see reference 15

MIDDLE CREEK DAM
CONTOURS OF MAXIMUM
HORIZONTAL ACCELERATIONS

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FIGURE 12

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E. CONCLUSIONS OF DYNAMIC STABILITY ANALYSES

Dynamic stability analyses have been performed using three methodologies; a comparative procedure, a liquefaction analysis and a deformation analysis. The comparative analysis by the Bureau of Reclamation procedure indicates that there is a potential for liquefaction while the analysis using the California procedure suggests that there is no problem.

The results of the liquefaction analysis indicate that the minimum factor of safety against liquefaction is at least 1.5. Therefore, liquefaction should not be a problem.

Calculations indicate that the maximum anticipated deformation resulting from the design earthquake is 1 foot. This amount of deformation would not create a freeboard problem.

XI. REHABILITATION ALTERNATIVES

A. GENERAL

Rehabilitation alternatives for Middle Creek Dam are limited to increasing the embankment height to store more water and pass a larger design flood. All alternatives are not fully developed, as yet. However, preliminary information suggests that it is desirable to raise the existing embankment approximately 10 feet to provide additional water storage and surcharge volume for storm control.

This initial analysis of alternatives to increase the embankment height was made to aid in project planning. Two methods of raising the crest appear feasible from the technical standpoint; adding soil to the downstream side of the embankment and placing a cap on the crest such as Reinforced Earth. Cross sections of the embankment, with the proposed alternatives for raising the embankment, are shown on Sheet No. 3 in Appendix A.

This section describes the two alternatives for raising the crest elevation in sufficient detail to allow comparison of the advantages and disadvantages of each. Seepage and stability considerations are discussed as they relate to these rehabilitation alternatives. The impact on the outlet conduit of raising the embankment crest is also discussed.

B. EARTH FILL ALTERNATIVE

The Earth Fill alternative consists of placing additional fill on the crest and over the downstream slope to raise the crest elevation.

The placement of additional fill on the downstream slope to raise the crest elevation 10 feet would include, but not be limited to, the following work items:

1. Strip approximately 11,500 cubic yards from the embankment and foundation area on the downstream side.
2. Extend the existing outlet conduit approximately 20 feet.
3. Place approximately 165,000 cubic yards of new embankment on the downstream face. This volume also includes a gravel road on the crest.
4. Riprap the upstream portion of the new embankment which will require approximately 4000 cubic yards of loose rock.

This alternative appears feasible from the stability standpoint as the embankment may be constructed to maintain the minimum required factor of safety of 1.5. There is an advantage to placing the earthfill on the downstream slope versus the upstream slope as removal and replacement of the existing riprap is costly.

The possibility of raising the embankment as much as 20 feet was also considered. Cost estimates for both earth fill alternatives are being prepared.

C. <u>REINFORCED EARTH ALTERNATIVE</u>

The Reinforced Earth alternative consists of constructing a Reinforced Earth cap on the crest of the dam, as presented on Sheet No. 3 in Appendix A. Preliminary estimates indicate that the cap would be 28 feet wide by 10 feet high. The length of the cap would be about 1100 feet. The remaining proposed crest length, 350 feet may be raised by placing an earth fill section over the downstream face.

In order to maintain adequate stability using this alternative, the phreatic surface through the embankment must be held near its present location. This could be accomplished by constructing an impervious barrier in a trench located upstream of the Reinforced Earth panel wall, as shown on Sheet No. 3 in Appendix A. The trench would need to be about 6 to 8 feet deep by 2 feet wide and about 1150 feet long. A small drain section with a drain collection pipe would be installed at the bottom of this trench to provide a more positive control of the phreatic surface. Seepage would be collected at the abutments and discharged to a monitoring system and then the stream. Stability using this alternative is discussed later in this section. This alternative would include but not be limited to the following work items:

1. Strip approximately 10,500 cubic yards of fill off the crest of the embankment.

2. Place approximately 30,800 square feet of Reinforced Earth concrete panel.

3. Backfill inside the panels with approximately 1500 cubic yards of drain fill and 16,000 cubic yards of granular (semi-pervious) fill. This would include road fill.

4. Construct the earth fill portion of the embankment which would consist of about 28,000 cubic yards of material.

5. Construct the impervious barrier to a depth of about 7 feet below the base of the Reinforced Earth foundation.

6. Replace the fill and riprap against the foundation of the Reinforced Earth cap.

7. Riprap the face of the new earth fill section which will require an estimated 800 cubic yards of loose rock.

Reinforced Earth is actually a registered commercial system. The discussion heretofore has been in reference to this particular system. However, other systems capable of performing the same function as Reinforced Earth, are also being considered. Included are; Retained Earth and Double Wall Construction. Neither of these systems appear to have as many advantages as Reinforced Earth. Studies of these other systems will continue as design details are developed.

D. ENGINEERING CONSIDERATIONS

1. Seepage

Seepage studies indicate that if the pool elevation is increased about 10 feet, there will be an increase in seepage from the left abutment. The amount of increase in seepage per lineal foot of drain trench will not be excessive. However, seepage will exit higher on the abutment. Therefore, the drain trench will need to be extended up the left abutment to the elevation of the new normal pool. The increase in seepage collected is expected to be at least proportioned to the increase in the drain trench length. The size of the drain trench pipe, (10-inch on the design drawings) appears sufficient to handle the additional seepage. However, the size and structural condition of the pipe have not been verified.

2. Stability

Slope stability analyses have been performed for the Reinforced Earth alternative. The calculations were performed using the same soil parameters, embankment and cross section (with cap added) as was used in the Static Stability Analysis previously described in Section IX. The Reinforced Earth configuration used is shown on Sheet No. 3 of Appendix A. Computer printouts of the results of the calculations are included in Appendix B and summarized as follows:

<u>Sudden Drawdown</u> (upstream slope)
Factor of Safety FS = 1.16

<u>Steady State Seepage</u> (downstream slope)
Factor of Safety FS = 1.58 (without seismic loading)

The phreatic surface used in the calculations is shown on Sheet No. 3 of Appendix A. In summary, static stability for the Reinforced Earth alternative does not appear to be a problem, provided the phreatic surface through the embankment is properly controlled. This control will be provided by the impervious barrier and drain illustrated on Sheet No. 3 of Appendix A.

Stability of the earth fill alternatives has not been calculated as the design is flexible and may be altered to achieve the desired stability factor of safety. Stability should not be a problem, however, additional stability computations will be performed as appropriate, as the design develops.

3. <u>Outlet Conduit</u>

The outlet conduit is a 60-inch diameter concrete pipe. It is lined with steel along a portion of its length. Its shape changes from round to horseshoe near the outlet.

This conduit was evaluated for both rehabilitation alternatives assuming the embankment crest is raised 10 feet. the calculations took into account the following loading conditions:

- The existing static stress (See Figure 9)
- Induced stress due to increased embankment height
- Induced dynamic stress for the existing embankment assuming the design seismic event

The induced maximum dynamic vertical stresses from the computerized finite element program ISBILD are presented in Figure 13. These stresses are also approximately representative of the Reinforced Earth alternative because the additional stress resulting from the new cap will be small.

In conclusion, the outlet conduit appears to have adequate strength for either rehabilitation alternative loadings. However, additional computations may be needed to estimate the induced dynamic loading on the conduit if the earth fill alternative is selected or if the height of new crest is more than 10 feet.

4. Induced Settlement

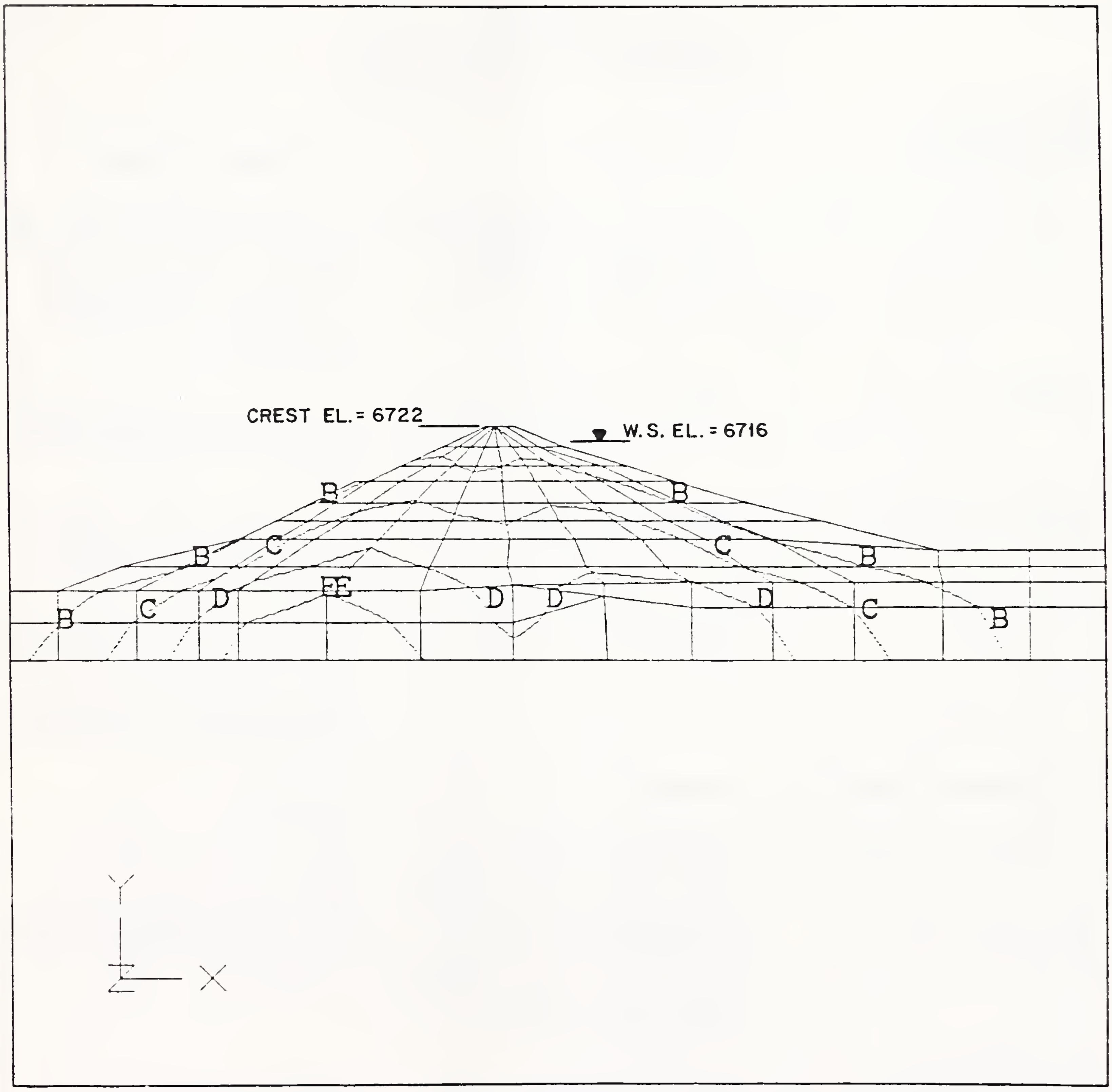
Additional settlement does not appear to be a problem with the reinforced earth alternative. There is a relatively small increase in stresses within the embankment and foundation resulting from the proposed cap.

Additional settlement resulting from placing the earth fill on the downstream slope is expected to be less than 3 inches. While this amount of settlement is tolerable, it is not desirable, particularly at the outlet conduit.

5. Summary

Table 16 summarizes the engineering considerations for the Earth Fill and the Reinforced Earth alternatives.

In conclusion, the Reinforced Earth alternative appears to have more favorable engineering aspects. However, additional study will be required as design details are developed to confirm the technical and economic feasibility.



CONTOUR	MAXIMUM DYNAMIC VERTICAL STRESS, σ_y, in psf
A	0
B	280
C	560
D	840
E	1120
F	1400

Source: Computer generated, see reference 15

MIDDLE CREEK DAM
CONTOURS OF MAXIMUM
DYNAMIC VERTICAL STRESS

8M087.113

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FIGURE 13

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Table 16

Earth Fill vs. Reinforced Earth

Engineering Considerations	Earth Fill	Reinforced Earth
1. Engineering technology	This is the conventional method most understood by the public and the engineers.	This method has been used on only a few dams. However, the technology is well developed and understood by the engineers.
2. Cost	Appears to be comparable to Reinforced Earth.	Appears to be comparable to the Earth Fill alternative.
3. Construction time	At least 2 construction seasons (summer).	It appears that construction could be completed in one season.
4. Asthetics	Asthetics will not change.	The exposed surface of the cap is concrete. The surface can be colored or sculptured to be made more acceptable.
5. Slope stability	Adequate.	Adequate.
6. Induced settlements	This alternative will add considerable weight to embankment. Additional settlements are inevitable but tolerable. The effect of the settlement on the conduit is not desirable.	The induced stresses resulting from the cap will be relatively small. Additional settlement will be minor.
7. Outlet conduit	Appears adequate. However this alternative will induce more stress on the conduit.	Appears adequate even during seismic event.
8. Borrow requirements	Approximately 165,00 CY of embankment fill plus 4000 CY of loose rock riprap are needed.	Approximately 48,000 CY of embankment fill plus 800 CY of loose rock riprap is required.

Source: HKM Associates

XII. RECOMMENDATIONS

A. SHORT-TERM

1. Continue to monitor ground water levels on a regular basis for at least 8 months to establish its approximate relationship with the fluctuation in the pool surface.

2. Restore the impervious blanket on the left abutment upstream of the embankment.

3. Collect and monitor the seepage volume and turbidity from the left abutment. The seepage volume from the right abutment should also be monitored.

B. LONG-TERM

1. Either of the following rehabilitation alternatives may be used to increase the elevation of the embankment.

   Earth Fill - Raise the crest by placing an earth fill on the crest and over the downstream face. This alternative would require extension of the outlet conduit.

   Reinforced Earth - Raise the crest by placing a Reinforced Earth cap from the left abutment approximately 1200 feet. The remaining crest may be raised by placing an Earth Fill section over the downstream face. A impervious barrier upstream of the Reinforced Earth wall is also required.

2. For either rehabilitation alternative the existing trench drain system should be maintained. The drain should be extended, at the upper end, to the elevation of the new pool (assuming the pool will be raised).

3. As design details develop, this contractor shall perform additional seepage and stability analysis to evaluate these designs.

4. For either rehabilitation alternative, at least two additional monitoring holes are needed. These may be installed at the time of construction. One should be located at the downstream edge of the crest and the other should be located at the berm on the downstream face. The purpose of these holes is to monitor the phreatic surface. These holes are approximately located on Sheet 2 in Appendix A.

5. For either rehabilitation alternative, maintain as many of the monitoring holes as is feasible.

6. Establish a long term seepage monitoring system for both abutments.

XIII. REFERENCES

1. Algermissen and Perkins, "A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States", U.S. Department of Interior Geological Survey, Open File Report 76-416, 1976.

2. Ballard, J.H., "Strong-Motion Accelerograms of Montana Earthquakes", Northwest Geology, Volume 8, 1979.

3. Bonnilla, M.G., Surface Faulting and Related Effects, in Earthquake Engineering, Prentice-Hall, New York, New York, 1970.

4. Bowles, Joseph E., Engineering Properties of Soils and Their Measurement, McGraw-Hill Inc., 1970.

5. Bureau of Reclamation, Design of Small Dams, Second Edition, 1974.

6. Bureau of Reclamation, Earth Manual, U.S. Department of Interior, First Edition, Revised Second Printing, 1968.

7. Chadwick, R.A., Igneous Geology of the Fridley Peak Quadrangle, Montana, Geologic Map 31, Montana Bureau of Mines and Geology, Montana College of Mineral Science and Technology, Butte, Montana, 1982.

8. Corps of Engineers, Engineering and Design Stability of Earth and Rock-fill Dams, Department of the Army, EM 1110-2-1902, April, 1970.

9. Corps of Engineers, Finite Element Solution of Steady State Potential Flow Problems, Department of the Army, Hydrologic Engineering Center, Program No. 723-G2-L2440, 1970.

10. Corps of Engineers, Office of the Chief Engineers Report to the U.S. Congress, National Program of Inspection of Dams, Vol. 1, Appendix D, "Recommended Guidelines for Safety Inspection of Dams, Washington, D.C., Department of the Army, May, 1975.

11. Corps of Engineers, Phase I Inspection Report, National Dam Safety Program, Middle Creek Dam, Bozeman, Montana, Gallatin County, MT-18, April 1980.

12. Corps of Engineers, Seismic Risks Map of the United States, Department of the Army, 1983.

13. Corps of Engineers, Slope Circle Slope Stability with Side Forces, Department of the Army, Reference File No. 741-11-F5030, December, 1968.

14. Hackett, O.M., Visher, F.N., McMurtrey, R.G., and Steinhilber, W.L., Geology and Ground Water Resources of the Gallatin Valley, Gallatin County Montana, United States Geological Survey Water Supply Paper No. 1482, United States Government Printing Office, Washington, D.C., 1960.

15. Idriss, Lysmer, Hwany and Seed, "Quad 4", National Information Service for Earthquake Engineers, NISEE Computer Application, University of California, Berkeley, Earthquake Engineering Research Center, Report No. EERC 73-16, July, 1973.

16. Horberg, L., Geomorphic Problems and Glacial Geology of the Yellowstone Valley, Park County, Montana, Journal of Geology, Vol. 48, No. 3, pp. 275-303, 1940.

17. Howard, Terry R. (Editor), Seismic Design of Embankments and Caverns [Proceedings of a symposium sponsored by the ASCE Geotechnical Engineering Division in conjunction with the ASCE National Convention, Philadelphia, Pennsylvania, May 16-20, 1983] ASCE, New York, New York.

18. Krinitesky, E.L., and Marcuson III, W.F., Principles for Selecting Earthquake Motions in Engineering Design, Bulletin of the Association of Engineering Geologists, Vol. 20, No. 3, pp. 253-265, 1983.

19. Makdisi and Seed, "Simplified Procedures for Estimating Dam and Embankment Earthquake-Induced Deformation", Journal of Geotechnical Engineering, July, 1978.

20. McMannis, W.J., and Chadwick, R.A., Geology of the Garnet Mountain Quadrangle Gallatin County, Montana, Bulletin 43, Montana Bureau of Mines and Geology, Montana College of Mineral Science and Technology, Butte, Montana, 1964.

21. Montague, J., and Chadwick, R.A., Cenozoic History of the Yellowstone Valley South of Livingston, Montana, Department of Earth Sciences Publication 12, Montana State University, Bozeman, Montana, 1982.

22. Montana Department of Natural Resources and Conservation, Middle Creek Design File, Engineering Bureau, 32 South Ewing, Helena, Montana, 1984.

23. Ozawa and Duncan, "ISBILD: A Computer Program for Analysis of Static Stresses and Movements in Embankments", Department of Civil Engineering Institute of Transportation and Traffic Engineering, University of California, Berkeley, Report No. TE-73-4.

24. Quamar, A.I., and Stickney, M.C., Montana Earthquakes, 1969-1979, Historical Seismisity and Earthquake Hazard, Memoir 51, Montana Bureau of Mines and Geology, Montana College of Mineral Science and Technology, Butte, Montana, 1983.

25. Roberts, A.E., Cretaceous and Early Tertiary Depositional and Tectonic History of the Livingston Area, Southwestern Montana, United States Geological Survey Professional Paper No. 526-c, U.S. Government Printing Office, Washington, D.C., 1972.

26. Seed and Idriss, Rock Motion Accelerograms for High Magnitude Earthquakes, University of California, Berkeley, Earthquake Engineering Research Center.

27. Seed and Idriss, "Soil Moduli and Damping Factors for Dynamic Response Analyses", University of California, Berkeley, Earthquake Engineering Research Center, Report No. EERC 70-10, December, 1970.

28. Seed, Idriss and Arango, "Evaluation of Liquefaction Potential Using Field Performance Data". Seismic Design of Embankments and Caverns. [Proceedings of a symposium sponsored by the ASCE Geotechnical Engineering Division in conjunction with the ASCE National Convention, Philadelphia, Pennsylvania, May 16-20, 1983] ASCE, New York, New York.

29. Sherard, Woodward, Gizienski and Clevenger, Earth and Earthrock Dams, 1963.

30. Seed, Idriss, Makdisi and Baugerjee, "Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analysis", University of California, Berkeley, Earthquake Engineering Research Center, Report No. EERC 75-29, October, 1975.

31. Vellinga, John, Middle Reservoir pool elevation provided to HKM Associates by Mr. John Vellinga by telephone conversation, February, 1984.

32. Weber, M.M., General Geology and Geomorphology of the Middle Creek Area, Gallatin County, Montana, Unpublished M.S. Thesis, Montana State College, Bozeman, Montana, 1965.

APPENDIX A

LEGEND

SLOPE	ANGLE	EXPLANATION
	>25°	These slopes are nearly at or exceed the natural angle of repose Presently, stability is provided by vegetation Any change in the present conditions of the slope may cause instability
	15° – 25°	These slopes are at less than or approach the natural angle of repose Under saturated conditions with a triggering mechanism (earthquake, rapid dewatering, etc), slope movement may occur
	<15°	These slopes are at less than the natural angle of repose There is minimal chance of instability on these slopes

NOTE

The valley walls in the reservoir consist of glacial moraine deposits which generally consist of heterogeneous gravel and cobble in a clay matrix The valley floor above the reservoir consists of alluvial silt, sand and gravel

0 200 400 800 1200

GRAPHIC SCALE IN FEET
ORIGINAL SCALE 1" = 400'
CONTOUR INTERVAL = 2'
ELEVATION IN NGVD

Ground Control Surveys Conducted August 1983 by HKM Assoc Billings, MT

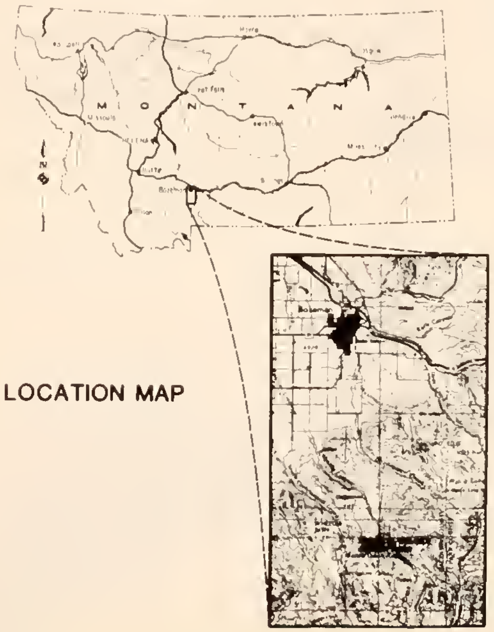
Topography prepared From Aerial Photography August 26, 1983, 1:12,000 Associated Surveys Inc Billings MT

LEGEND

PAVED ROAD	TREES
IMPROVED ROAD	DRAINAGE
TRAIL	POND
BRIDGE	SWAMP
CATTLE GUARD	INDEX CONTOUR
CULVERT	INTERMEDIATE CONTOUR
FENCE	DEPRESSION CONTOUR
RAILROAD	BUILDING
POWER POLE	DEPTH OF SUBSIDENCE
TP TEST PIT	

(NOTE All other test pits and drill holes are located on Sheet No 2 of 7)

LOCATION MAP



Project No 8M087-113 | I.D. No | Date FEB 1984 | Designed DCN Drawn JAE Checked RJP Approved

No	Revision	By	Date
1			

MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION

RESERVOIR PLAN, LOCATION AND SLOPE STEEPNESS MAP

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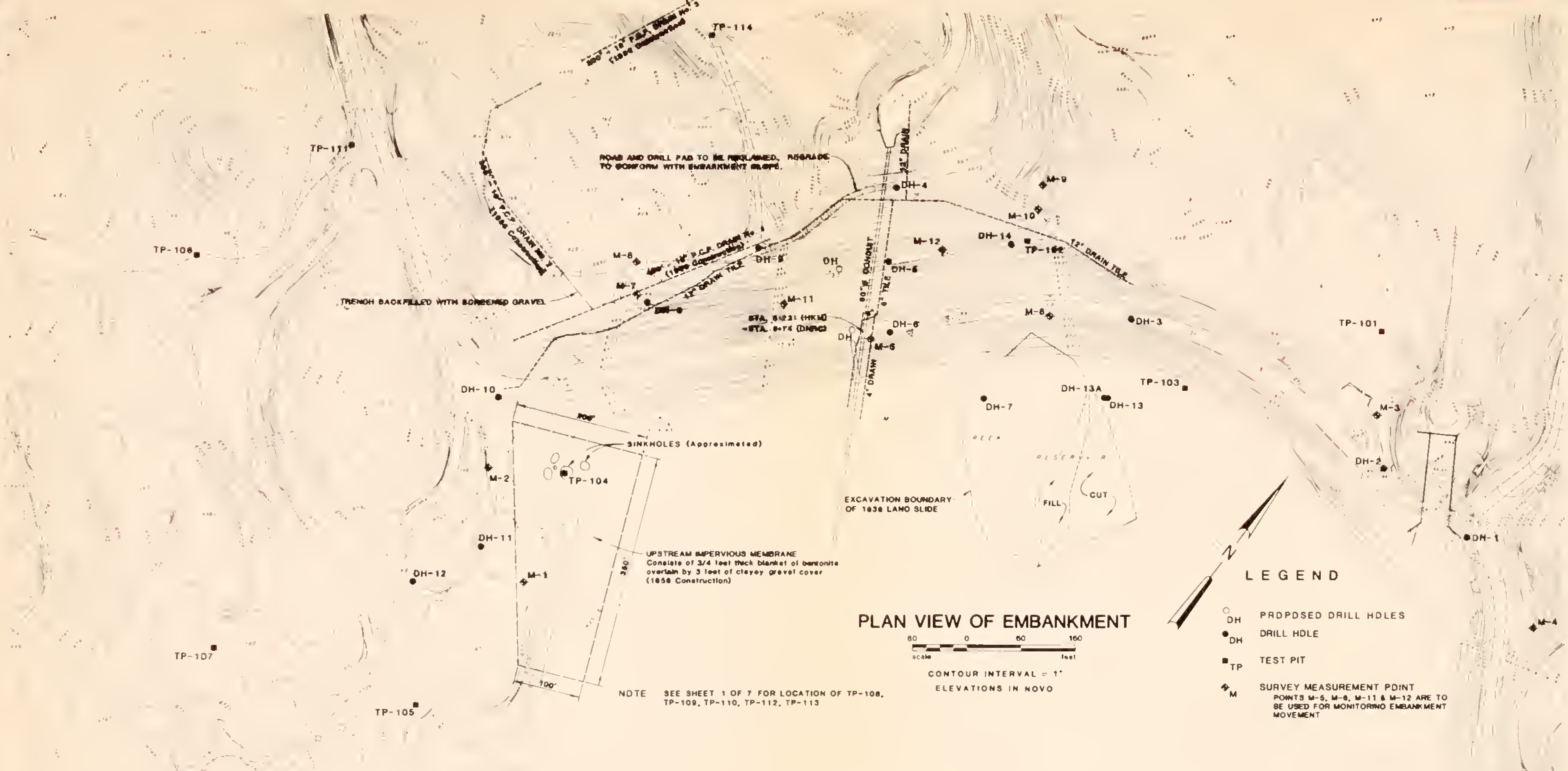
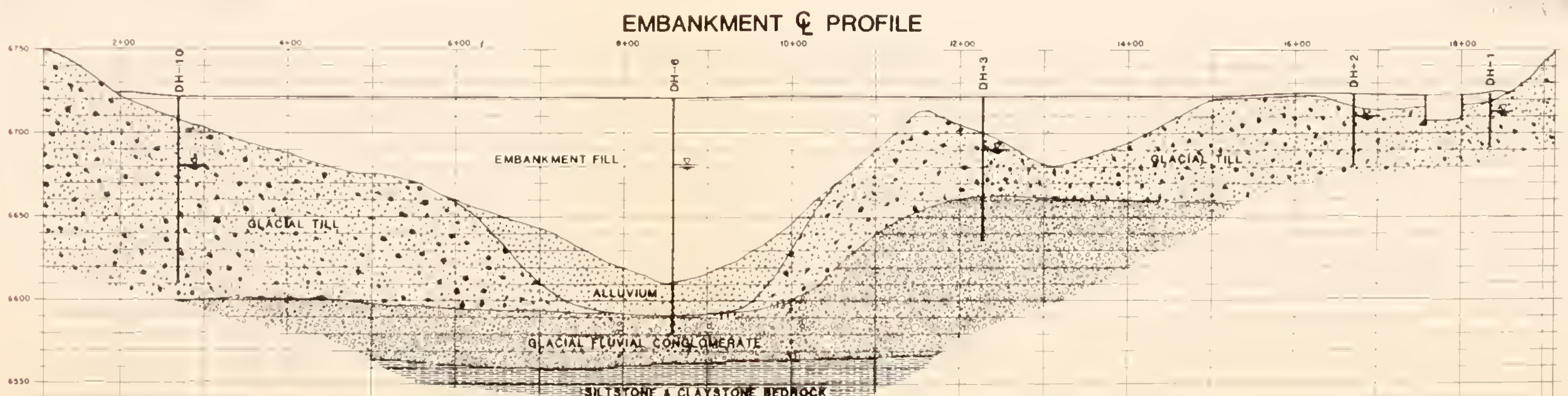
Sheet No 1 of 7





DRILL HOLE	COORDINATES NORTH	COORDINATES EAST
1	13094.13	6314.70
2	13088.47	6060.07
3	13048.82	4917.22
4	12880.80	4220.88
5	12866.06	4276.86
6	12914.10	4343.80
7	13019.34	4914.30
8	13031.20	4036.12
9	12783.31	4109.84
10	12382.70	3930.38
11	12180.76	4082.44
12	12085.88	4003.84
13	13039.83	4880.04
13A	13021.?	4892.?
14	13037.33	4408.08

NOTE. THE LOCATIONS OF THE TEST PITS ARE APPROXIMATED

Project No 8M087 113 | I.D. No | Date FEB 1984 | Designed DLD | Drawn GAE | Checked RJP | Approved

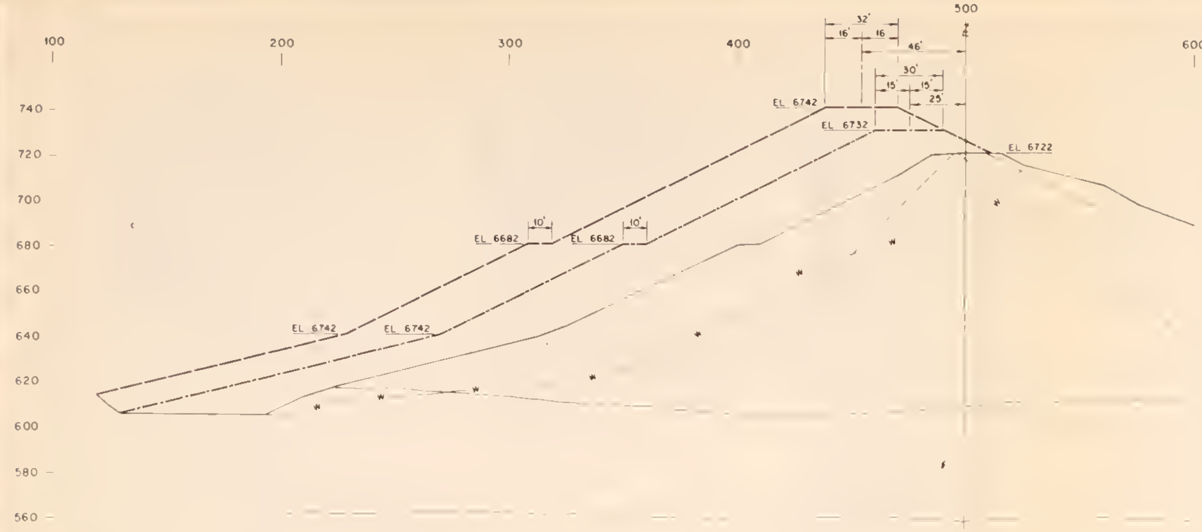
No	Revision	By	Date
1		DLD	4/10/84

MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION
EMBANKMENT PLAN AND PROFILE

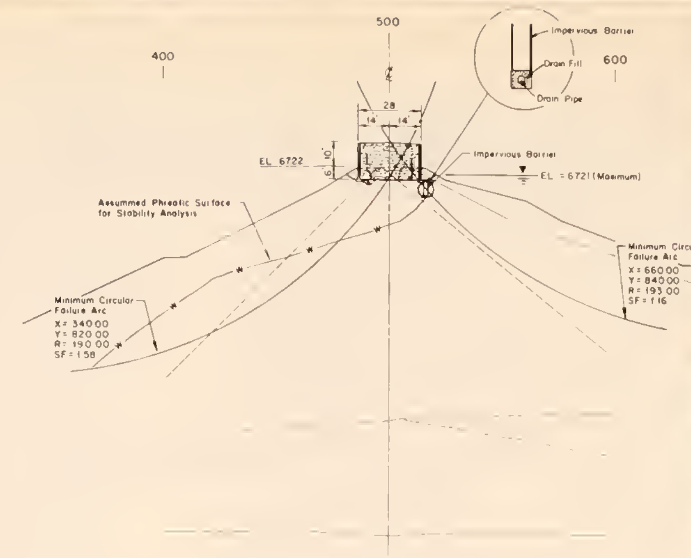
HKM ASSOCIATES
ENGINEERS·PLANNERS
Airport Industrial Park
P.O. Box 31318
Billings, Montana 59107

Branch Offices
• Libby, Montana
• Sheridan, Wyoming

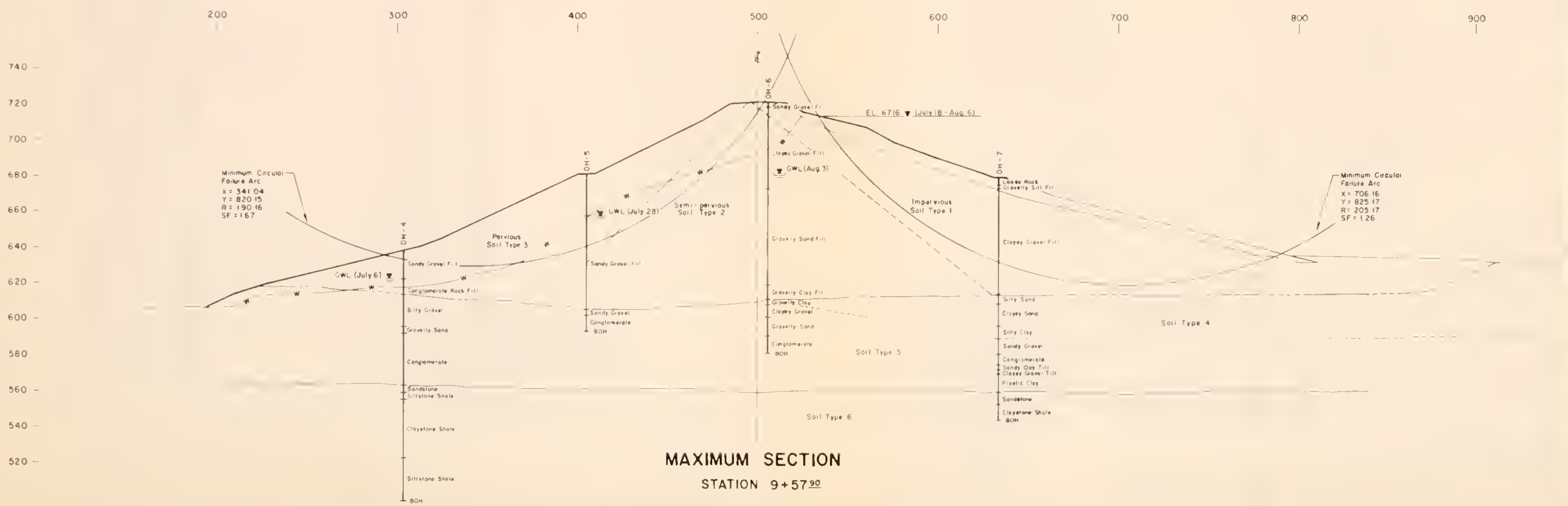
Sheet No 2 of 7



EARTH FILL ALTERNATIVES



REINFORCED EARTH ALTERNATIVE



MAXIMUM SECTION
STATION 9+57.90

MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION
CROSS SECTIONS

Project No 8M087.113 · Date FEB 1984

HKM ASSOCIATES
ENGINEERS·PLANNERS

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Billings, Montana 59107

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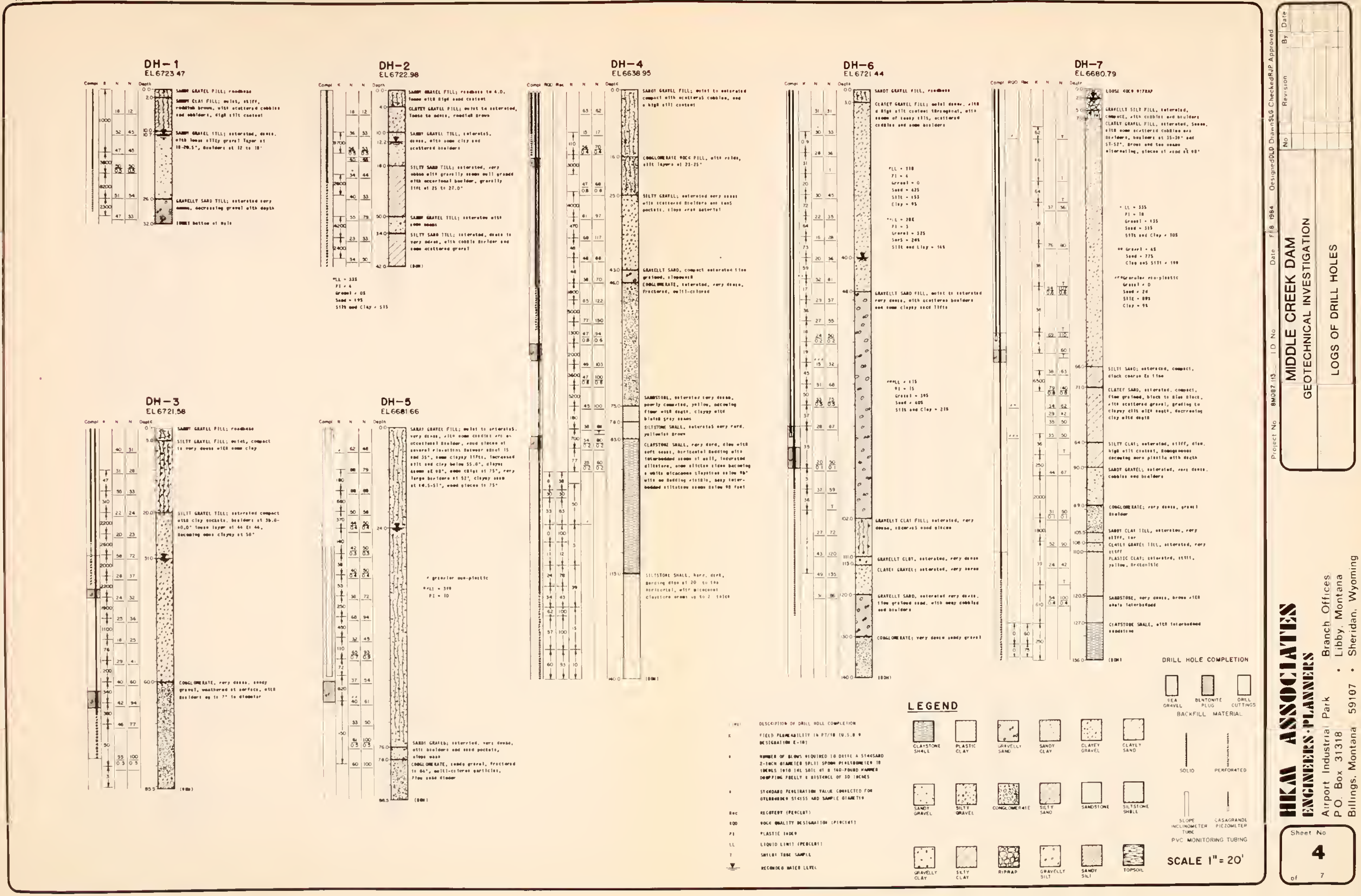
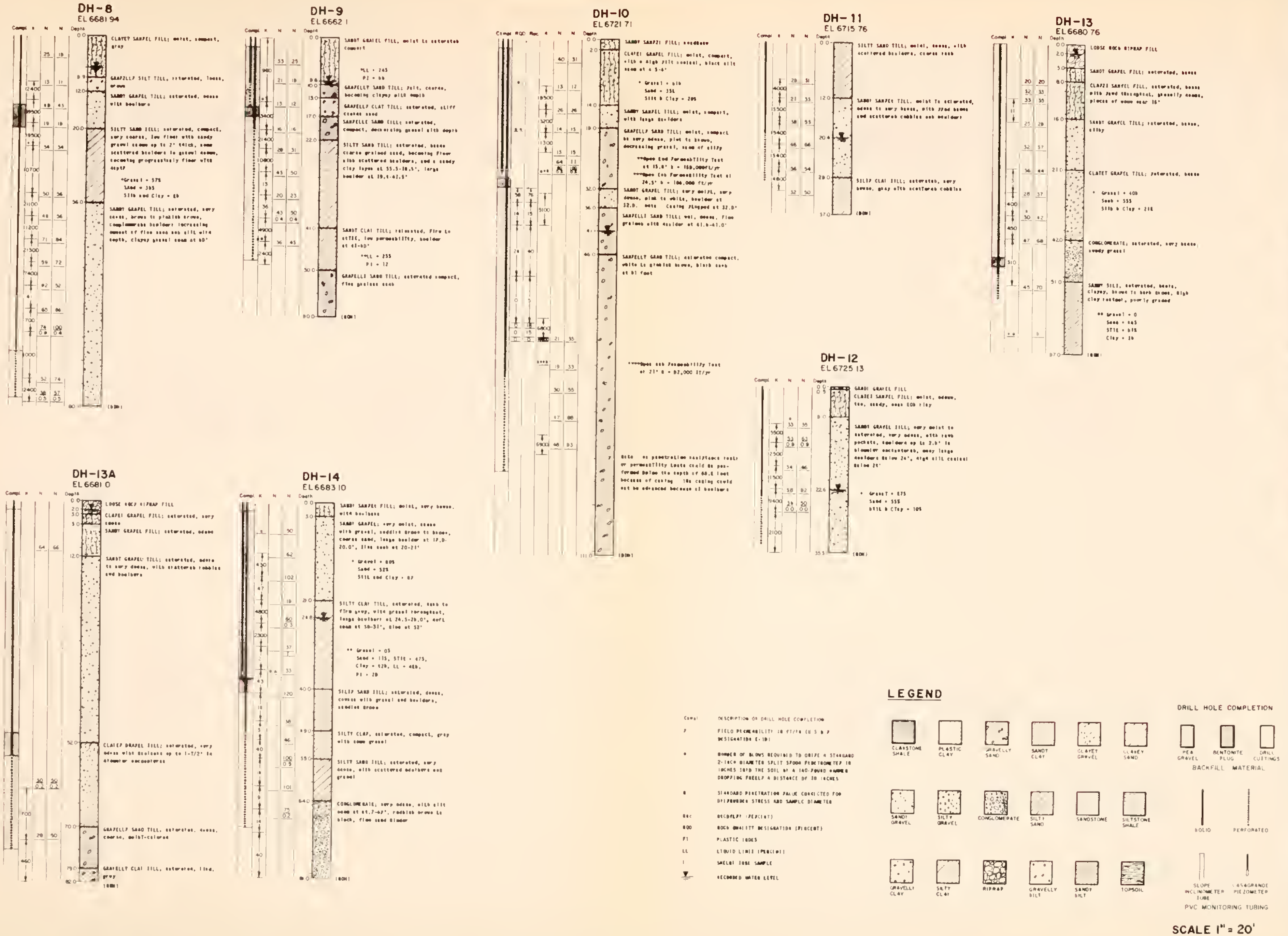
Sheet No
3
of 7


DH-1
EL 6723.47
DH-2
EL 6722.98
DH-4
EL 6638.95
DH-6
EL 6721.44
DH-7
EL 6680.79
DH-3
EL 6721.58
DH-5
EL 6681.66
LEGEND
CLAYSTONE SHALE
PLASTIC CLAY
GRAVELLY SAND
SANDY CLAY
CLAYEY GRAVEL
CLAYEY SAND
SANDY GRAVEL
SILTY GRAVEL
CONGLOMERATE
SILTY SAND
SANDSTONE
SILTSTONE SHALE
GRAVELLY CLAY
SILTY CLAY
RIPRAP
GRAVELLY SILT
SANDY SILT
TOPSOIL
DRILL HOLE COMPLETION
PEA GRAVEL
BENTONITE PLUG
DRILL CUTTINGS
BACKFILL MATERIAL
SOLID
PERFORATED
SLOPE INCLINOMETER TUBE
CASAGRANDE PIEZOMETER
PVC MONITORING TUBING
SCALE 1" = 20'
HKM ASSOCIATES
ENGINEERS·PLANNERS
Airport Industrial Park
P.O. Box 31318
Billings, Montana 59107
Branch Offices
Libby, Montana
Sheridan, Wyoming
MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION
LOGS OF DRILL HOLES
Project No 8M087.113
Date FEB 1984
Designed DLD
Drawn SLG
Checked RJP
Approved
Sheet No 4 of 7



Designed DLD Drawn SLG Checked RJP Approved

Project No 8M087.113 | I.D. No | Date FEB 1984

No | Revision | By | Date

MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION

LOGS OF DRILL HOLES

HKM ASSOCIATES
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Billings, Montana 59107

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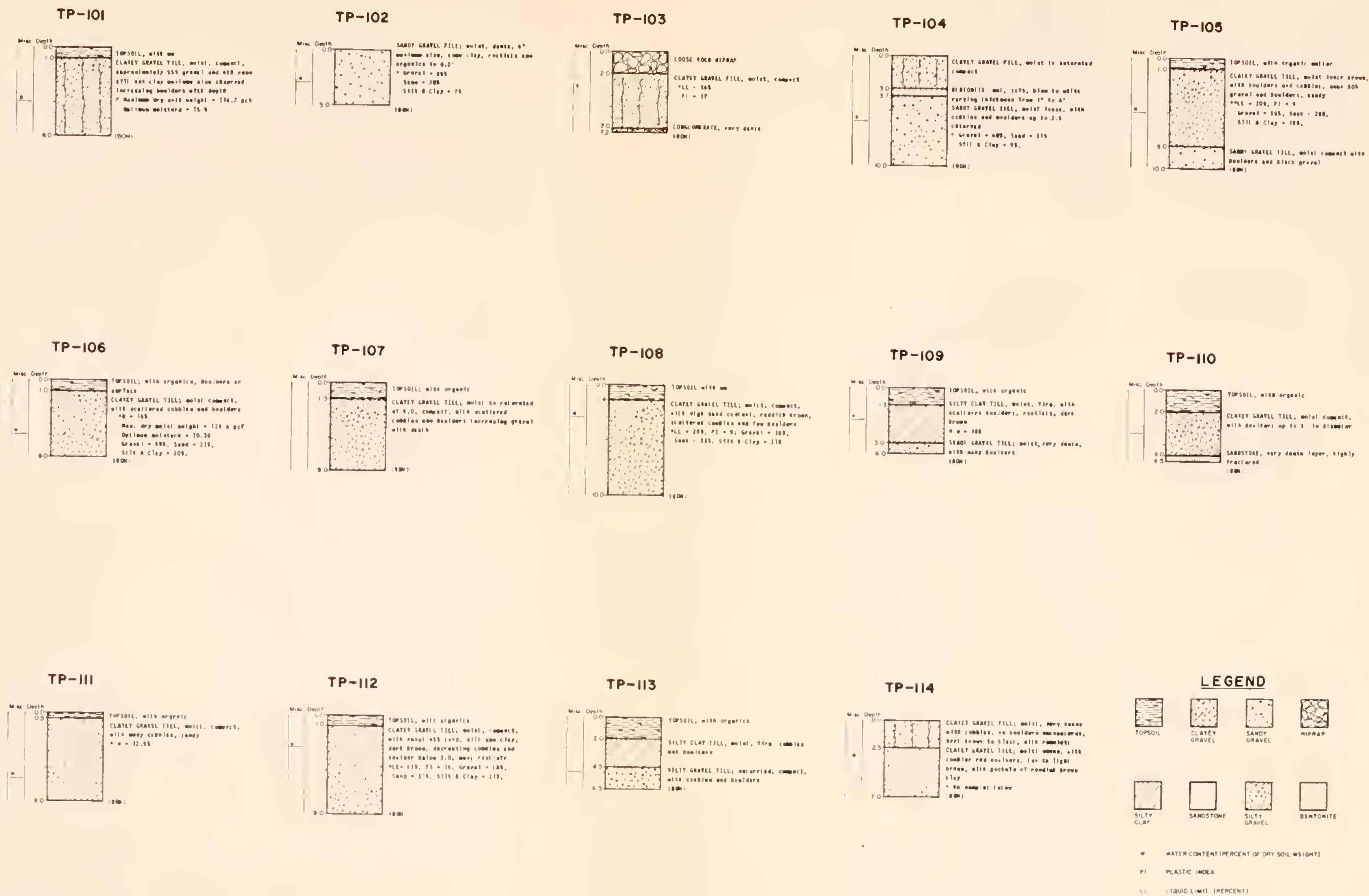
Sheet No 5 of 7



SCALE 1" = 10'

MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION
LOGS OF TEST PITS

Project No 8M087 113 | I.D. No | Date Feb 1984 | Designed DLD | Drawn SLG | Checked RJP | Approved

No | Revision | By | Date

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ENGINEERS-PLANNERS
Airport Industrial Park
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Billings, Montana 59107
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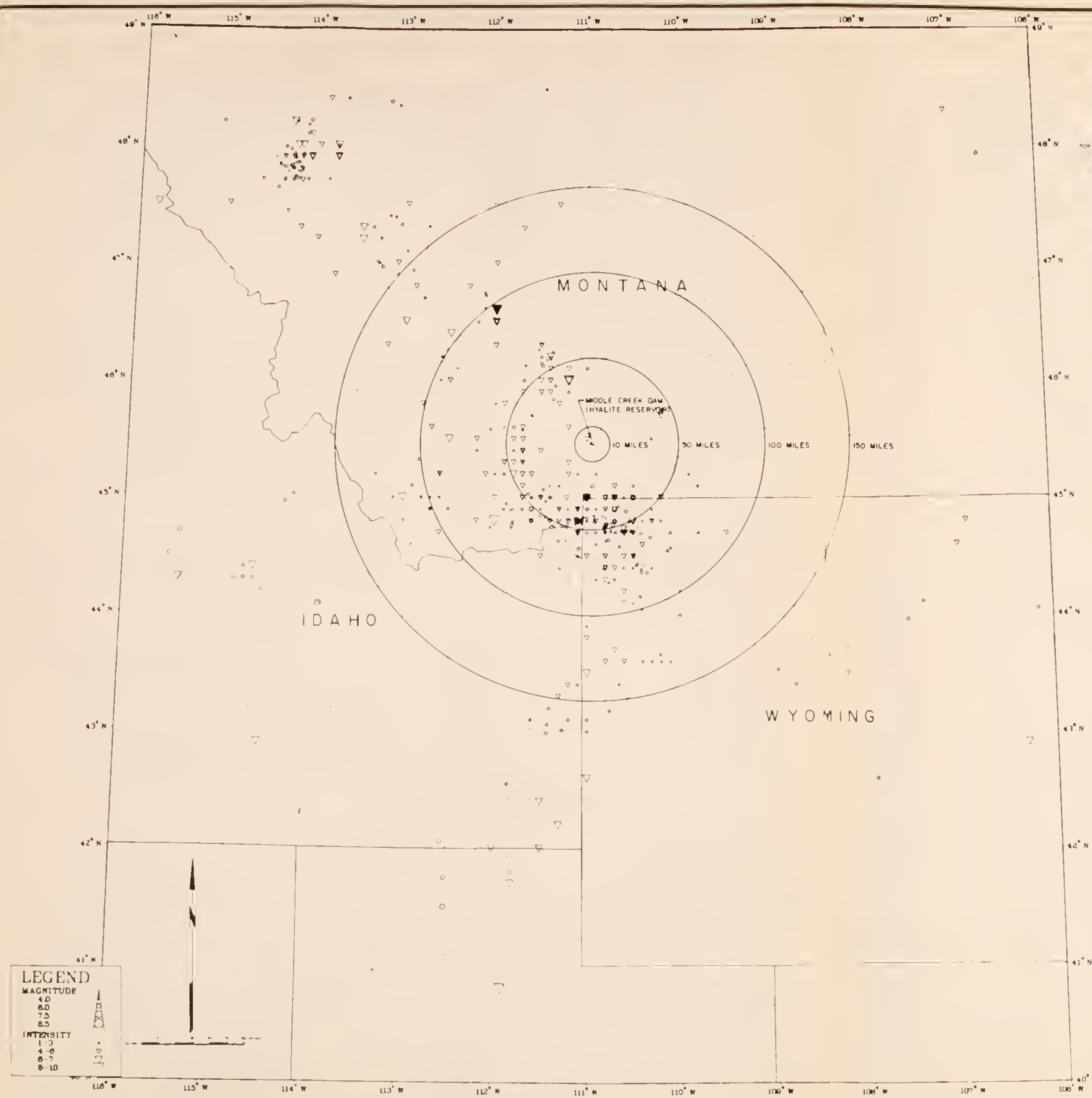
Sheet No 6 of 7



SEISMIC PROBABILITY		
ZONE	DAMAGE	COEFF (g)
0	NONE	0
1	MINOR	0.025
2	MODERATE	0.05
3	MAJOR	0.10
	GREAT	0.15

SEISMIC ZONE MAP of CONTIGUOUS STATES

From SEISMIC RISKS MAP OF USA U.S. Army 1982

RADIUS LEGEND

RADIAL DISTANCE FROM FAULT (MILES)	PROBABLE ATTENUATION OF DAMAGING ACTION MIDDLE CREEK DAM MAY EXPERIENCE
0-10	NO DIMINUTION OF THE DAMAGING MOVEMENTS EXPERIENCED AT THE FAULT
AT 50	50 TO 75 PERCENT OF THE DAMAGE WHICH COULD BE EXPERIENCED NEAR THE FAULT
AT 150	LITTLE DAMAGE COMPARED TO WHAT COULD BE EXPECTED NEAR THE FAULT
○	10 MILE RADIUS FROM MIDDLE CREEK DAM

Legend Source: James L. Sherard, ScD. et al. 1963, EARTH AND EARTH-ROCK DAMS, John Wiley and Sons, Inc.

Project No 8M087 113 I.D. No Date FEB 1984 Designed DCN Drawn GAE Checked RJP Approved

No	Revision	By	Date

MIDDLE CREEK DAM
GEOTECHNICAL INVESTIGATION
SEISMICITY MAP

HKM ASSOCIATES
ENGINEERS — PLANNERS

Airport Industrial Park
P O Box 31318
Billings Montana 59107

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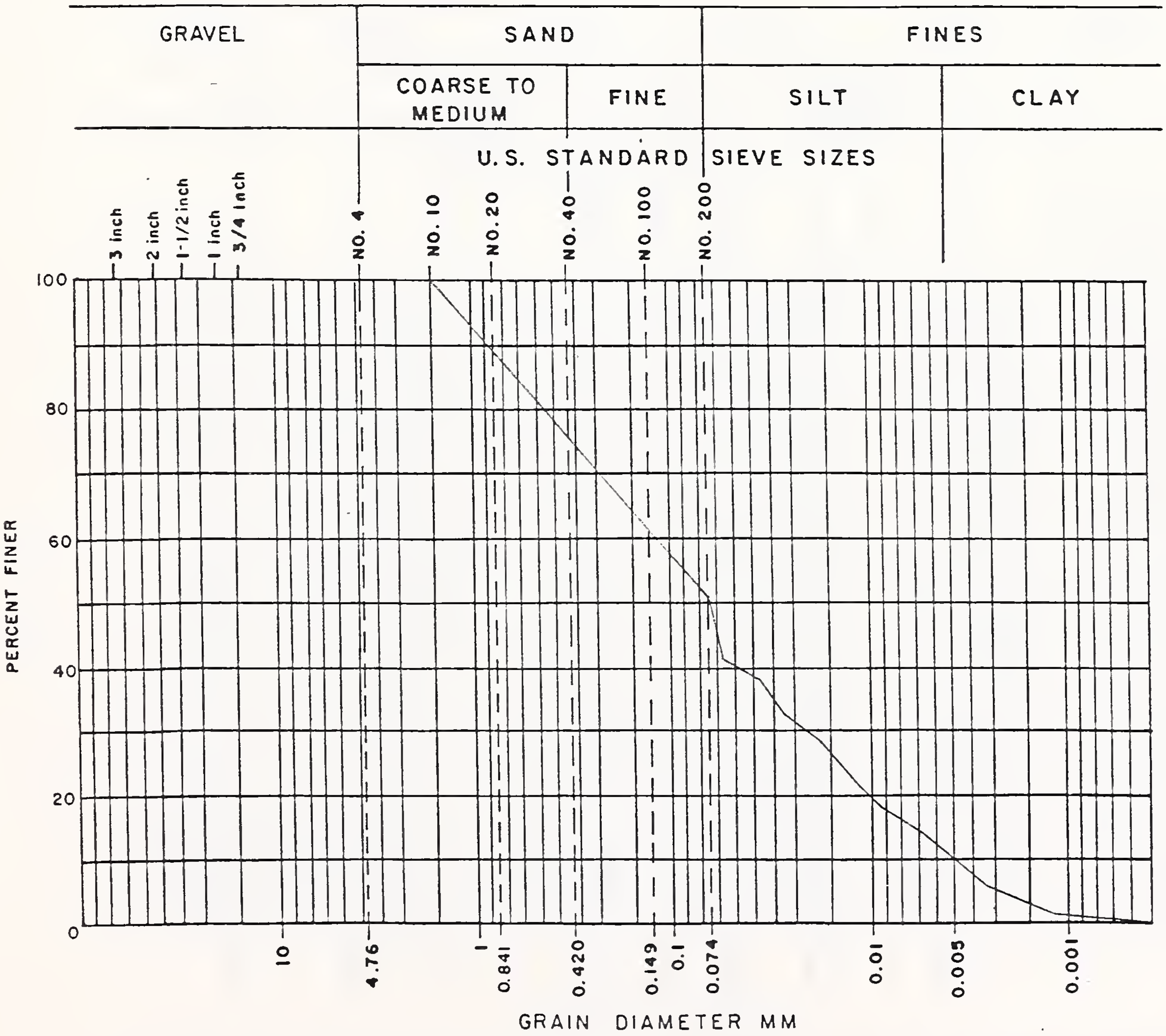
Sheet No
7
of 7

FORM ST-5-80
2869

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO.	8M087.113	LAB NO.	2869
PROJECT NAME	Middle Creek Dam	FIELD NO.	DH-2 (20.0-22.0')
DATE SAMPLED	8-3-83	DATE TESTED	9-23-83
SAMPLED BY	DLD	TESTED BY	CS
TYPE SAMPLE	Drive	SAMPLE LOCATION	Bridge Abutment
SOIL DESCRIPTION	SANDY SILT (ML)		



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve	0 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve.	49 %
3)_ SILT & CLAY, Passing no. 200 sieve	51 %

Liquid Limit = 33%
Plastic Index = 4

Plate No. 1

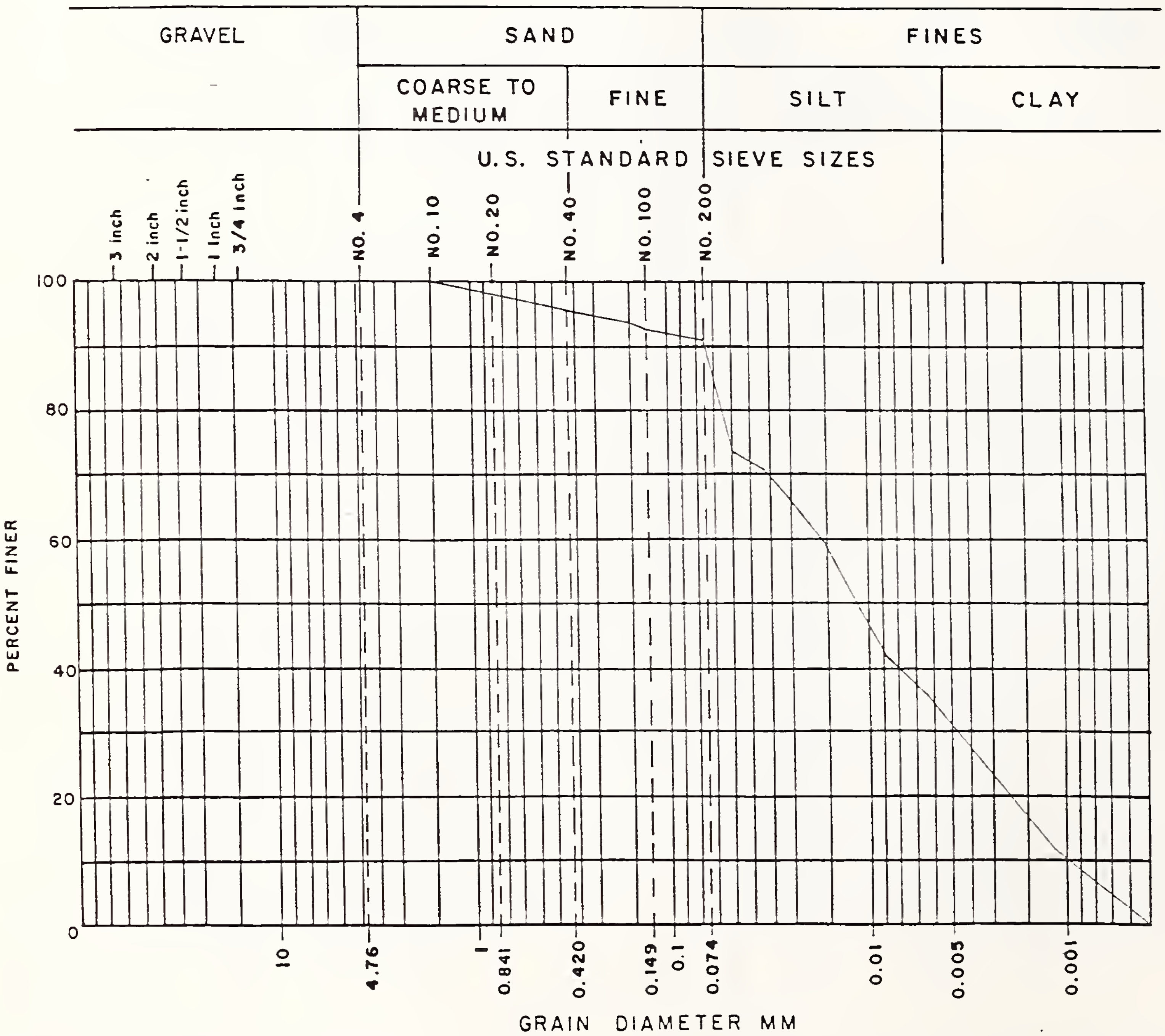
FORM ST-5-80

2896

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO.	8M087.113	LAB NO.	2896
PROJECT NAME	Middle Creek Dam	FIELD NO.	DH-4 (103.8-104.8')
DATE SAMPLED	8-10-83	DATE TESTED	9-21-83
SAMPLED BY	DN	TESTED BY	CS
TYPE SAMPLE	Core	SAMPLE LOCATION	Bedrock at Downstream
SOIL DESCRIPTION	CLAYSTONE SHALE (CL-2)		



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 0 %

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 8 %

3)_ SILT & CLAY, Passing no. 200 sieve 92 %

Liquid Limit = 49%
Plastic Index = 21

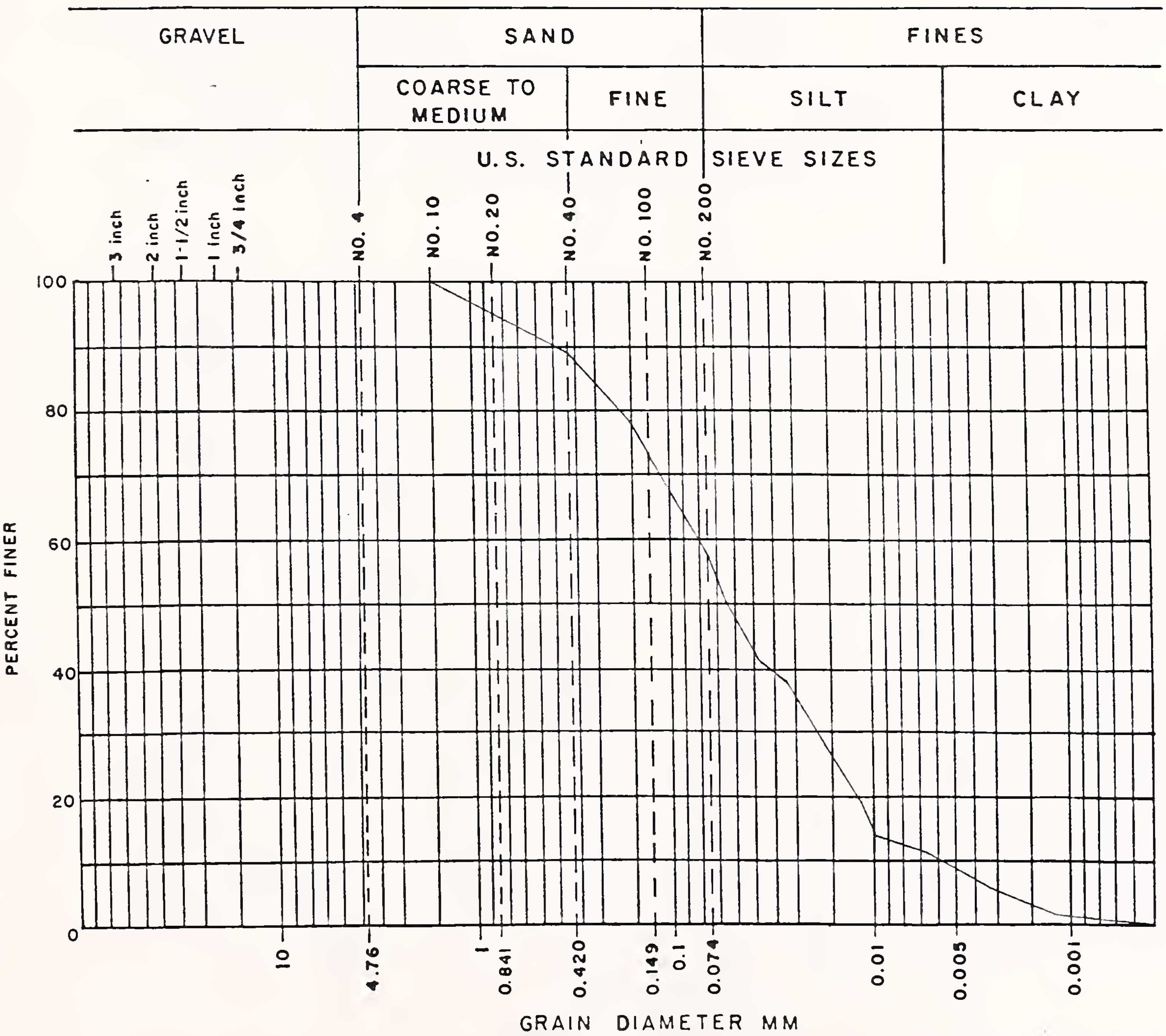
Plate No. 2

FORM ST-5-80
2912

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 2912
PROJECT NAME Middle Creek Dam FIELD NO. DH-6 (10.0-12.0')
DATE SAMPLED 8-3-83 DATE TESTED 9-23-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Drive SAMPLE LOCATION Semi-pervious Zone
SOIL DESCRIPTION SANDY SILT (ML)



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve ___0___%
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. ___42___%
3)_ SILT & CLAY, Passing no. 200 sieve ___58___%

Liquid Limit = 31%
Plastic Index = 4

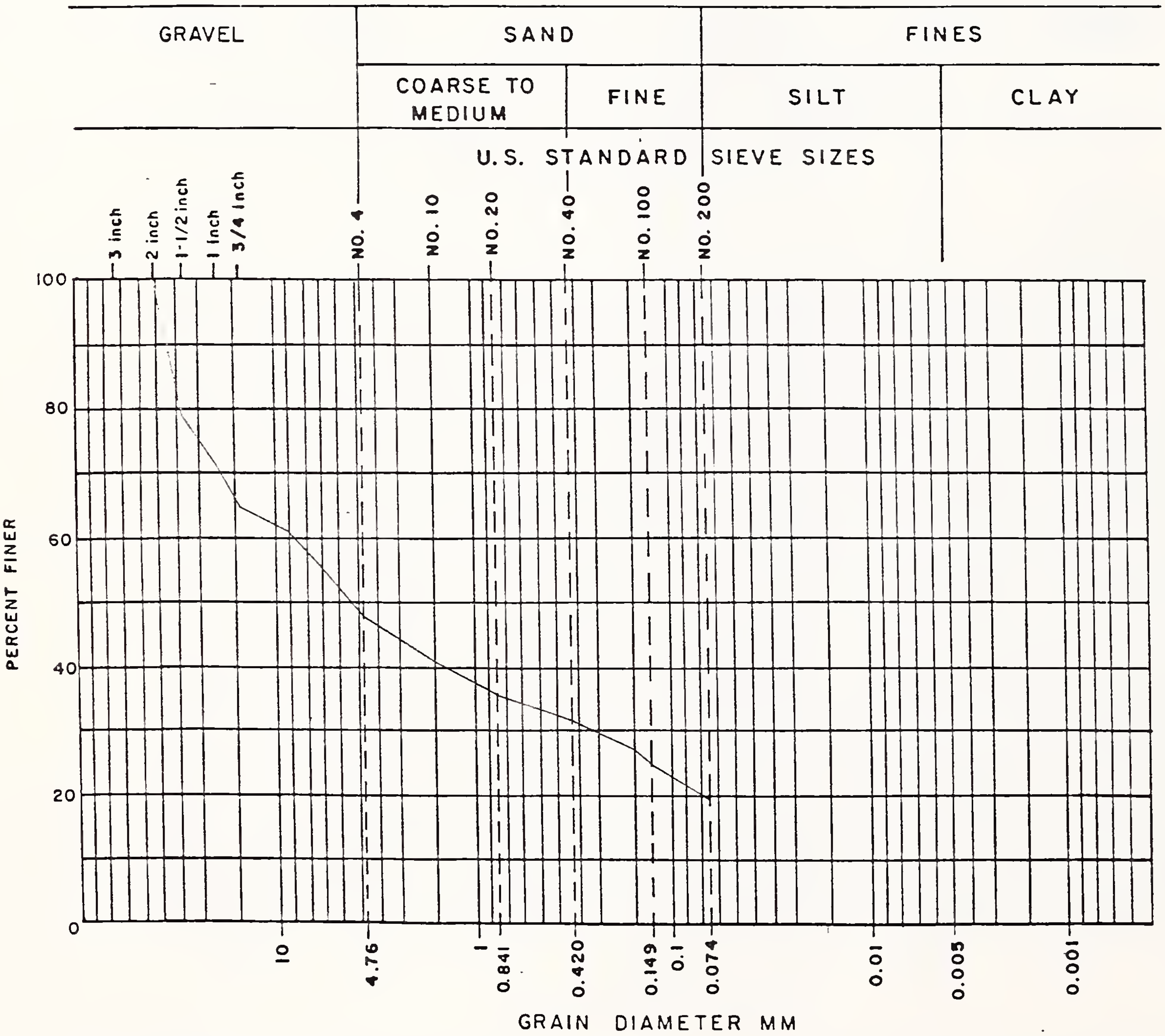
Plate No. 3

FORM ST-5-80
2916

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO.	8M087.113	LAB NO.	2916
PROJECT NAME	Middle Creek Dam	FIELD NO.	DH-6 (45.0 - 47.1)
DATE SAMPLED	8-4-83	DATE TESTED	10-13-83
SAMPLED BY	DLD	TESTED BY	CS
TYPE SAMPLE	Drive	SAMPLE LOCATION	Semi-pervious Zone
SOIL DESCRIPTION	SILTY GRAVEL (GM)		



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve	52 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve.	29 %
3)_ SILT & CLAY, Passing no. 200 sieve	19 %

Liquid Limit = 28%
Plastic Index = 3

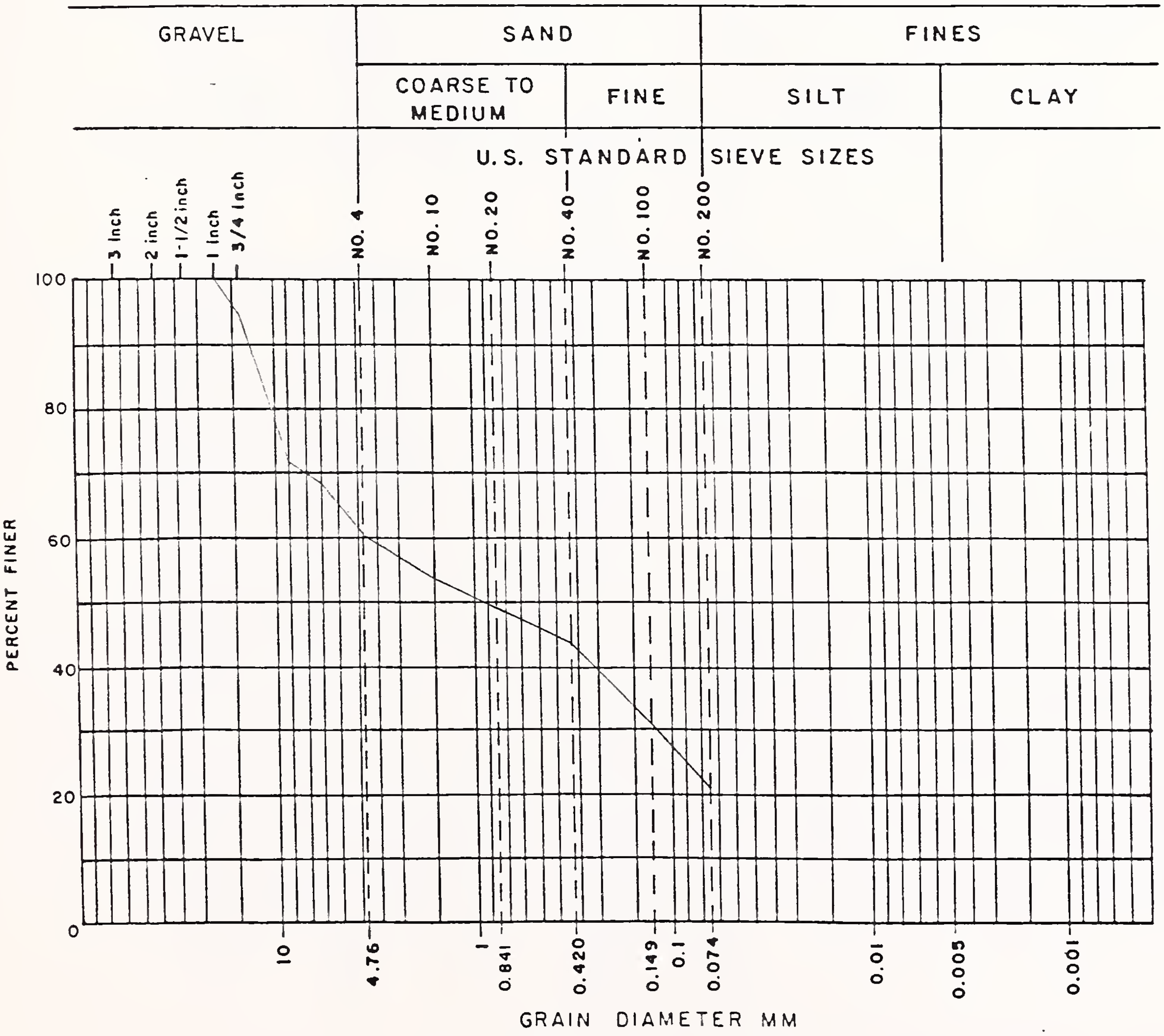
Plate No. 4

FORM ST-5-80
2918

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 2918
PROJECT NAME Middle Creek Dam FIELD NO. DH-6 (65.0-66.5')
DATE SAMPLED 8-3-83 DATE TESTED 10-13-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Drive SAMPLE LOCATION Semi-pervious Zone
SOIL DESCRIPTION CLAYEY SAND (SC)



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 39 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 40 %
3)_ SILT & CLAY, Passing no. 200 sieve 21 %

Liquid Limit = 41%
Plastic Index = 15

Plate No. 5

FORM ST-5-80

3010, 11 & 12

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3010

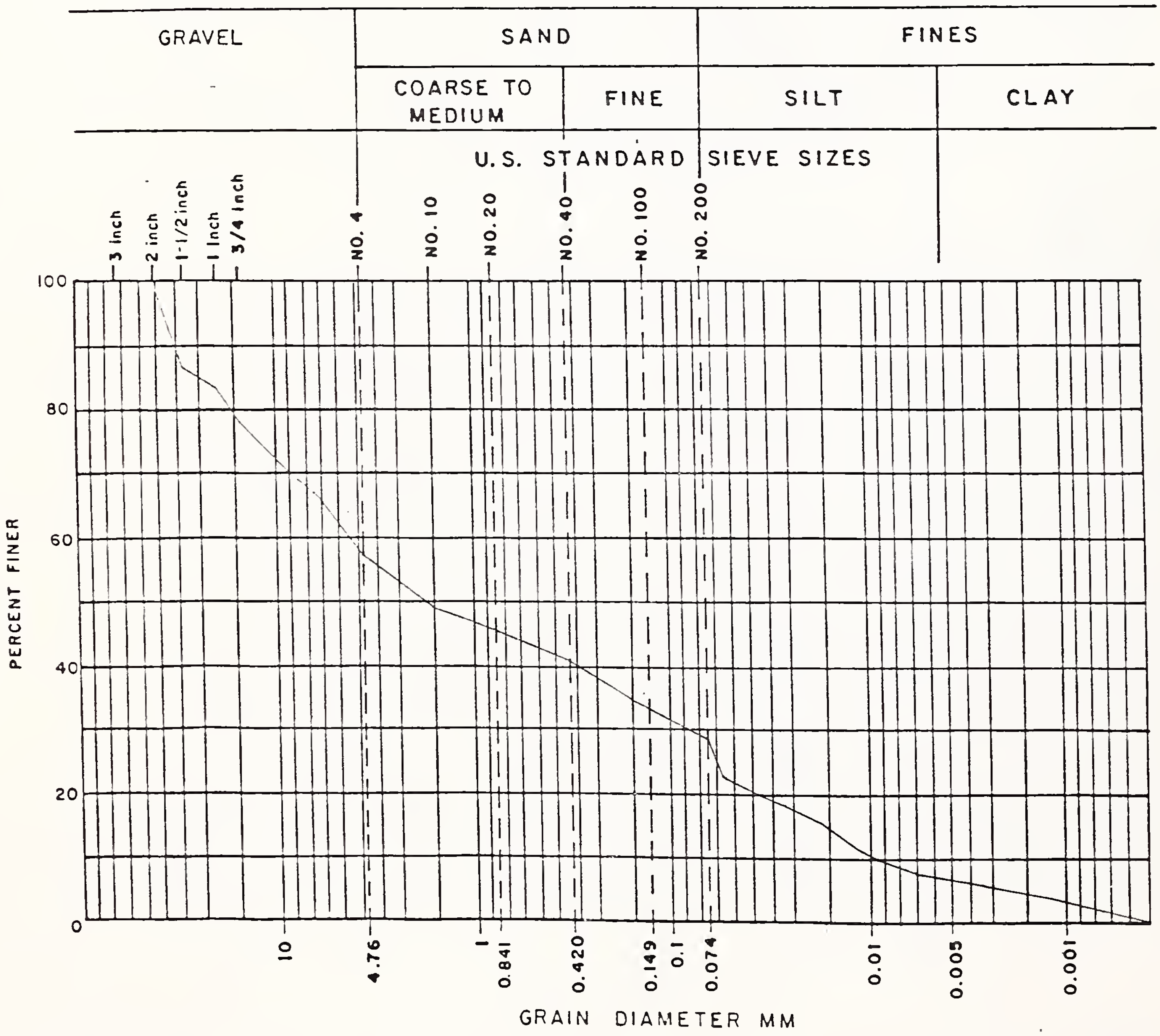
PROJECT NAME Middle Creek Dam FIELD NO. DH-7 (12.0-12.5')

DATE SAMPLED 9-28-83 DATE TESTED 10-21-83

SAMPLED BY DLD TESTED BY JP

TYPE SAMPLE Drive SAMPLE LOCATION Impervious Zone

SOIL DESCRIPTION CLAYEY GRAVEL (GC)



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 43 %

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 28 %

3)_ SILT & CLAY, Passing no. 200 sieve 29 %

Liquid Limit = 33%
Plastic Index = 18

Plate No. 6

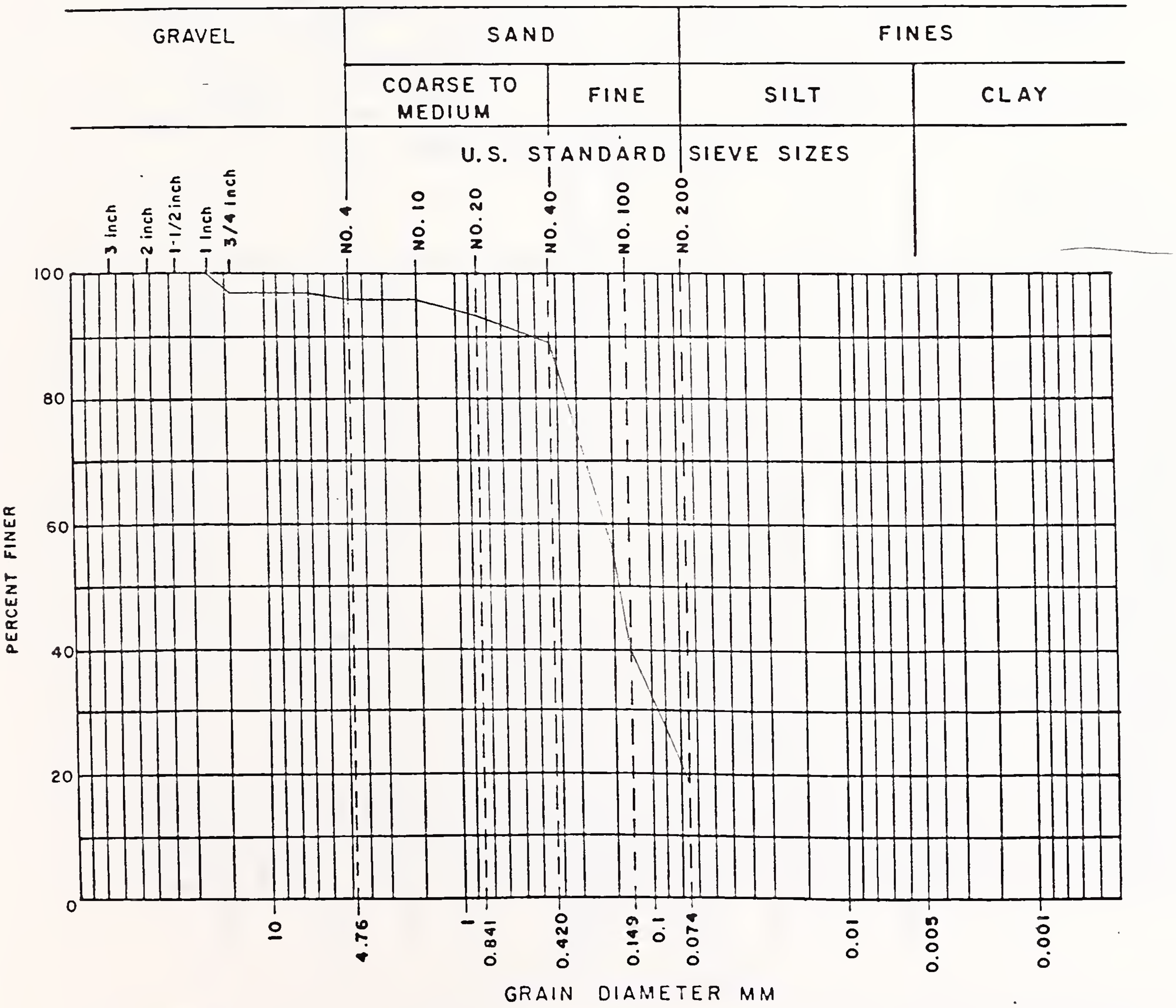
HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

FORM ST-5-80

3019

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3019
PROJECT NAME Middle Creek Dam FIELD NO. DH-7 (75.0-76.5')
DATE SAMPLED 10-6-83 DATE TESTED 10-24-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Drive SAMPLE LOCATION Natural Sand Deposit under Upstream Face
SOIL DESCRIPTION CLAYEY SAND



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 4 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 77 %
3)_ SILT & CLAY, Passing no. 200 sieve 19 %

Fine Sand = 70% Passing no. 40 and retained on no. 200

Plate No. 7

FORM ST-5-80
3020

HKM ASSOCIATES
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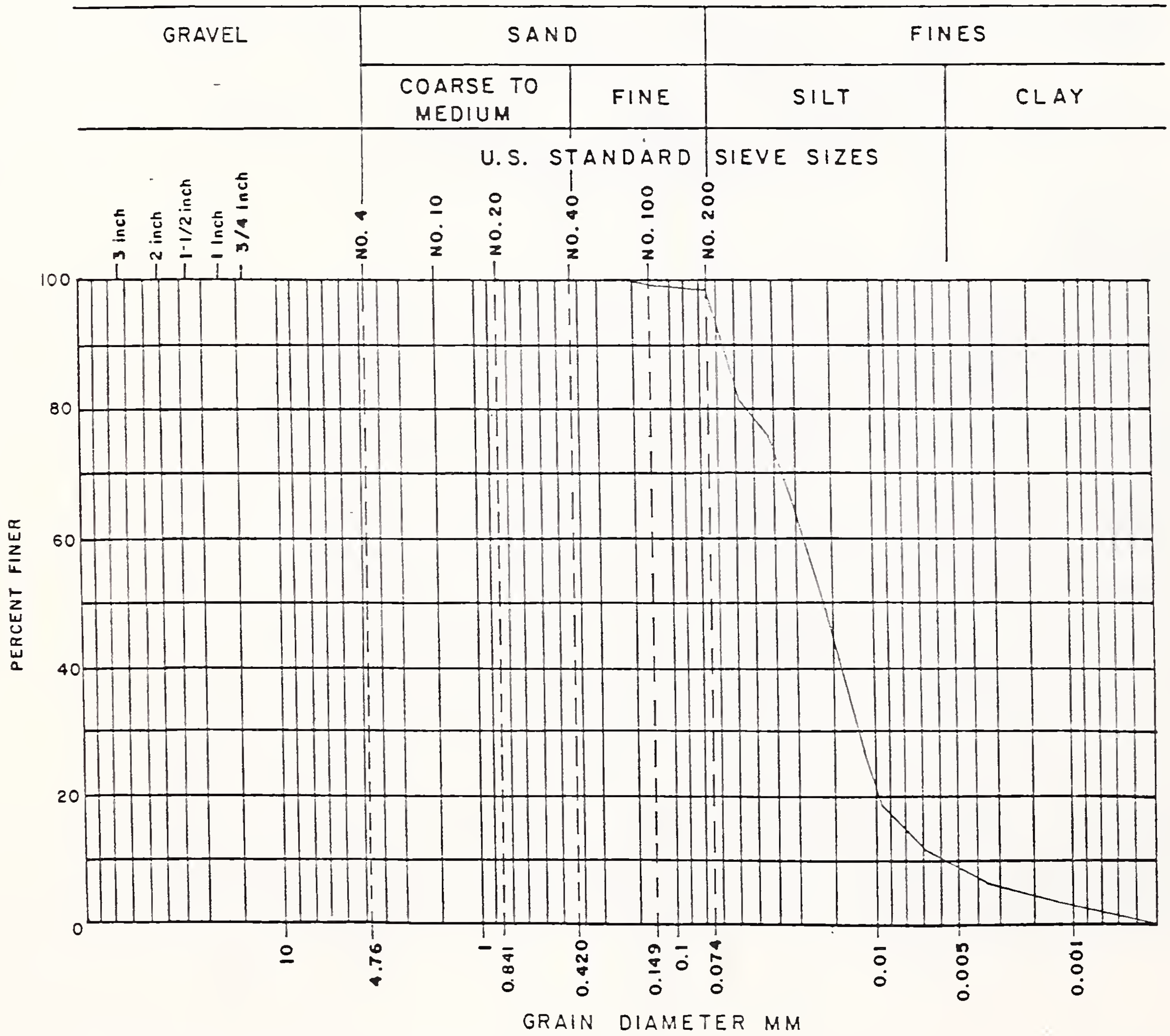
GRAIN SIZE DISTRIBUTION

PROJECT NO.	8M087.113	LAB NO.	3020
PROJECT NAME	Middle Creek Dam	FIELD NO.	DH-7 (82.0-83.5')
DATE SAMPLED	10-7-83	DATE TESTED	10-25-83
SAMPLED BY	DLD	TESTED BY	JP
TYPE SAMPLE	Drive	SAMPLE LOCATION	Foundation Soil Under Upstream Slope
SOIL DESCRIPTION	CLAYEY SILT		



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve ______ 0 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. ______ 2 %
3)_ SILT & CLAY, Passing no. 200 sieve ______ 98 %

Granular Non-plastic

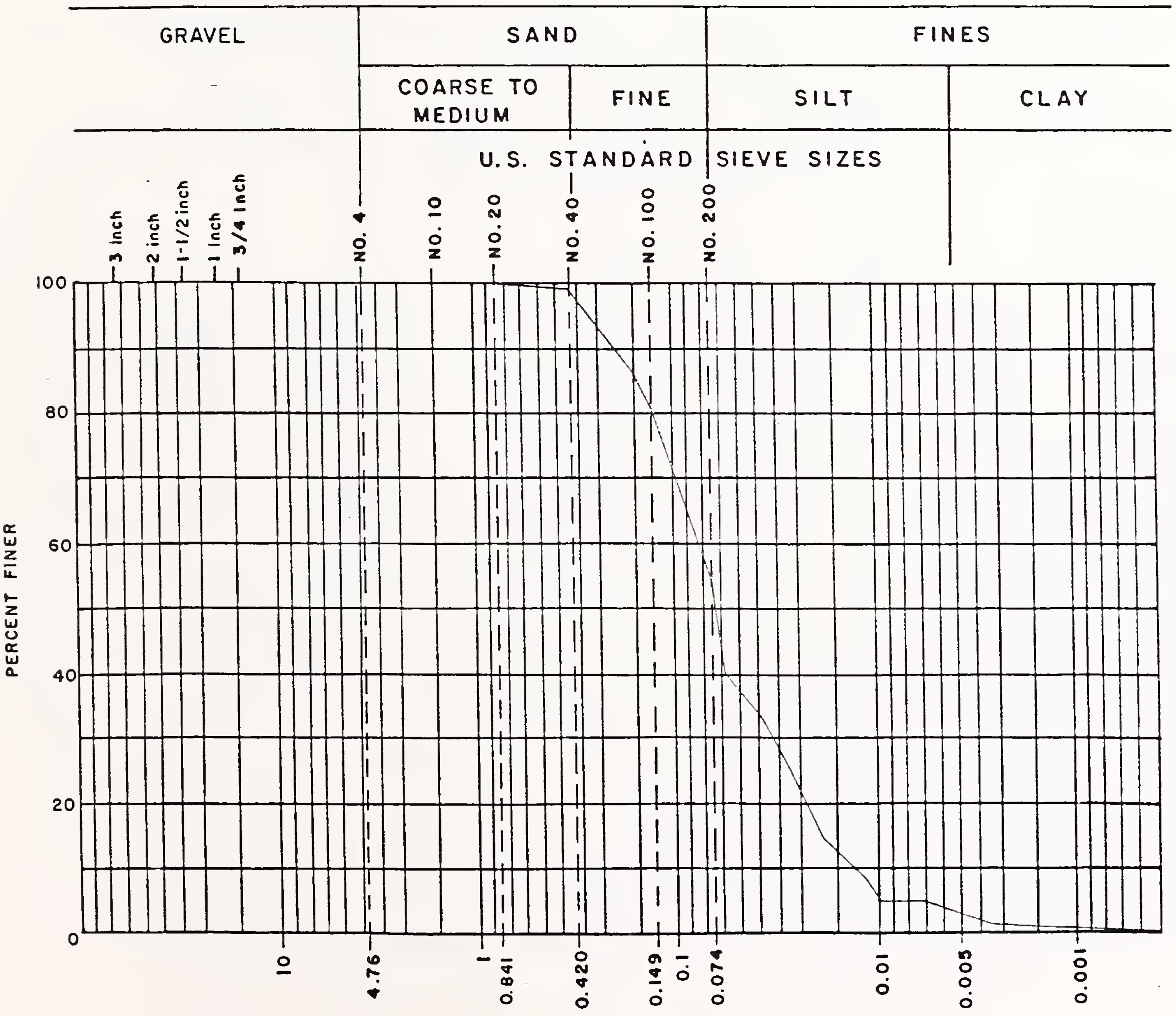
Plate No. 8

FORM ST-5-80
3049

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3049
PROJECT NAME Middle Creek Dam FIELD NO. DH-13 (57.0-67.0')
DATE SAMPLED 10-12-83 DATE TESTED 10-21-83
SAMPLED BY DLD TESTED BY JP
TYPE SAMPLE Sack (cuttings) SAMPLE LOCATION Right Abutment
SOIL DESCRIPTION SANDY SILT Foundation Area



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 0 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 46 %
3)_ SILT & CLAY, Passing no. 200 sieve 54 %

FORM ST-5-80

2926

HKM ASSOCIATES

ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 | LAB NO. 2926

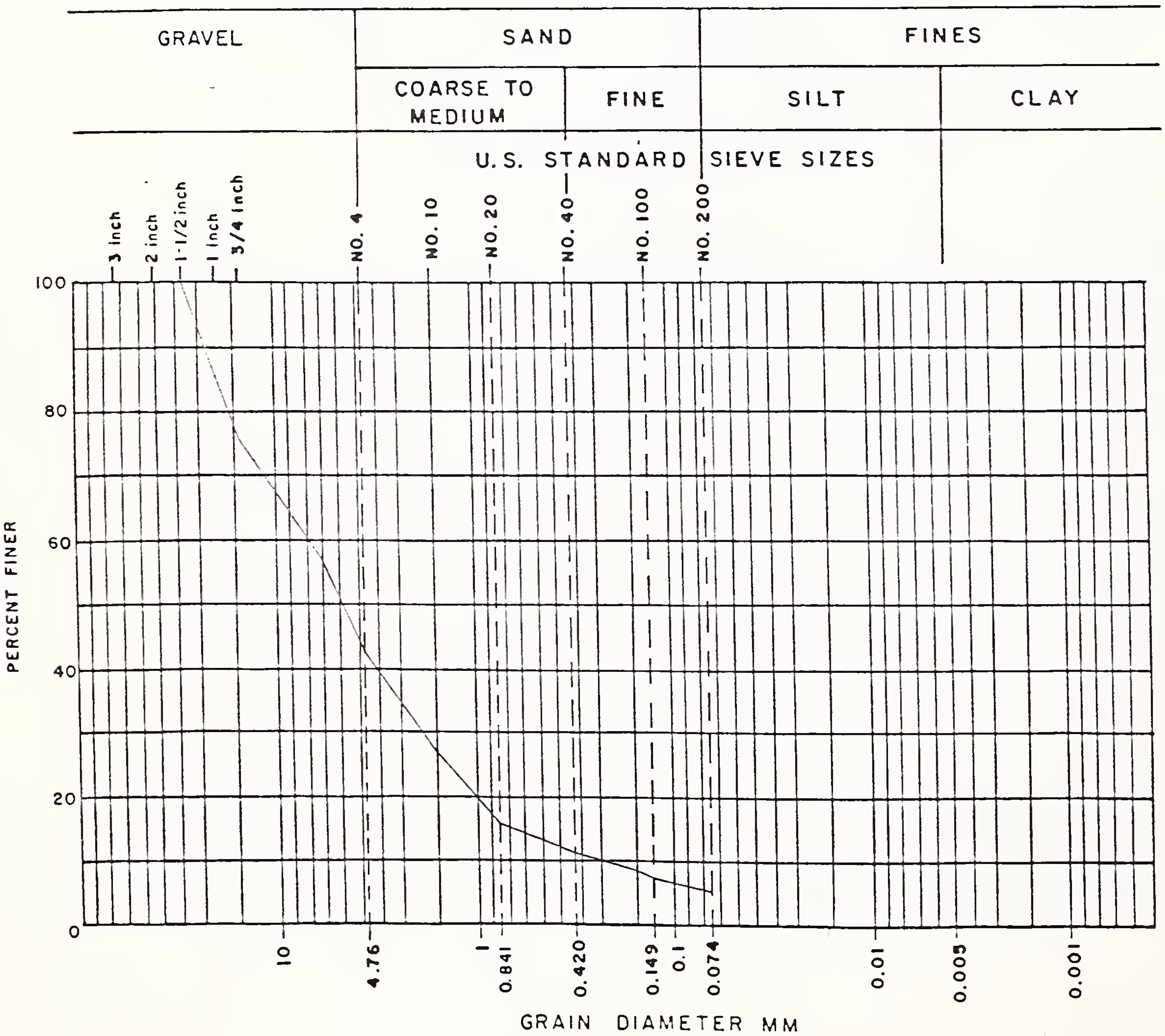
PROJECT NAME Middle Creek Dam | FIELD NO. DH-8 (24.0-25.5')

DATE SAMPLED 7-26-83 | DATE TESTED October 1983

SAMPLED BY DLD | TESTED BY CS

TYPE SAMPLE Drive | SAMPLE LOCATION Left Abutment

SOIL DESCRIPTION SANDY GRAVEL TILL



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve ___ 57 ___ %

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. ___ 38 ___ %

3)_ SILT & CLAY, Passing no. 200 sieve ___ 5 ___ %

Plate No. 10

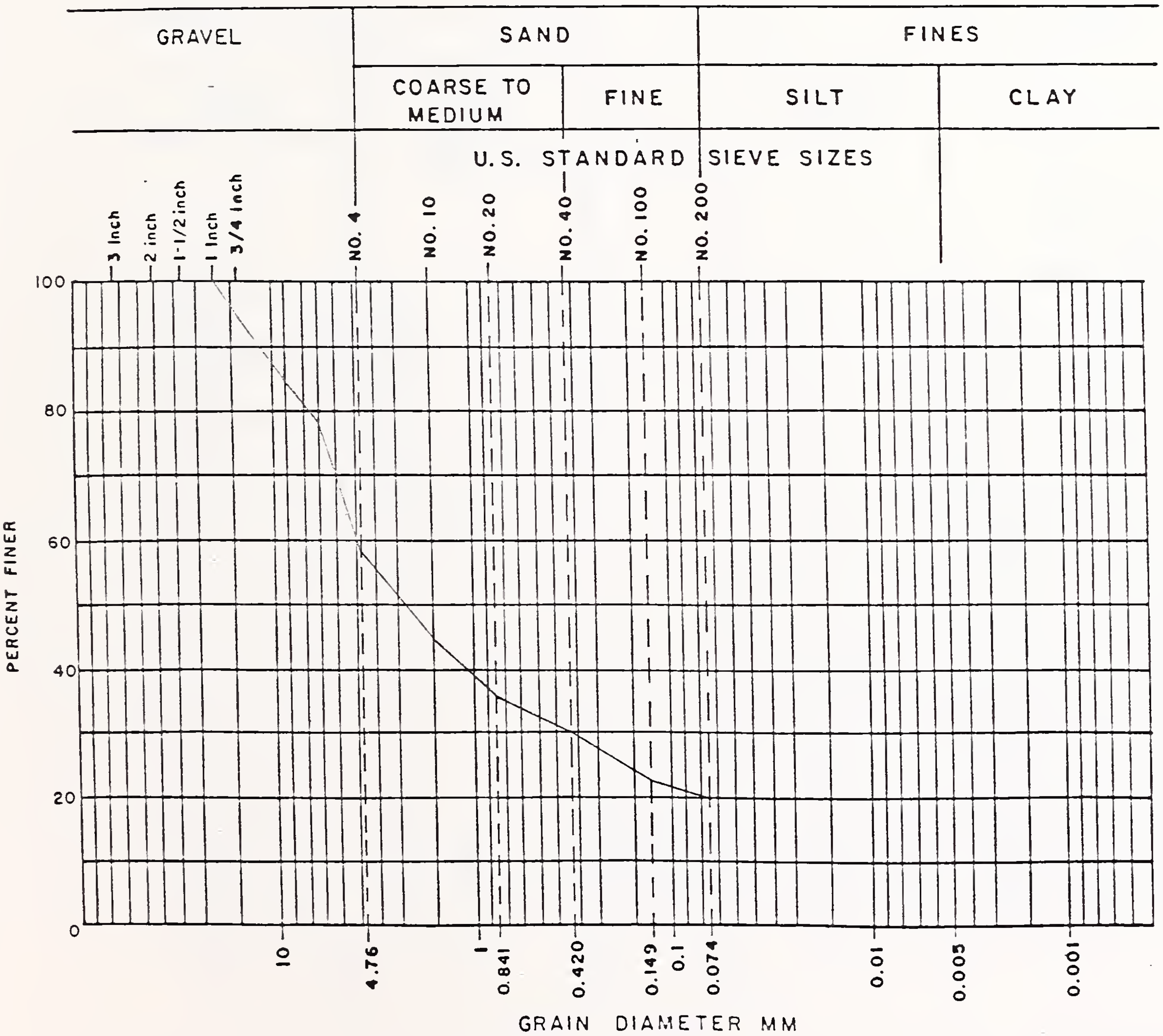
FORM ST-5-80

2944

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO.	8M087.113	LAB NO.	2944
PROJECT NAME	Middle Creek Dam	FIELD NO.	DH-10 (10.0-11.5')
DATE SAMPLED	7-22-83	DATE TESTED	10-8-83
SAMPLED BY	DLD	TESTED BY	CS
TYPE SAMPLE	Drive	SAMPLE LOCATION	Left Abutment
SOIL DESCRIPTION	CLAYEY GRAVEL FILL		



1)- GRAVEL, Passing 3" & retained on no. 4 sieve	41 %
2)- SAND, Passing no. 4 sieve & retained on no. 200 sieve.	39 %
3)- SILT & CLAY, Passing no. 200 sieve	20 %

Plate No. 11

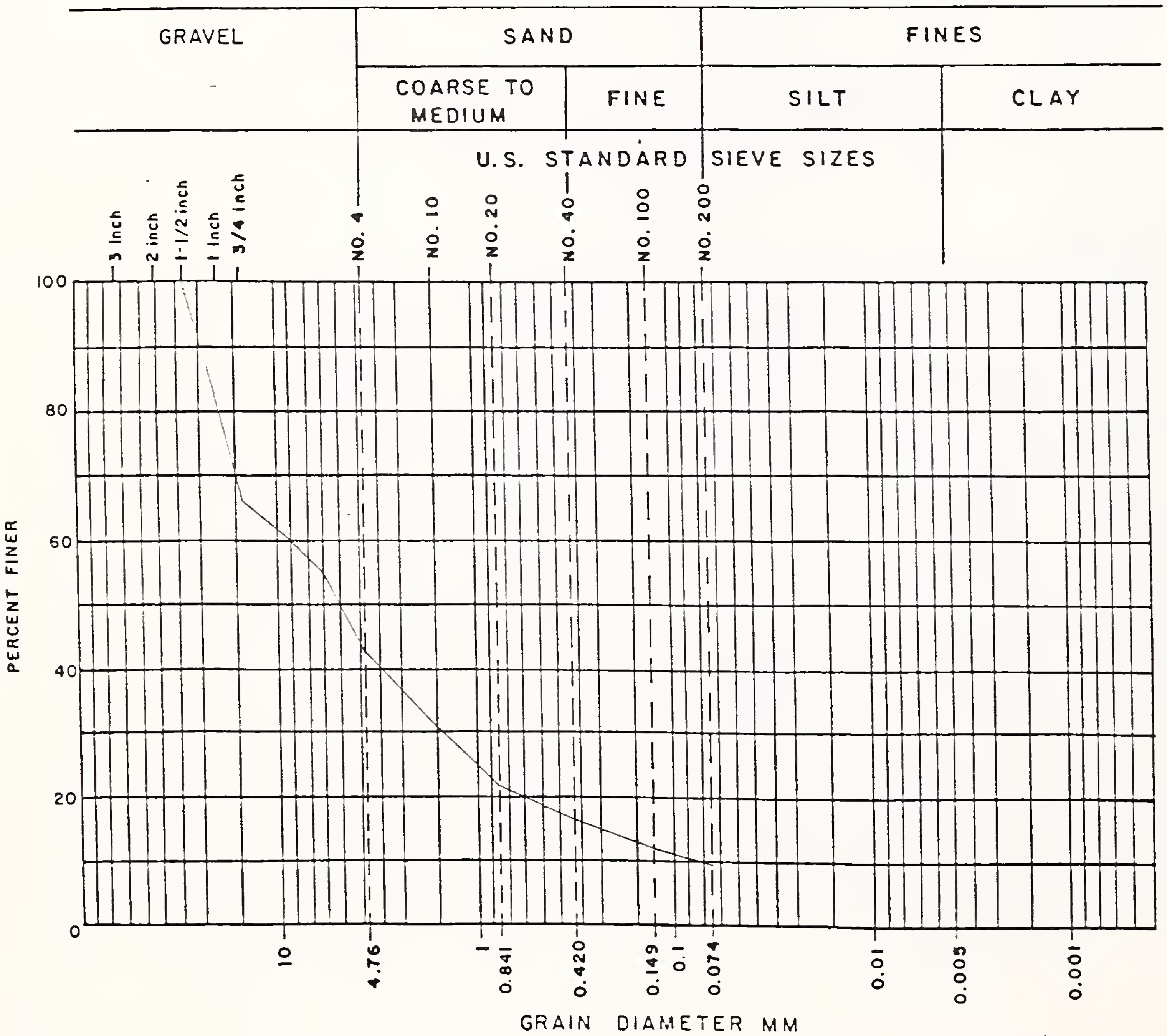
FORM ST-5-80

3039

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3039
PROJECT NAME Middle Creek Dam FIELD NO. DH-12 (8.0-9.5')
DATE SAMPLED 10-12-83 DATE TESTED 11-2-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Drive SAMPLE LOCATION Potential Auxiliary
SOIL DESCRIPTION SANDY GRAVEL TILL Spillway Area



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 57 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 33 %
3)_ SILT & CLAY, Passing no. 200 sieve 10 %

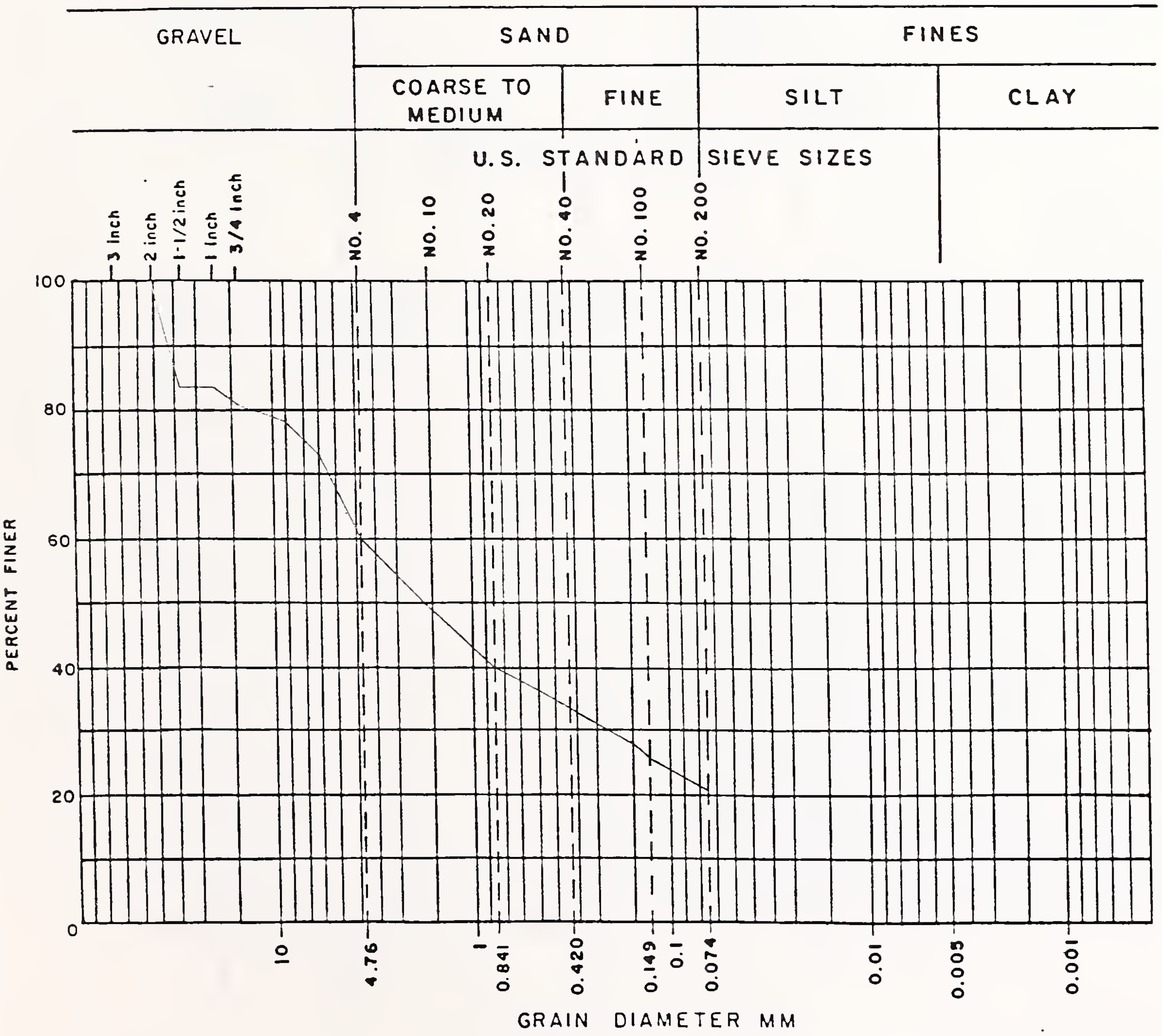
FORM ST-5-80

3044

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO.	8M087.113	LAB NO.	3044
PROJECT NAME	Middle Creek Dam	FIELD NO.	DH-13 (27.0-29.0')
DATE SAMPLED	10-10-83	DATE TESTED	11-2-83
SAMPLED BY	DLD	TESTED BY	CS
TYPE SAMPLE	Drive	SAMPLE LOCATION	Upstream Face- Impervious Zone
SOIL DESCRIPTION	CLAYEY GRAVEL FILL		



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve — 40 %

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. — 39 %

3)_ SILT & CLAY, Passing no. 200 sieve — 21 %

Plate No. 13

FORM ST-5-80

2956 # 57

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 2956

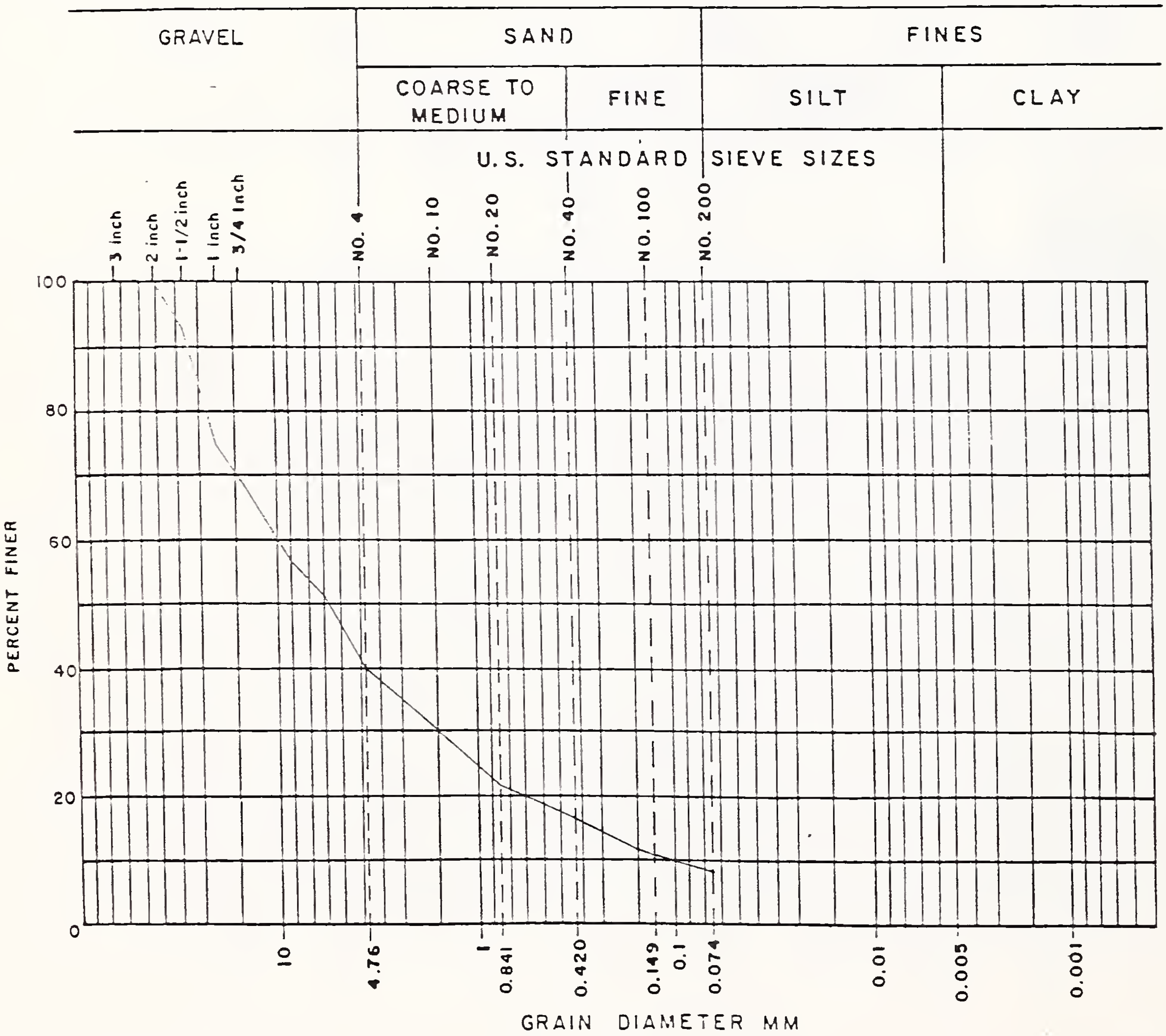
PROJECT NAME Middle Creek Dam FIELD NO. DH-14 (6.0-7.5')

DATE SAMPLED 8-14-83 DATE TESTED 10-13-83

SAMPLED BY DLD TESTED BY CS

TYPE SAMPLE Drive SAMPLE LOCATION Pervious Zone

SOIL DESCRIPTION SANDY GRAVEL FILL Near Right Abutment



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve ---- 60 --%

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. ---- 32 --%

3)_ SILT & CLAY, Passing no. 200 sieve ---- 8 --%

Plate No. 14

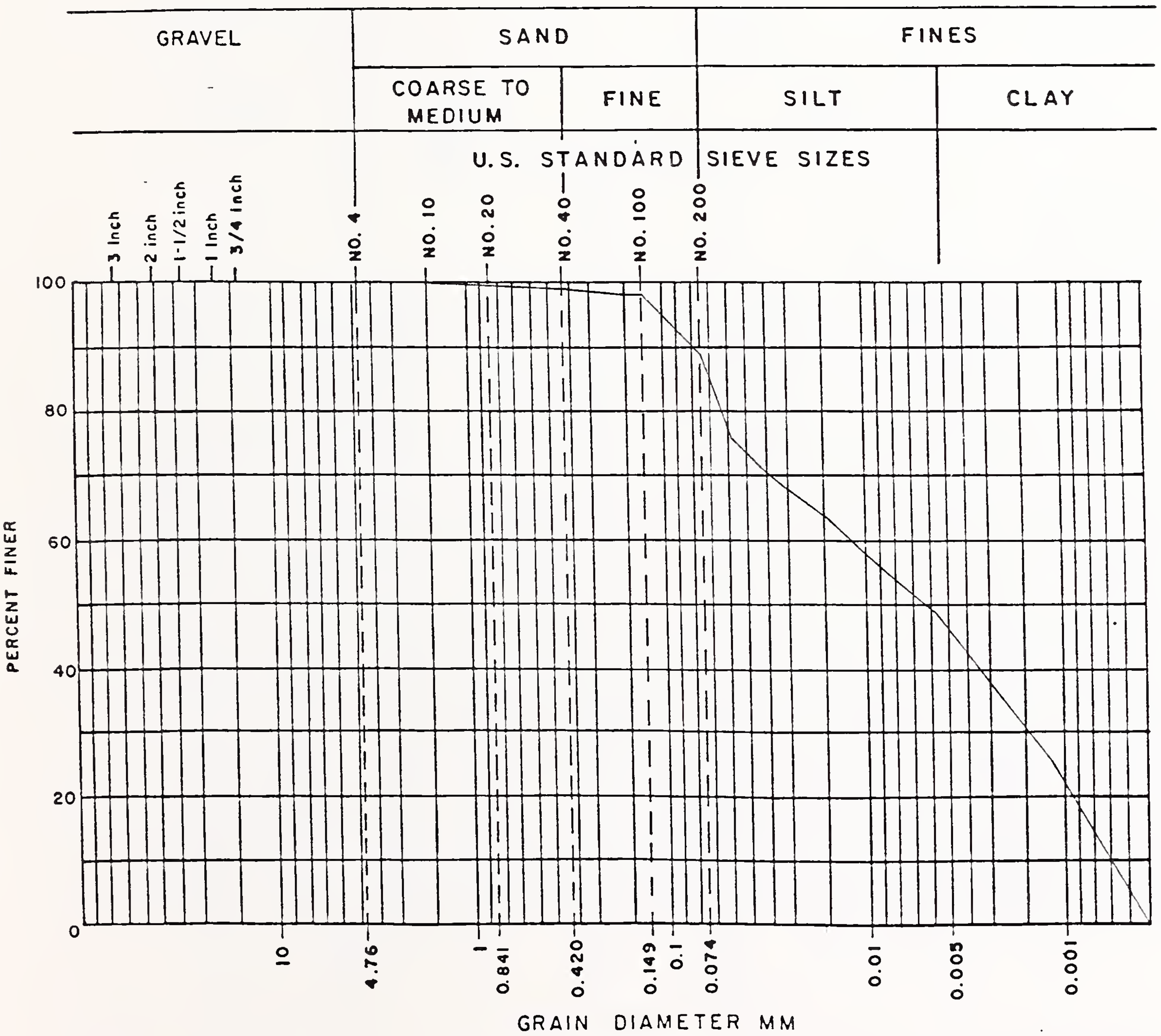
FORM ST-5-80

2960

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 2960
PROJECT NAME Middle Creek Dam FIELD NO. DH-14 (36.0-37.5)
DATE SAMPLED 8-15-83 DATE TESTED 9-23-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Drive SAMPLE LOCATION Right Abutment
SOIL DESCRIPTION SILTY CLAY (CL-2)



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 0 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 11 %
3)_ SILT & CLAY, Passing no. 200 sieve 89 %

Liquid Limit = 46%
Plastic Index = 28

Plate No. 15

FORM ST-5-80

3052

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3052

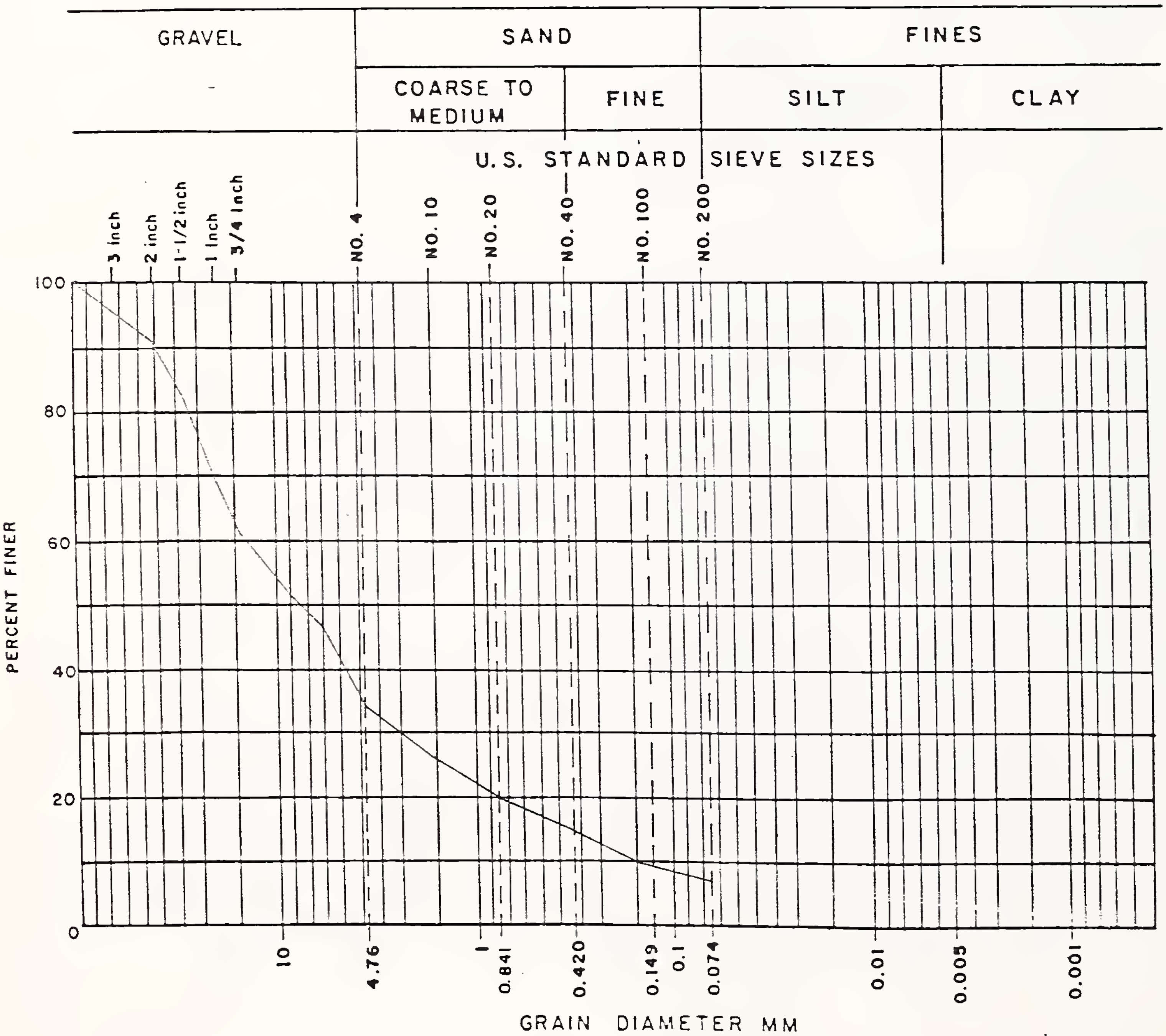
PROJECT NAME Middle Creek Dam FIELD NO. TP-102 (1.0-4.0')

DATE SAMPLED 10-19-83 DATE TESTED 10-24-83

SAMPLED BY DLD TESTED BY CS

TYPE SAMPLE Sack SAMPLE LOCATION Pervious Zone

SOIL DESCRIPTION SANDY GRAVEL FILL Near Right Abutment



1)_ GRAVEL, Passing 4" & retained on no. 4 sieve 65 %

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 28 %

3)_ SILT & CLAY, Passing no. 200 sieve 7 %

Maximum Size = 4"

Plate No. 16

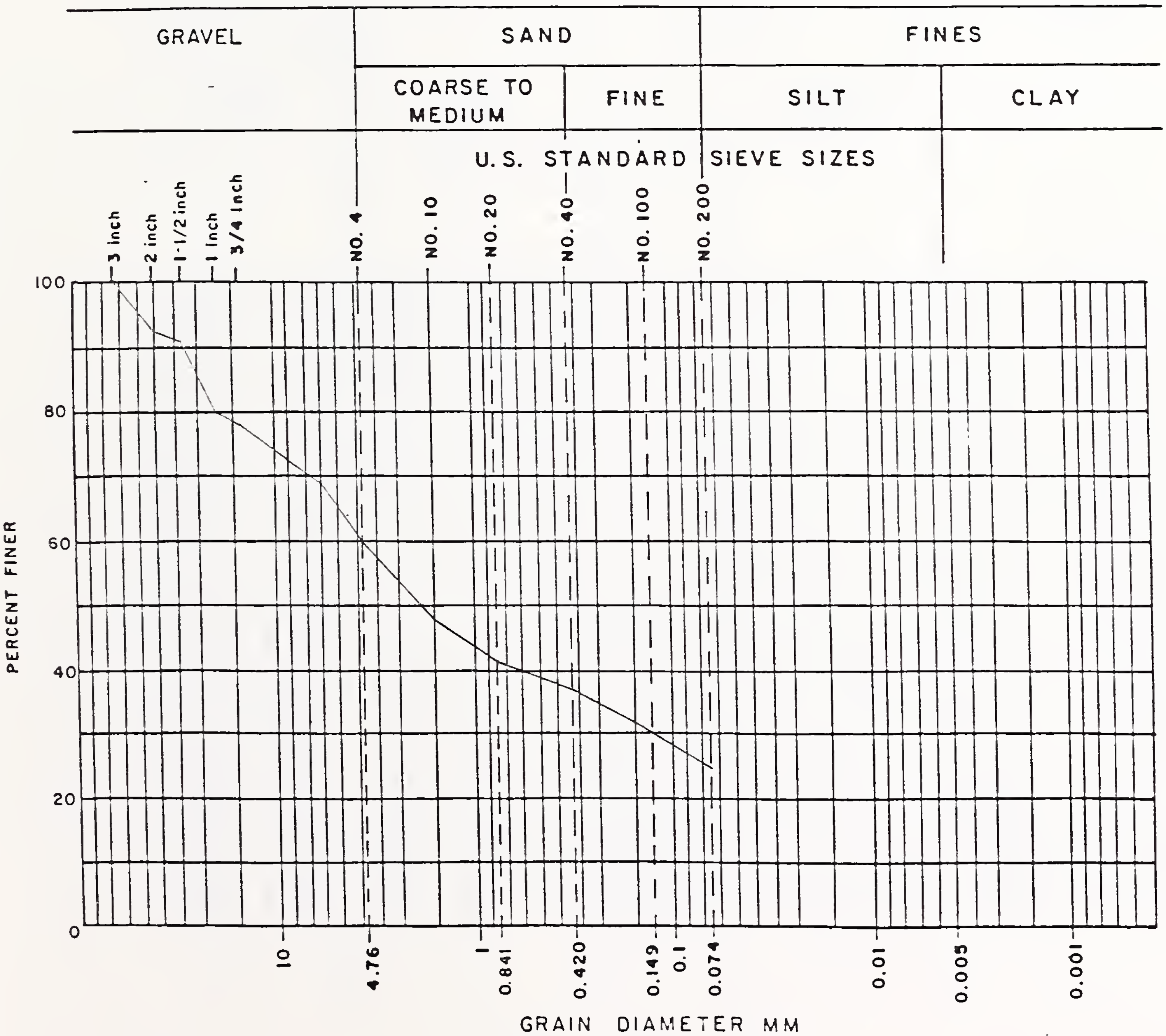
FORM ST-5-80

3053

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3053
PROJECT NAME Middle Creek Dam FIELD NO. TP-103 (4.0-7.0')
DATE SAMPLED 10-12-83 DATE TESTED 10-24-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Sack SAMPLE LOCATION Right Abutment
SOIL DESCRIPTION CLAYEY GRAVEL (GC) On Upstream Face



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 40 %

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 35 %

3)_ SILT & CLAY, Passing no. 200 sieve 25 %

Liquid Limit = 36%
Plastic Index = 17

Plate No. 17

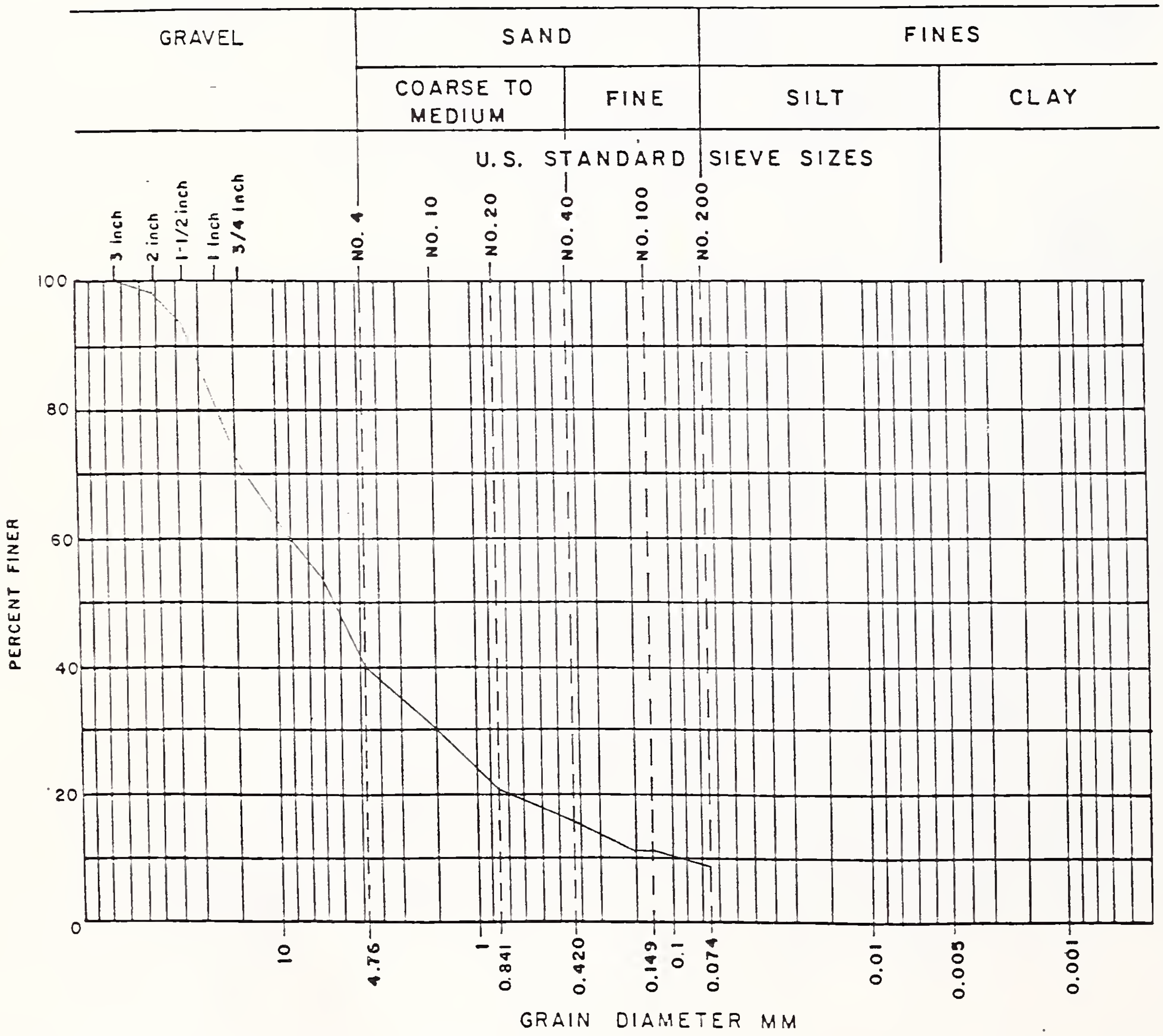
FORM ST-5-80

3026

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3026
PROJECT NAME Middle Creek Dam FIELD NO. TP-104 (3.7 - 10.0')
DATE SAMPLED 10-12-83 DATE TESTED 11-2-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Sack SAMPLE LOCATION Sink Hole Area
SOIL DESCRIPTION SANDY GRAVEL TILL



1)_ GRAVEL, Passing 3" & retained on no. 4 sieve 60 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 31 %
3)_ SILT & CLAY, Passing no. 200 sieve 9 %

Plate No. 18

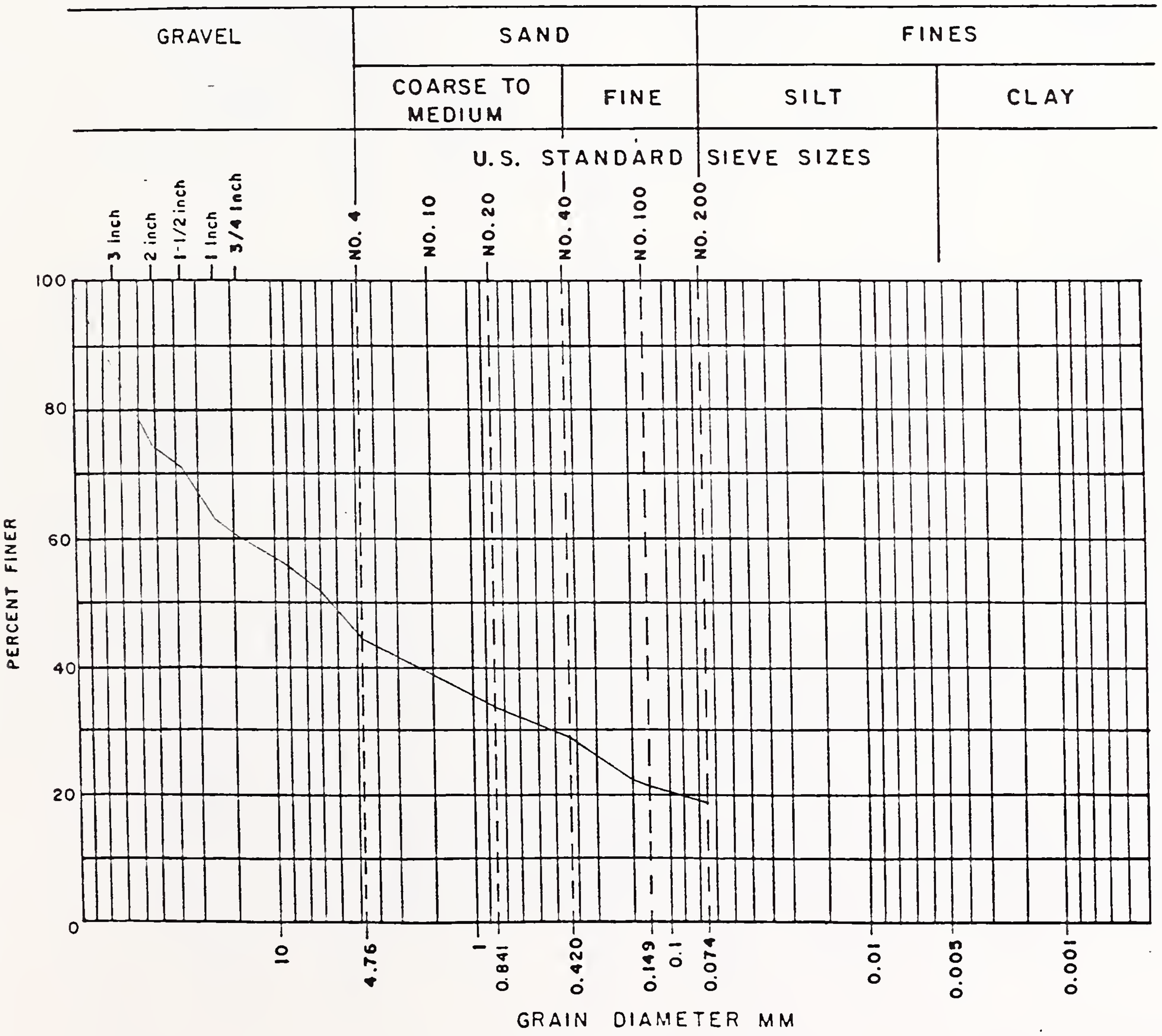
FORM ST-5-80

3054

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 LAB NO. 3054
PROJECT NAME Middle Creek Dam FIELD NO. TP-105 (1.0-8.0')
DATE SAMPLED 10-19-83 DATE TESTED 11-2-83
SAMPLED BY DLD TESTED BY CS
TYPE SAMPLE Sack SAMPLE LOCATION Potential Auxiliary Spillway Area
SOIL DESCRIPTION CLAYEY GRAVEL (GC)



1)_ GRAVEL, Passing 6" & retained on no. 4 sieve 55 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. 26 %
3)_ SILT & CLAY, Passing no. 200 sieve 19 %

Maximum Size = 6"
Liquid Limit = 30%
Plastic Index = 9

Plate No. 19

FORM ST-5-80

3056

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113 | LAB NO. 3056

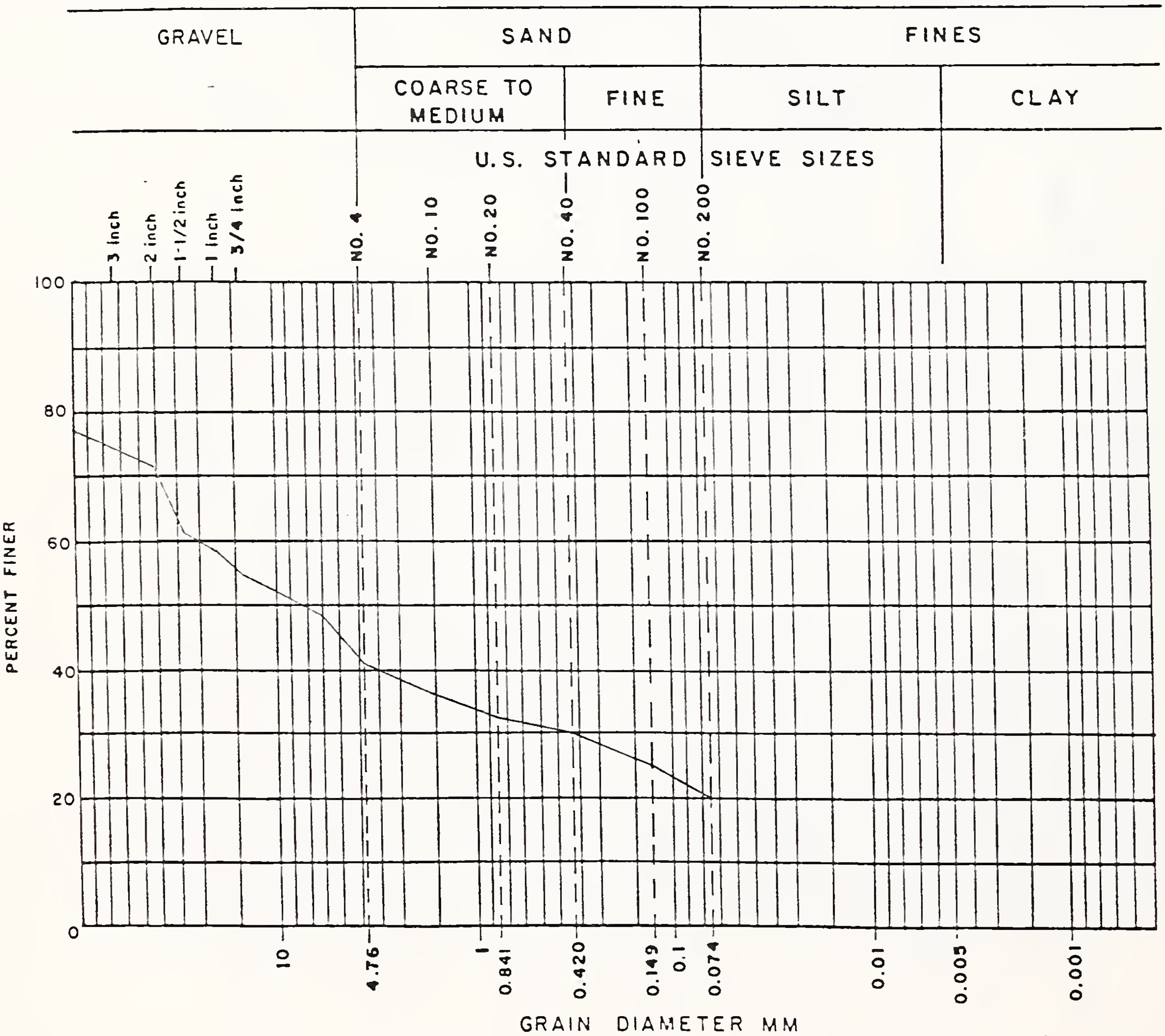
PROJECT NAME Middle Creek Dam | FIELD NO. TP-106 (3.0-8.0')

DATE SAMPLED 10-19-83 | DATE TESTED 10-28-83

SAMPLED BY DLD | TESTED BY JP

TYPE SAMPLE Sack | SAMPLE LOCATION Potential Auxiliary Spillway Area

SOIL DESCRIPTION CLAYEY GRAVEL (GC)



1)_ GRAVEL, Passing 6" & retained on no. 4 sieve — 59 %

2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. — 21 %

3)_ SILT & CLAY, Passing no. 200 sieve — 20 %

Maximum Size = 6"

Plate No. 20

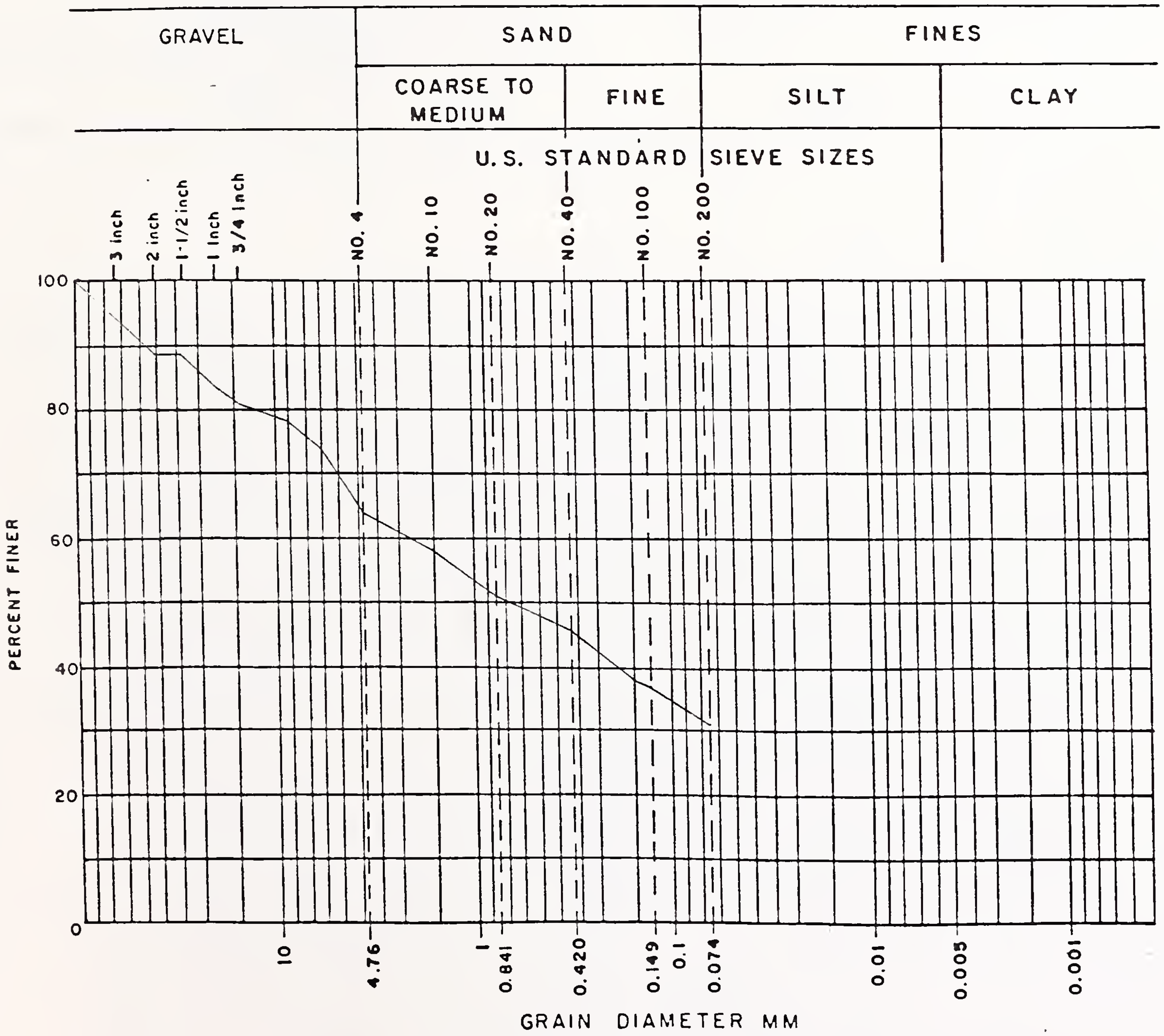
FORM ST-5-80

3058

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113
LAB NO. 3058
PROJECT NAME Middle Creek Dam
FIELD NO. TP-108 (1.4-6.0')
DATE SAMPLED 10-18-83
DATE TESTED 11-2-83
SAMPLED BY DLD
TESTED BY CS
TYPE SAMPLE Sack
SAMPLE LOCATION Borrow Area Southeast of Embankment
SOIL DESCRIPTION CLAYEY GRAVEL (GC)



1)_ GRAVEL, Passing 4" & retained on no. 4 sieve — 36 %
2)_ SAND, Passing no. 4 sieve & retained on no. 200 sieve. — 33 %
3)_ SILT & CLAY, Passing no. 200 sieve — 31 %

Maximum Size = 4"
Liquid Limit = 29%
Plastic Index = 9

Plate No. 21

FORM ST-5-80

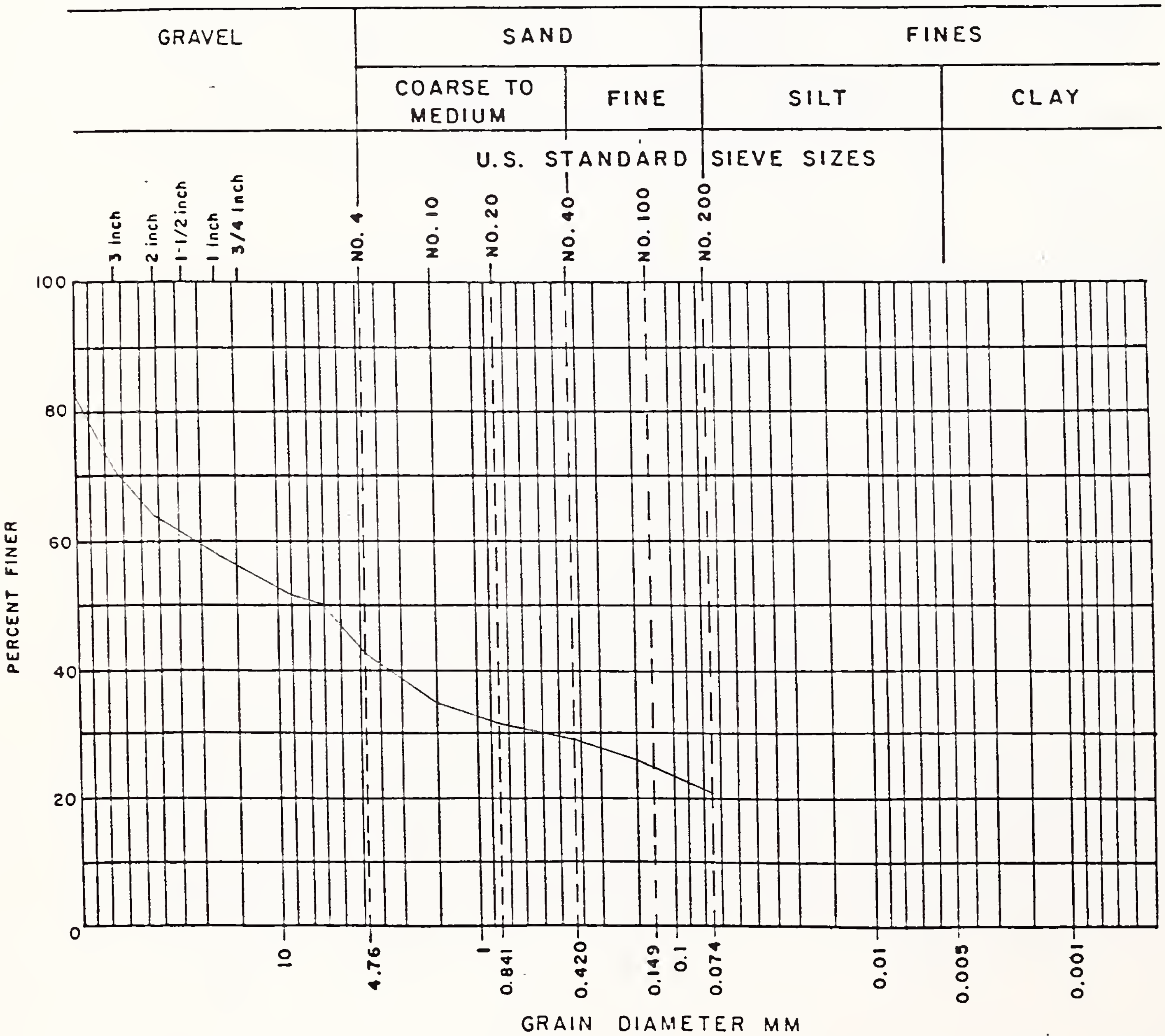
3025

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

GRAIN SIZE DISTRIBUTION

PROJECT NO. 8M087.113
PROJECT NAME Middle Creek Dam
DATE SAMPLED 10-12-83
SAMPLED BY DLD
TYPE SAMPLE Sack
SOIL DESCRIPTION CLAYEY GRAVEL (GC)

LAB NO. 3025
FIELD NO. TP-112 (1.2 - 4.0')
DATE TESTED 10-25-83
TESTED BY JP
SAMPLE LOCATION Borrow Area



1)- GRAVEL, Passing 6" & retained on no. 4 sieve 58 %
2)- SAND, Passing no. 4 sieve & retained on no. 200 sieve. 21 %
3)- SILT & CLAY, Passing no. 200 sieve 21 %

Maximum Size = 6"
Liquid Limit = 41%
Plastic Index = 16

Plate No. 22

IKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

MOISTURE DENSITY RELATIONS OF SOILS

3051

Project No.	8M087.113	Lab No.	3051 TP-101 (2.0 - 8.0')
Project Name	Middle Creek Dam	Test Designation	ASTM 0698
Date Sampled	10-18-83	Date Tested	10-31-83
Soil Class.	GRAVELLY CLAY TILL	% Passing # 4 Sieve	-
Maximum Dry Density	114.7 pcf		
Optimum Moisture	15.1%		

MOISTURE-DENSITY GRAPH

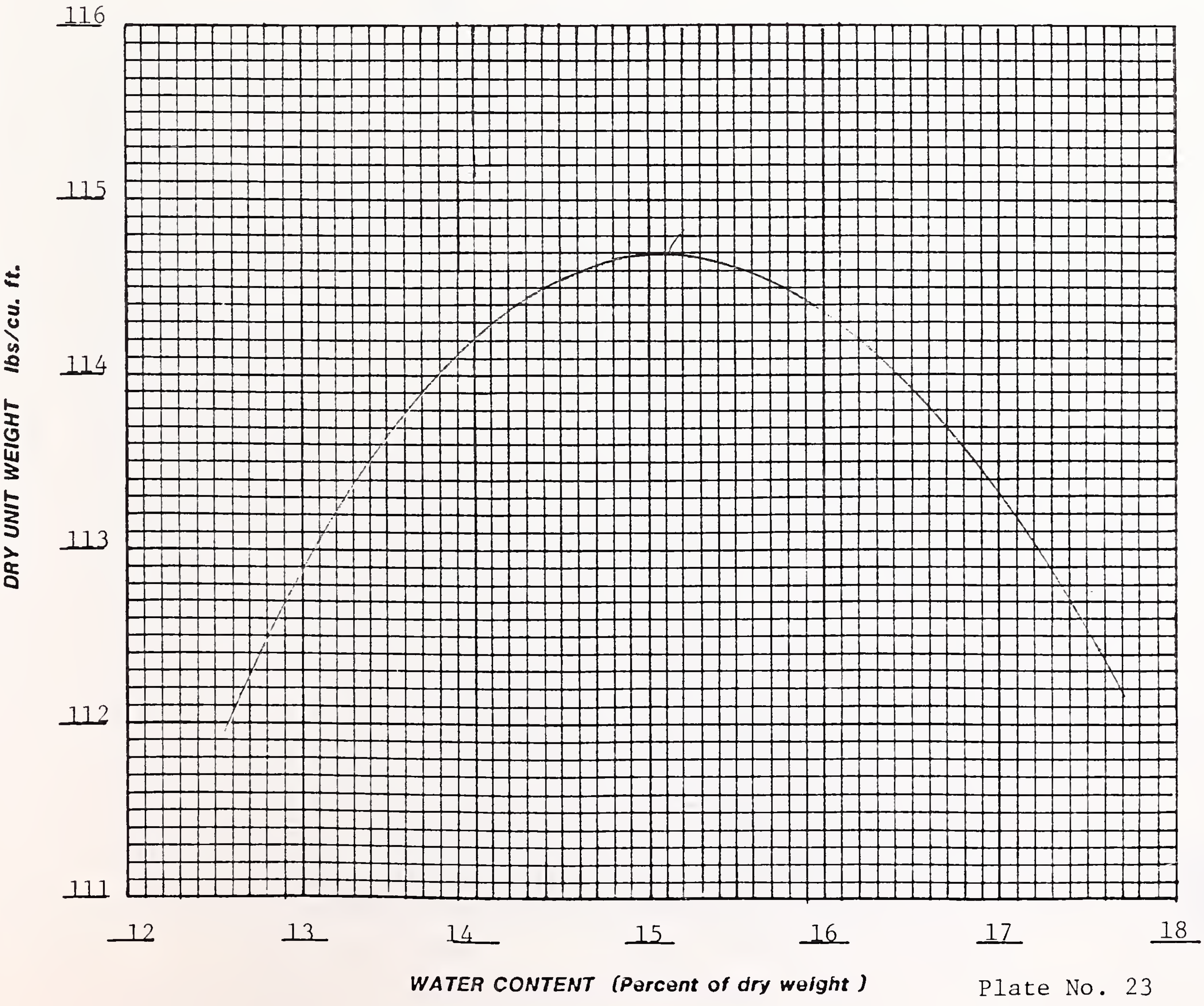


Plate No. 23

HKM ASSOCIATES
ENGINEERS · ARCHITECTS · PLANNERS

MOISTURE DENSITY RELATIONS OF SOILS

3056

Project No. 8M087.113

Project Name Middle Creek Dam

Date Sampled 10-19-83

Soil Class. CLAYEY GRAVEL (GC)

Maximum Dry Density 125.6 pcf

Optimum Moisture 10.3%

Lab No. 3056 TP-106 (3.0-8.0')

Test Designation ASTM D698

Date Tested 11-1-83

% Passing # 4Sieve 41

MOISTURE-DENSITY GRAPH

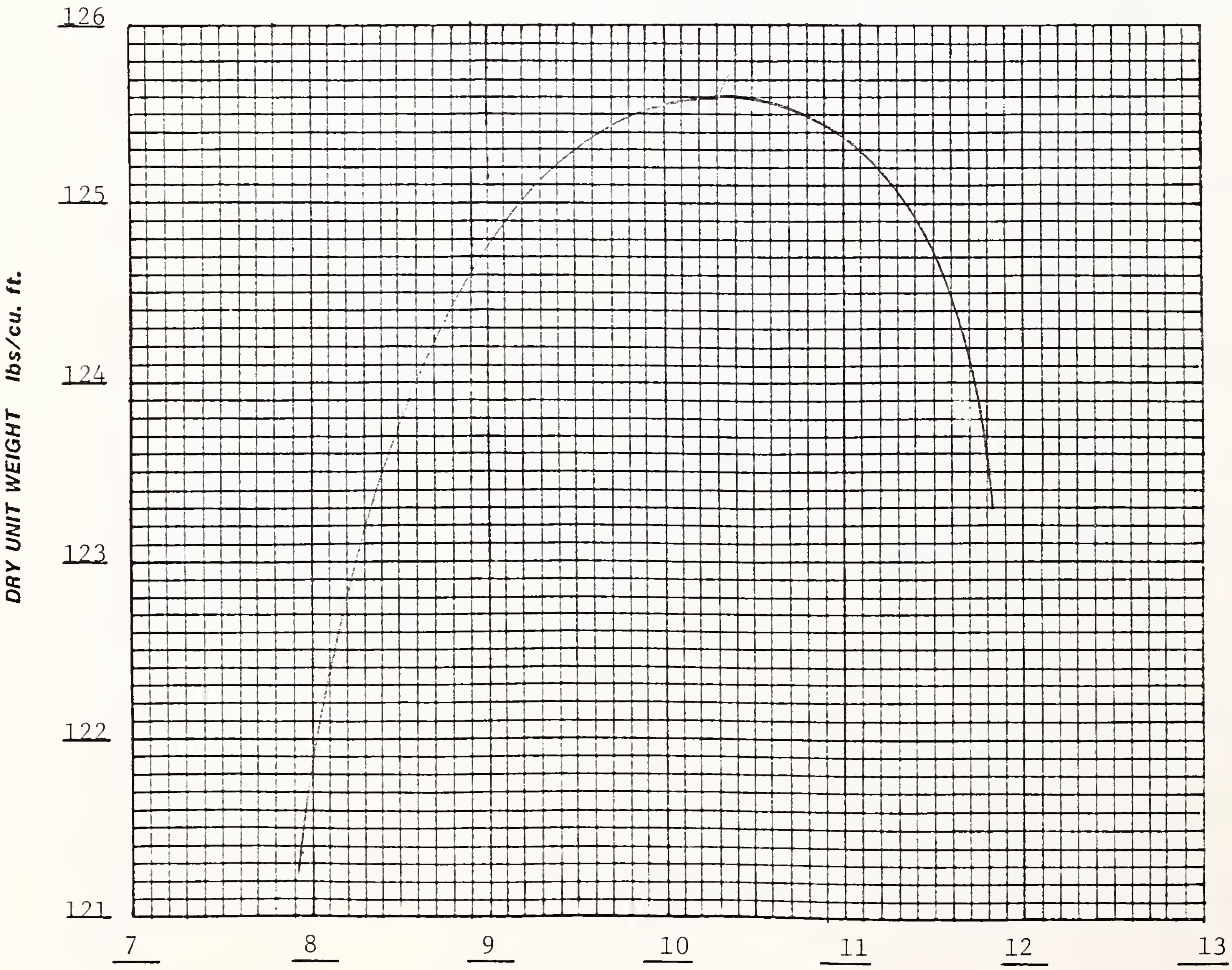
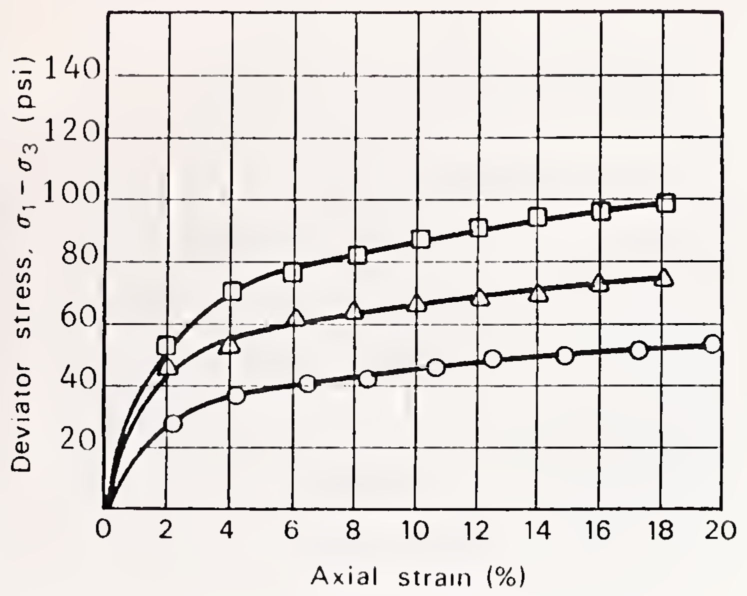
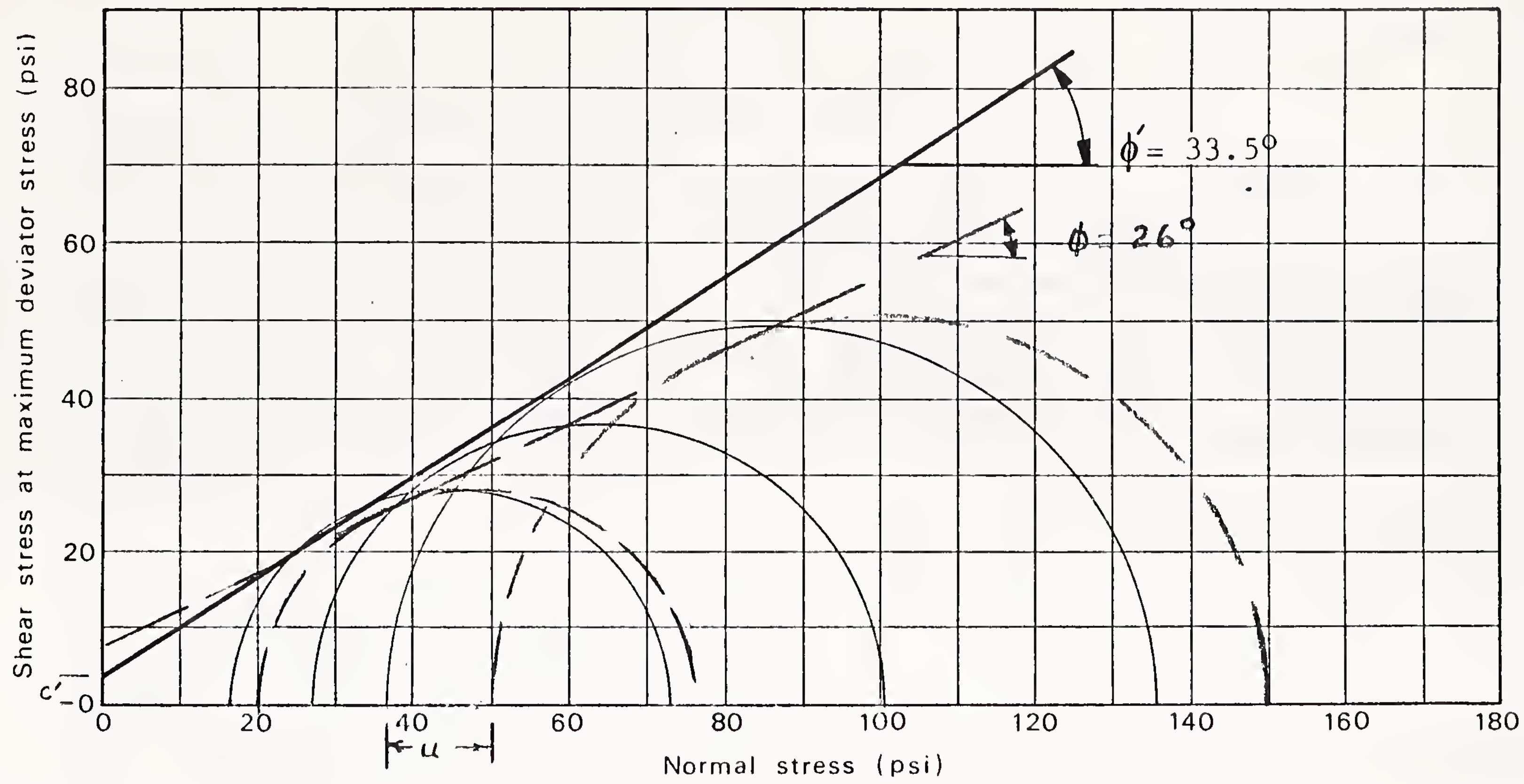
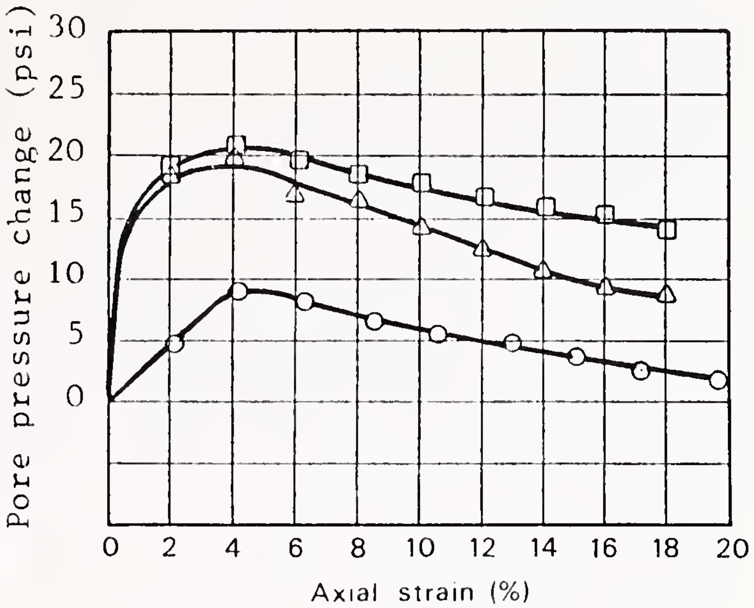


Plate No. 24







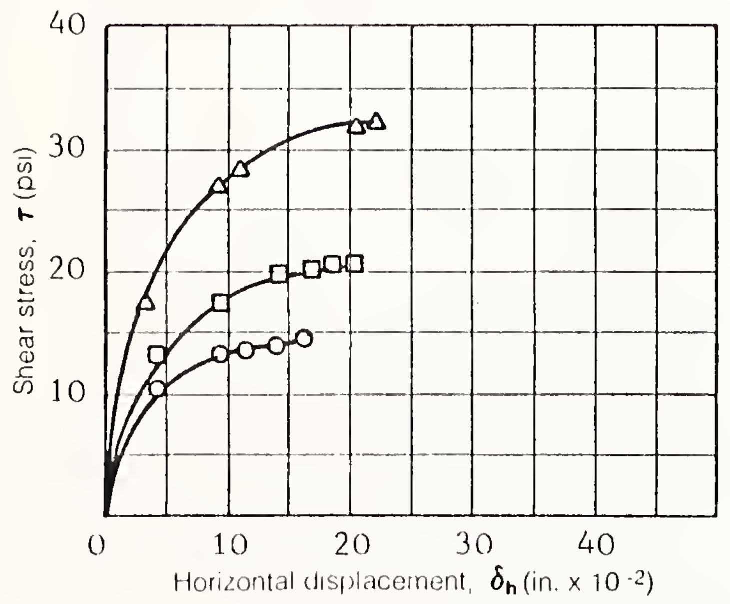
Test no. or symbol	Boring no. or depth	Sample data		Degree of saturation (%)	Confining pressure (psi)	Maximum deviator stress (psi)	Strength values at failure		Sample size, L/D (inches)	Strain rate (inches/minute)
		Dry density (pcf)	Moisture content (%)				Friction angle ϕ (degrees)	Cohesion (c/psi)		
○	6A	109.7	16.4	100	20	55.1	33.5	3.5	2.8/1.32	.0023
△	64–65'	109.7	16.4	100	35	73.5	33.5	3.5	5/2.5	.0025
□	20–21'	109.7	16.4	100	50	100.2	33.5	3.5	5/2.5	.0025

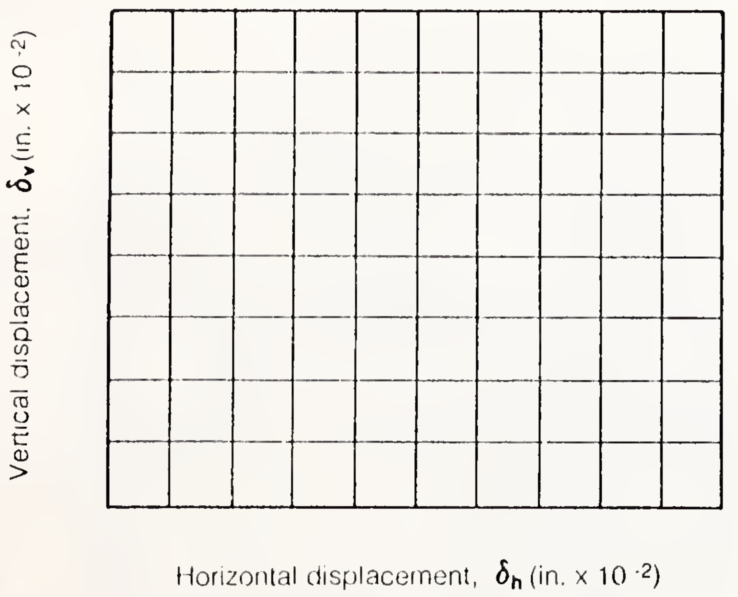
ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

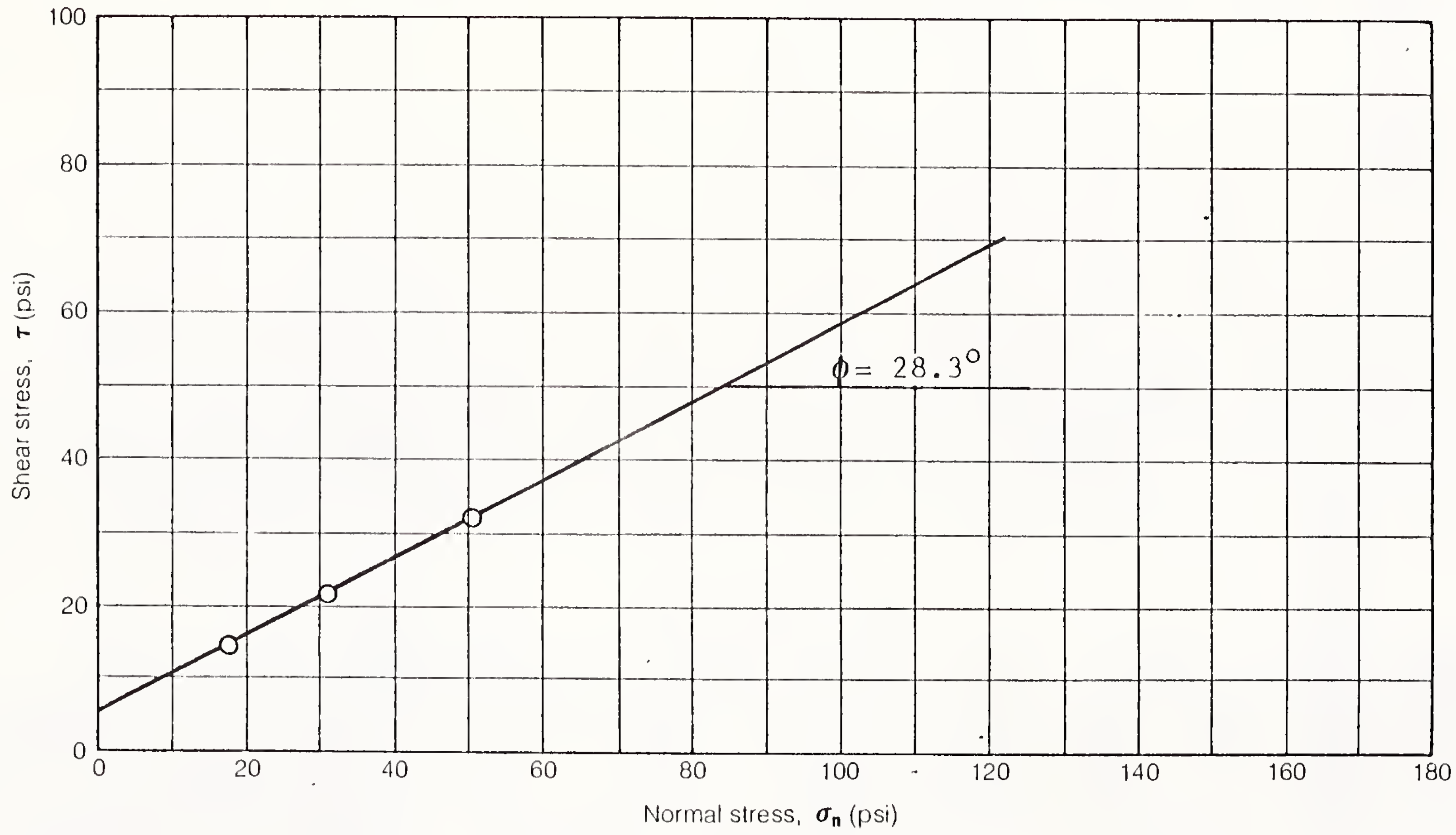
TRIAXIAL SHEAR TEST
Project: Middle Creek Dam

HOLE NO. 6,6A & 7
DEPTH: Impervious zone

Plate NO. 25







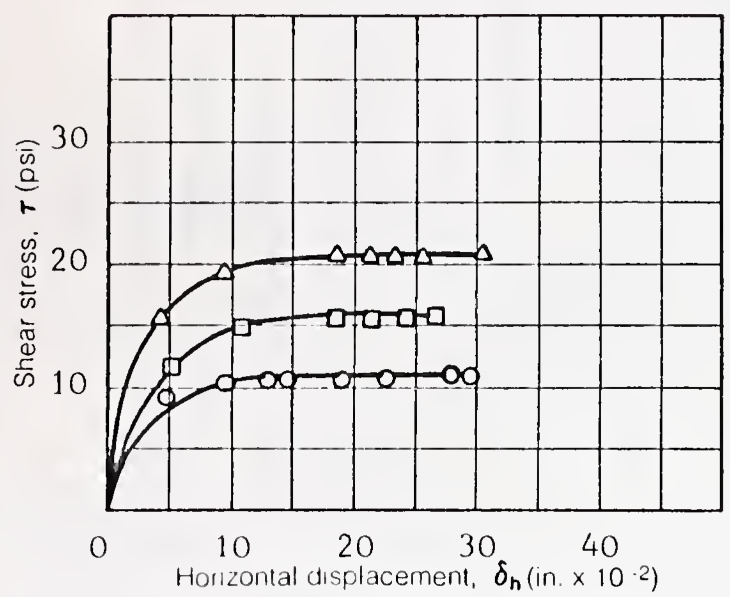
Test no. or symbol	Sample size (inches)	Sample data		Degree of saturation (%)	Normal stress σ_n (psi)	Maximum shear stress τ (psi)	Strain rate (inches / minute)	Shear strength parameters	
		Dry density (pcf)	Moisture content (%)					Friction angle ϕ (degrees)	Cohesion (c / psi)
○	2	86.8	32.8	100	17.8	14.9	.0071	28.3	5.0
□	2	86.8	32.8	100	30.6	20.9	.0071		
△	2	86.8	32.8	100	49.6	32.7	.0071		

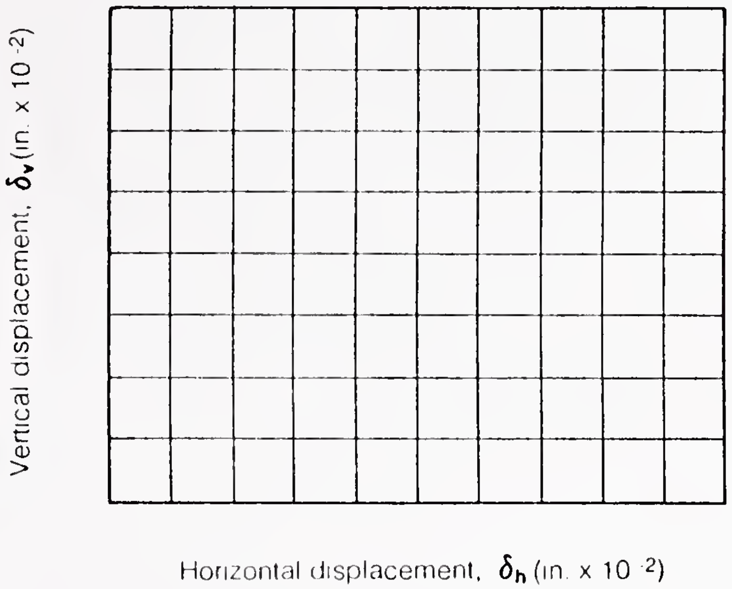
ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

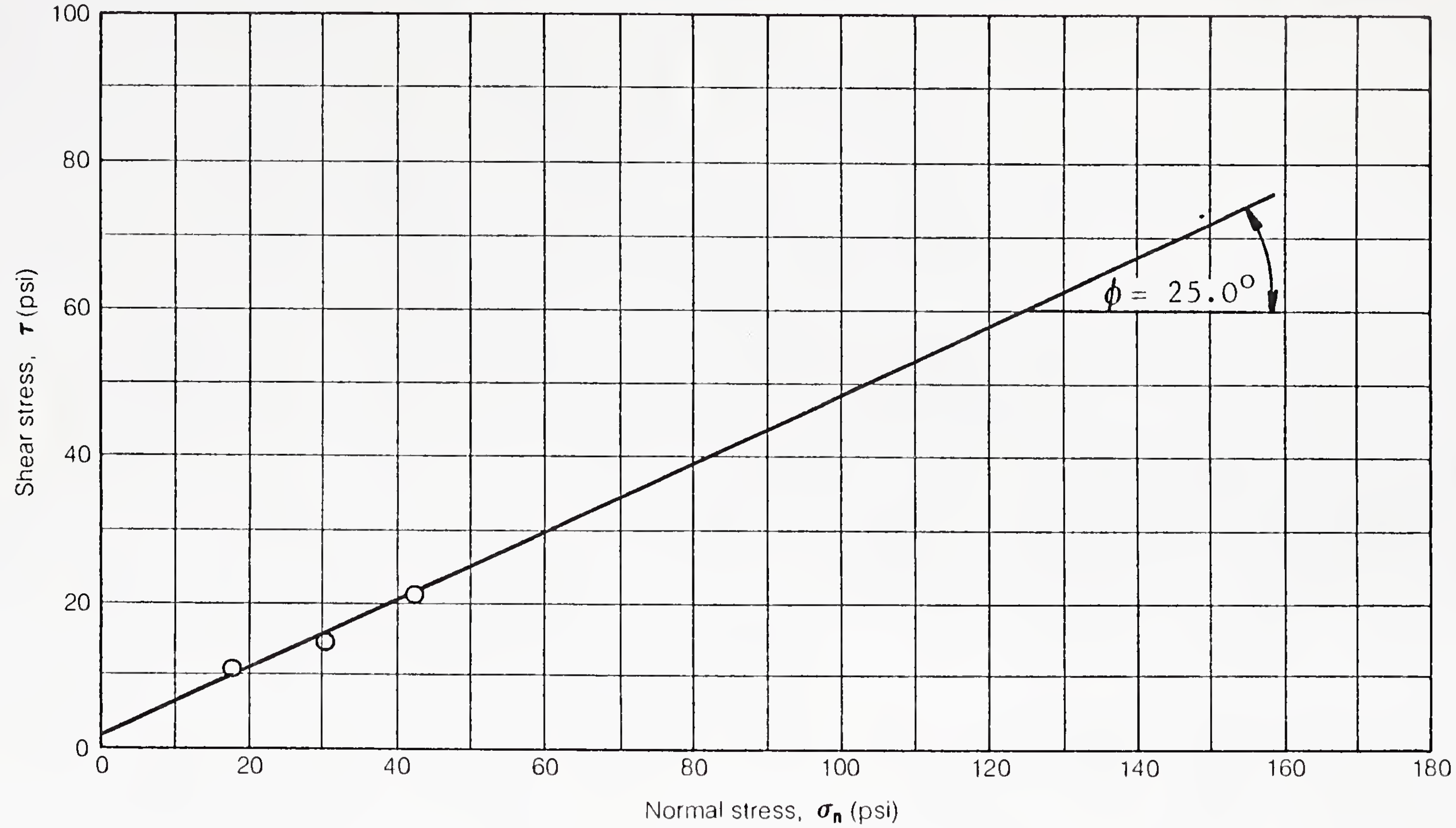
DIRECT SHEAR TEST
Project: Middle Creek Dam

HOLE NO. 7
DEPTH: 87-88' (Foundation)

Plate NO. 26







Test no. or symbol	Sample size (inches)	Sample data		Degree of saturation (%)	Normal stress σ_n (psi)	Maximum shear stress τ (psi)	Strain rate (inches / minute)	Shear strength parameters	
		Dry density (pcf)	Moisture content (%)					Friction angle ϕ (degrees)	Cohesion (c / psi)
○	2	110.3	18.3	100	17.8	11.5	.0009	25.0	2.0
□	2	110.3	18.3	100	30.2	15.4	.0009		
△	2	110.3	18.3	100	42.4	21.0	.0009		

ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

DIRECT SHEAR TEST
Project: Middle Creek Dam

HOLE NO. 14
DEPTH. 32.5-33.9'
Right Abutment

Plate NO. 27

APPENDIX B

Contents	Exhibit
Computer Printouts	1-5
USBR Seismic Reevaluation of Embankment Dams Criteria	6
California's Seismic Reevaluation of Dams Criteria	7
Inclinometer Data Sheet	8

EXHIBIT 1

PROGRAM SLOPE - SLOPE STABILITY DATE: 2/ 9/84

MIDDLE CREEK DAM
FEBRUAY 9 1984
SUDDEN DRAWDOWN UPSTREAM SLOPE OF EXISTING STRUCTURE
COE PROGRAM WITH SEISMIC COEF, OF 0.0
FAILURE PLANE ELEVATION 620

****

INPUT DATA

MINIMUM ELEVATION OF CIRCLE= 620.0 X-START= 780.0 Y-START= 810.0
SEARCH INCREMENT= 5.0 F.S. MIN.= 0.0

NUMBER OF LINES= 22 MIN. NO. OF SLICES= 8 SEISMIC COEFF.= 0.0

EMBANKMENT AND FOUNDATION PROFILE:
****

XTOEL	YTOEL	XTOPL	YTOPL	XTOER	YTOER	XTOPR	YTOPR
*****	*****	*****	*****	*****	*****	*****	*****
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

	X1(I)	Y1(I)	X2(I)	Y2(I)	TYPE
(1)	-5000.0	606.0	206.0	606.0	5
(2)	206.0	606.0	230.0	620.0	5
(3)	230.0	620.0	313.0	641.0	3
(4)	313.0	641.0	400.0	682.0	3
(5)	400.0	682.0	410.0	682.0	3
(6)	410.0	682.0	490.0	722.0	3
(7)	490.0	722.0	495.0	722.0	3
(8)	495.0	722.0	510.0	722.0	1
(9)	510.0	722.0	644.0	675.0	1
(10)	644.0	675.0	796.0	637.0	1
(11)	796.0	637.0	943.0	632.0	1
(12)	943.0	632.0	5000.0	632.0	4
(13)	230.0	620.0	382.0	608.0	5
(14)	382.0	608.0	495.0	722.0	2
(15)	382.0	608.0	506.0	611.0	5
(16)	495.0	722.0	630.0	614.0	2
(17)	506.0	611.0	630.0	614.0	4
(18)	506.0	611.0	634.0	590.0	5
(19)	630.0	614.0	916.0	597.0	4
(20)	634.0	590.0	5000.0	590.0	5
(21)	916.0	597.0	943.0	632.0	4
(22)	-5000.0	560.0	5000.0	560.0	6

SOIL CONSTANTS;
****
NUMBER OF SOIL TYPES= 6

	TYPE NO.	WT.MOIST	WT. SAT.	C(1)	PHI(1)	C(2)	PHI(2)
(1)	1	120.0	125.0	200.0	26.0	0.0	0.0
(2)	2	125.0	130.0	0.0	30.0	0.0	0.0
(3)	3	125.0	130.0	0.0	34.0	0.0	0.0
(4)	4	100.0	105.0	0.0	28.0	0.0	0.0
(5)	5	130.0	130.0	0.0	34.0	0.0	0.0
(6)	6	135.0	135.0	1000.0	30.0	0.0	0.0

```
PIEZOMETRIC SURFACE DATA:
************************
NUMBER OF POINTS FOR PIEZOMETRIC SURFACE=  9

       XPIEZ(I)     YPIEZ(I)
( 1)    5000.0       716.0
( 2)     524.0       716.0
( 3)     514.0       699.0
( 4)     416.0       665.0
( 5)     369.0       632.0
( 6)     326.0       620.0
( 7)     220.0       611.0
( 8)     206.0       606.0
( 9)   -5000.0       606.0

DRAWDOWN SURFACE DATA:
*********************
NUMBER OF POINTS FOR DRAWDOWN SURFACE=  4
        XSUD(I)     YSUD(I)
( 1)    5000.0       637.0
( 2)     796.0       637.0
( 3)     220.0       611.0
( 4)   -5000.0       606.0
*******************************************************************
```

MIDDLE CREEK DAM
FEBRUAY 9 1984
SUDDEN DRAWDOWN   UPSTREAM SLOPE OF EXISTING STRUCTURE
COE PROGRAM WITH SEISMIC COEF, OF 0.0
FAILURE PLANE ELEVATION 620

CHECK DATA FOR THE BEFORE OR NON-DRAWDOWN CRITICAL ARC
****

FACTOR OF SAFETY= 2.565
LOCATION OF CENTER; X= 706.16 Y= 825.17
ARC RADIUS= 205.17

TABULATION OF SLICE DATA
****

SLICE NO.	SLICE WIDTH	X-COORD. OF SLICE	TOTAL WEIGHT	WATER FORCE	DIR. OF W. FORCE	C-FORCE DEVEL.	DIR. OF C-FORCE
1	32.00	549.99	63.45	31.67	90.00	3.85	-49.57
2	32.00	581.99	139.65	69.71	90.00	3.13	-37.24
3	32.00	614.00	174.65	87.18	90.00	2.79	-26.69
4	4.00	632.00	22.67	11.32	90.00	0.33	-21.19
5	10.00	639.00	56.81	28.36	90.00	0.83	-19.11
6	31.08	659.54	177.74	88.73	90.00	2.49	-13.14
7	31.08	690.62	166.12	82.92	90.00	2.43	-4.34
8	28.23	720.28	125.06	62.43	90.00	2.21	3.94
9	28.23	748.50	86.30	43.08	90.00	2.25	11.91
10	28.23	776.73	32.82	16.38	90.00	2.34	20.12

SLICE NO.	PHI DEVEL.	NORMAL STRESS	NORMAL FORCE	ALPHA TOP	ALPHA BOTTOM	SIDE FORCES E1	SIDE FORCES E2
1	10.77	0.59	29.22	-19.33	-19.33	0.0	17.11
2	10.77	1.60	64.33	-19.33	-19.33	17.11	45.43
3	10.77	2.26	80.87	-19.33	-19.33	45.43	66.73
4	10.77	2.48	10.64	-19.33	-19.33	66.73	68.47
5	10.77	2.85	30.16	-19.33	-16.68	68.47	71.29
6	10.77	2.82	89.97	-16.68	-14.04	71.29	71.80
7	10.77	2.73	85.05	-14.04	-14.04	71.80	59.32
8	10.77	2.43	68.34	-14.04	-14.04	59.32	38.70
9	10.77	1.82	52.59	-14.04	-14.04	38.70	15.17
10	10.77	0.80	23.95	-14.04	-14.04	15.17	0.00

*** NOTES ***
1) ALL ANGLES MEASURED FROM THE (+) X AXIS.
2) ALL FORCES ARE MEASURED IN KIPS.
3) HORIZONTAL EARTHQUAKE FORCE MAY BE CALCULATED BY MULTIPLYING THE TOTAL SLICE WT. BY THE EARTHQUAKE COEFFICIENT

EXHIBIT 1 Continued

SLICE DATA FOR THE CRITICAL ARC AFTER DRAWDOWN
****

FACTOR OF SAFETY AFTER DRAWDOWN= 1.263
LOCATION OF CENTER; X= 706.16 Y= 825.17
ARC RADIUS= 205.17

TABULATION OF SLICE DATA
****

SLICE NO.	SLICE WIDTH	X-COORD. OF SLICE	TOTAL WEIGHT	WATER FORCE	DIR. OF W. FORCE	C-FORCE DEVEL.	DIR. OF C-FORCE
******	******	*******	*******	******	*******	********	********
1	32.00	549.99	63.45	28.95	0.0	7.81	-49.57
2	32.00	581.99	139.65	64.01	0.0	6.36	-37.24
3	32.00	614.00	174.65	80.53	0.0	5.67	-26.69
4	4.00	632.00	22.67	10.60	0.0	0.68	-21.19
5	10.00	639.00	56.81	29.98	0.0	1.68	-19.11
6	31.08	659.54	167.13	79.31	0.0	5.05	-13.14
7	31.03	690.62	143.51	62.64	0.0	4.93	-4.34
8	28.23	720.28	101.99	45.26	0.0	4.43	3.94
9	28.23	743.50	67.91	32.78	0.0	4.57	11.91
10	28.23	776.73	26.45	16.57	0.0	4.76	20.12

SLICE NO.	PHI DEVEL.	NORMAL STRESS	NORMAL FORCE	ALPHA TOP	ALPHA BOTTOM	SIDE FORCES E1	SIDE FORCES E2
******	******	*******	*******	******	*******	********	********
1	21.11	0.59	29.22	-19.33	-19.33	0.0	33.80
2	21.11	1.60	64.38	-19.33	-19.33	33.80	89.81
3	21.11	2.26	80.87	-19.33	-19.33	89.81	131.72
4	21.11	2.48	10.64	-19.33	-19.33	131.72	135.13
5	21.11	2.85	30.16	-19.33	-16.68	135.13	140.53
6	21.11	2.82	89.97	-16.68	-14.04	140.53	138.47
7	21.11	2.73	85.05	-14.04	-14.04	138.47	111.19
8	21.11	2.43	58.34	-14.04	-14.04	111.19	71.16
9	21.11	1.82	52.59	-14.04	-14.04	71.16	27.92
10	21.11	0.80	23.95	-14.04	-14.04	27.92	0.00

*** NOTES ***
1) ALL ANGLES MEASURED FROM THE (+) X AXIS.
2) ALL FORCES ARE MEASURED IN KIPS.
3) HORIZONTAL EARTHQUAKE FORCE MAY BE CALCULATED BY MULTIPLYING THE TOTAL SLICE WT. BY THE EARTHQUAKE COEFFICIENT

EXHIBIT 2

PROGRAM SLOPE - SLOPE STABILITY DATE: 2/ 9/34

MIDDLE CREEK DAM 000100
FEBRUAY 9 1984 000200
SUDDEN DRAWDOWN WITH THE REINFORCED EARTH ALTERNATIVE 000300
COE PROGRAM WITH SEISMIC COEF, OF 0.0 000400
FAILURE PLANE ELEVATION 647 000500

****

INPUT DATA

MINIMUM ELEVATION OF CIRCLE= 647.0 X-START= 660.0 Y-START= 840.0
SEARCH INCREMENT= 0.0 F.S. MIN.= 0.0

NUMBER OF LINES= 24 MIN. NO. OF SLICES= 8 SEISMIC COEFF.= 0.0

EMBANKMENT AND FOUNDATION PROFILE:
****

XTOEL	YTOEL	XTOPL	YTOPL	XTOER	YTOER	XTOPR	YTOPR
*****	*****	*****	*****	*****	*****	*****	*****
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

	X1(I)	Y1(I)	X2(I)	Y2(I)	TYPE
(1)	-5000.0	606.0	206.0	606.0	5
(2)	206.0	606.0	230.0	620.0	5
(3)	230.0	620.0	313.0	641.0	3
(4)	313.0	641.0	400.0	682.0	3
(5)	400.0	682.0	410.0	682.0	3
(6)	410.0	682.0	438.0	722.0	3
(7)	488.0	722.0	495.0	722.0	3
(8)	488.0	722.0	488.1	732.0	1
(9)	488.1	732.0	516.0	732.0	1
(10)	516.0	732.0	516.1	720.0	1
(11)	516.1	720.0	644.0	675.0	1
(12)	644.0	675.0	796.0	637.0	1
(13)	796.0	637.0	943.0	632.0	1
(14)	943.0	632.0	5000.0	632.0	4
(15)	230.0	620.0	382.0	603.0	5
(16)	382.0	603.0	495.0	722.0	2
(17)	382.0	603.0	506.0	611.0	5
(18)	495.0	722.0	630.0	614.0	2
(19)	506.0	611.0	630.0	614.0	4
(20)	506.0	611.0	634.0	590.0	5
(21)	630.0	614.0	916.0	597.0	4
(22)	634.0	590.0	5000.0	590.0	5
(23)	916.0	597.0	943.0	632.0	4
(24)	-5000.0	560.0	5000.0	560.0	6

SOIL CONSTANTS;
****
NUMBER OF SOIL TYPES= 6

	TYPE NO.	WT.MOIST	WT. SAT.	C(1)	PHI(1)	C(2)	PHI(2)
(1)	1	120.0	125.0	200.0	26.0	0.0	0.0
(2)	2	125.0	130.0	0.0	30.0	0.0	0.0
(3)	3	125.0	130.0	0.0	34.0	0.0	0.0
(4)	4	100.0	105.0	0.0	28.0	0.0	0.0
(5)	5	130.0	130.0	0.0	34.0	0.0	0.0
(6)	6	135.0	135.0	1000.0	30.0	0.0	0.0

```
PIEZOMETRIC SURFACE DATA;
*************************
NUMBER OF POINTS FOR PIEZOMETRIC SURFACE=  9

      XPIEZ(I)    YPIEZ(I)
( 1)   5000.0       721.0
( 2)    513.0       721.0
( 3)    517.0       710.0
( 4)    512.0       700.0
( 5)    420.0       670.0
( 6)    369.0       632.0
( 7)    326.0       620.0
( 8)    220.0       611.0
( 9)  -5000.0       611.0

DRAWDOWN SURFACE DATA;
**********************
NUMBER OF POINTS FOR DRAWDOWN SURFACE=  4
       XSUD(I)    YSUD(I)
( 1)   5000.0       637.0
( 2)    796.0       637.0
( 3)    220.0       611.0
( 4)  -5000.0       606.0
************************************************************************
```

MIDDLE CREEK DAM
FEBRUAY 9 1984
SUDDEN DRAWDOWN WITH THE REINFORCED EARTH ALTERNATIVE
COE PROGRAM WITH SEISMIC COEF, OF 0.0
FAILURE PLANE ELEVATION 647

CHECK DATA FOR THE BEFORE OR NON-DRAWDOWN CRITICAL ARC
****

FACTOR OF SAFETY= 2.119
LOCATION OF CENTER; X= 660.00 Y= 840.00
ARC RADIUS= 193.00

TABULATION OF SLICE DATA
****

SLICE NO.	SLICE WIDTH	X-COORD. OF SLICE	TOTAL WEIGHT	WATER FORCE	DIR. OF W. FORCE	C-FORCE DEVEL.	DIR. OF C-FORCE
*******	*******	********	********	*******	********	*********	*********
1	5.95	503.02	3.06	0.0	0.0	0.97	-54.42
2	10.00	511.00	17.60	0.0	0.0	1.43	-50.54
3	0.10	516.05	0.17	0.0	0.0	0.01	-48.23
4	22.78	527.49	46.45	23.19	90.00	2.96	-43.36
5	22.78	550.27	76.17	38.02	90.00	2.61	-34.65
6	22.78	573.05	91.88	45.87	90.00	2.41	-26.78
7	22.78	595.33	96.72	48.28	90.00	2.23	-19.42
8	22.78	618.61	92.38	46.12	90.00	2.20	-12.38
9	4.00	632.00	15.09	7.53	90.00	0.38	-8.34
10	10.00	639.00	35.77	17.85	90.00	0.95	-6.25
11	16.00	652.00	51.67	25.79	90.00	1.51	-2.38
12	19.67	669.84	52.36	26.14	90.00	1.86	2.92
13	19.67	689.51	35.30	17.62	90.00	1.83	8.79
14	19.67	709.18	13.11	6.55	90.00	1.92	14.76

SLICE NO.	PHI DEVEL.	NORMAL STRESS	NORMAL FORCE	ALPHA TOP	ALPHA BOTTOM	SIDE FORCES E1	SIDE FORCES E2
*******	*******	********	********	*******	********	*********	*********
1	12.96	0.29	2.96	0.0	0.0	0.0	1.45
2	12.96	0.71	11.13	0.0	-44.76	1.45	10.52
3	12.96	-11.48	-1.72	-44.76	-54.45	10.52	11.08
4	12.96	0.36	26.91	-54.45	-19.38	11.03	19.36
5	12.96	1.24	34.42	-19.38	-19.38	19.36	30.92
6	12.96	1.65	42.20	-19.38	-19.38	30.92	39.60
7	12.96	1.89	45.68	-19.38	-19.38	39.60	42.91
8	12.96	1.95	45.53	-19.38	-19.38	42.91	40.14
9	12.96	1.30	7.68	-19.38	-19.38	40.14	39.06
10	12.96	2.02	20.34	-19.38	-16.71	39.06	34.94
11	12.96	1.31	23.93	-16.71	-14.04	34.94	27.31
12	12.96	1.48	29.21	-14.04	-14.04	27.31	16.94
13	12.96	1.08	21.43	-14.04	-14.04	16.94	6.60
14	12.96	0.47	9.53	-14.04	-14.04	6.60	-0.00

*** NOTES ***
1) ALL ANGLES MEASURED FROM THE (+) X AXIS.
2) ALL FORCES ARE MEASURED IN KIPS.
3) HORIZONTAL EARTHQUAKE FORCE MAY BE CALCULATED BY MULTIPLYING THE
TOTAL SLICE WT. BY THE EARTHQUAKE COEFFICIENT

EXHIBIT 2 Continued

SLICE DATA FOR THE CRITICAL ARC AFTER DRAWDOWN

****

FACTOR OF SAFETY AFTER DRAWDOWN= 1.162
LOCATION OF CENTER; X= 660.00 Y= 340.00
ARC RADIUS= 193.00

TABULATION OF SLICE DATA

****

SLICE NO.	SLICE WIDTH	X-COORD. OF SLICE	TOTAL WEIGHT	WATER FORCE	DIR. OF W. FORCE	C-FORCE DEVEL.	DIR. OF C-FORCE
1	5.95	503.02	3.06	0.00	0.0	1.76	-54.42
2	10.00	511.00	17.60	-15.69	0.0	2.71	-50.54
3	0.10	516.05	0.17	-0.28	0.0	0.03	-48.23
4	22.78	527.49	46.45	13.62	0.0	5.39	-43.36
5	22.78	550.27	76.17	34.81	0.0	4.77	-34.65
6	22.78	573.05	91.83	42.33	0.0	4.39	-26.78
7	22.78	595.83	96.72	45.54	0.0	4.16	-19.42
8	22.78	618.61	92.38	45.11	0.0	4.01	-12.38
9	4.00	632.00	15.09	7.59	0.0	0.70	-3.34
10	10.00	639.00	35.77	19.47	0.0	1.73	-6.25
11	16.00	652.00	51.67	28.02	0.0	2.76	-2.38
12	19.67	669.84	52.36	28.66	0.0	3.39	2.92
13	19.67	689.51	35.30	20.91	0.0	3.43	8.79
14	19.67	709.18	13.11	9.11	0.0	3.50	14.76

SLICE NO.	PHI DEVEL.	NORMAL STRESS	NORMAL FORCE	ALPHA TOP	ALPHA BOTTOM	SIDE FORCES E1	SIDE FORCES E2
1	22.77	0.17	1.76	0.0	0.0	0.0	0.0
2	22.77	1.69	26.56	0.0	-44.76	0.0	0.0
3	22.77	2.31	0.35	-44.76	-54.45	0.0	0.0
4	22.77	0.86	26.91	-54.45	-19.38	0.0	16.64
5	22.77	1.24	34.42	-19.38	-19.38	16.64	41.62
6	22.77	1.65	42.20	-19.38	-19.38	41.62	61.08
7	22.77	1.89	45.68	-19.38	-19.38	61.08	69.91
8	22.77	1.95	45.53	-19.38	-19.38	69.91	66.58
9	22.77	1.90	7.68	-19.38	-19.38	66.58	64.81
10	22.77	2.02	20.34	-19.38	-16.71	64.81	57.70
11	22.77	1.81	28.93	-16.71	-14.04	57.70	44.04
12	22.77	1.48	29.21	-14.04	-14.04	44.04	24.39
13	22.77	1.08	21.48	-14.04	-14.04	24.39	5.54
14	22.77	0.47	9.53	-14.04	-14.04	5.54	-6.84

*** NOTES ***

1) ALL ANGLES MEASURED FROM THE (+) X AXIS.
2) ALL FORCES ARE MEASURED IN KIPS.
3) HORIZONTAL EARTHQUAKE FORCE MAY BE CALCULATED BY MULTIPLYING THE TOTAL SLICE WT. BY THE EARTHQUAKE COEFFICIENT

EXHIBIT 3

PROGRAM SLOPE - SLOPE STABILITY DATE: 2/ 9/84

MIDDLE CREEK DAM
FEBRUARY 9 1984
STEADY STATE SEEPAGE DOWNSTREAM SLOPE FOR EXISTING STRUCTURE
COE PROGRAM WITH SEISMIC COEF, OF 0.0
FAILURE PLANE ELEVATION 630

****

INPUT DATA

MINIMUM ELEVATION OF CIRCLE= 630.0 X-START= 330.0 Y-START= 810.0
SEARCH INCREMENT= 5.0 F.S. MIN.= 0.0

NUMBER OF LINES= 22 MIN. NO. OF SLICES= 8 SEISMIC COEFF.= 0.0

EMBANKMENT AND FOUNDATION PROFILE:
****

XTOEL	YTOEL	XTOPL	YTOPL	XTOER	YTOER	XTOPR	YTOPR
*****	*****	*****	*****	*****	*****	*****	*****
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

	X1(I)	Y1(I)	X2(I)	Y2(I)	TYPE
(1)	-5000.0	606.0	206.0	606.0	5
(2)	206.0	606.0	230.0	620.0	5
(3)	230.0	620.0	313.0	641.0	3
(4)	313.0	641.0	400.0	682.0	3
(5)	400.0	682.0	410.0	682.0	3
(6)	410.0	682.0	490.0	722.0	3
(7)	490.0	722.0	495.0	722.0	3
(8)	495.0	722.0	510.0	722.0	1
(9)	510.0	722.0	644.0	675.0	1
(10)	644.0	675.0	796.0	637.0	1
(11)	796.0	637.0	943.0	632.0	1
(12)	943.0	632.0	5000.0	632.0	4
(13)	230.0	620.0	382.0	603.0	5
(14)	332.0	608.0	495.0	722.0	2
(15)	382.0	603.0	506.0	611.0	5
(16)	495.0	722.0	630.0	614.0	2
(17)	506.0	611.0	630.0	614.0	4
(18)	506.0	611.0	634.0	590.0	5
(19)	630.0	614.0	916.0	597.0	4
(20)	634.0	590.0	5000.0	590.0	5
(21)	916.0	597.0	943.0	632.0	4
(22)	-5000.0	560.0	5000.0	560.0	6

SOIL CONSTANTS:
****
NUMBER OF SOIL TYPES= 6

	TYPE NO.	WT.MOIST	WT. SAT.	C(1)	PHI(1)	C(2)	PHI(2)
(1)	1	120.0	125.0	200.0	32.5	0.0	0.0
(2)	2	125.0	130.0	0.0	34.0	0.0	0.0
(3)	3	125.0	130.0	0.0	34.0	0.0	0.0
(4)	4	100.0	105.0	0.0	28.0	0.0	0.0
(5)	5	130.0	130.0	0.0	34.0	0.0	0.0
(6)	6	135.0	135.0	1000.0	30.0	0.0	0.0

```
PIEZOMETRIC SURFACE DATA;
*************************
NUMBER OF POINTS FOR PIEZOMETRIC SURFACE=  9

        XPIEZ(I)     YPIEZ(I)
( 1)    5000.0        716.0
( 2)     524.0        716.0
( 3)     514.0        699.0
( 4)     416.0        665.0
( 5)     369.0        632.0
( 6)     326.0        620.0
( 7)     220.0        611.0
( 8)     206.0        606.0
( 9)   -5000.0        606.0

DRAWDOWN SURFACE DATA;
**********************
NUMBER OF POINTS FOR DRAWDOWN SURFACE=  0
***************************************************************************
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MIDDLE CREEK DAM
FEBRUARY 9 1984
STEADY STATE SEEPAGE DOWNSTREAM SLOPE FOR EXISTING STRUCTURE
COE PROGRAM WITH SEISMIC COEF, OF 0.0
FAILURE PLANE ELEVATION 630

CHECK DATA FOR THE BEFORE OR NON-DRAWDOWN CRITICAL ARC
****
FACTOR OF SAFETY= 1.667
LOCATION OF CENTER: X= 341.04 Y= 820.15
ARC RADIUS= 190.16

TABULATION OF SLICE DATA
****

SLICE NO.	SLICE WIDTH	X-COORD. OF SLICE	TOTAL WEIGHT	WATER FORCE	DIR. OF W. FORCE	C-FORCE DEVEL.	DIR. OF C-FORCE
1	2.96	502.42	0.85	0.0	0.0	0.67	58.07
2	5.94	497.97	6.73	0.0	0.0	0.0	55.62
3	5.00	492.50	10.51	0.0	0.0	0.0	52.80
4	22.83	478.59	78.23	0.0	0.0	0.0	46.33
5	22.83	455.76	104.96	15.51	109.13	0.0	37.11
6	22.83	432.94	115.49	25.93	109.13	0.0	28.90
7	11.52	415.76	58.14	17.16	125.07	0.0	23.14
8	10.00	405.00	51.96	12.35	125.07	0.0	19.66
9	18.00	391.00	93.40	14.78	125.07	0.0	15.23
10	20.48	371.76	92.81	2.25	125.07	0.0	9.30
11	20.48	351.28	73.64	0.0	0.0	0.0	3.09
12	14.02	334.03	36.42	0.0	0.0	0.0	-2.11
13	14.02	320.01	23.02	0.0	0.0	0.0	-6.35
14	19.56	303.22	11.56	0.0	0.0	0.0	-11.47

SLICE NO.	PHI DEVEL.	NORMAL STRESS	NORMAL FORCE	ALPHA TOP	ALPHA BOTTOM	SIDE FORCES E1	SIDE FORCES E2
1	20.91	0.08	0.45	0.0	0.0	0.0	0.0
2	22.03	0.72	7.55	0.0	0.0	0.0	4.51
3	22.03	1.08	8.94	0.0	13.28	4.51	9.70
4	22.03	1.90	62.89	13.28	26.57	9.70	41.77
5	22.03	2.59	74.26	26.57	26.57	41.77	70.76
6	22.03	2.93	76.39	26.57	26.57	70.76	91.29
7	22.03	4.66	58.44	26.57	13.28	91.29	95.29
8	22.03	3.65	38.77	13.28	12.62	95.29	100.54
9	22.03	2.81	52.45	12.62	25.23	100.54	110.45
10	22.03	4.63	96.11	25.23	25.23	110.45	86.63
11	22.03	4.20	36.10	25.23	25.23	86.63	53.30
12	22.03	3.34	46.90	25.23	25.23	53.30	30.43
13	22.03	2.42	34.14	25.23	19.72	30.43	10.65
14	22.03	0.84	16.85	19.72	14.20	10.65	-0.00

*** NOTES ***
1) ALL ANGLES MEASURED FROM THE (+) X AXIS.
2) ALL FORCES ARE MEASURED IN KIPS.
3) HORIZONTAL EARTHQUAKE FORCE MAY BE CALCULATED BY MULTIPLYING THE TOTAL SLICE WT. BY THE EARTHQUAKE COEFFICIENT

EXHIBIT 4

PROGRAM SLOPE - SLOPE STABILITY DATE: 2/ 9/84

MIDDLE CREEK DAM
FEBRUARY 9 1984
STEADY STATE SEEPAGE DOWNSTREAM SLOPE FOR REINFORCED EARTH ALTERNATIVE
COE PROGRAM WITH SEISMIC COEF, OF 0.0
FAILURE PLANE ELEVATION 630
690

****

INPUT DATA

MINIMUM ELEVATION OF CIRCLE= 630.0 X-START= 340.0 Y-START= 820.0
SEARCH INCREMENT= 0.0 F.S. MIN.= 0.0

NUMBER OF LINES= 24 MIN. NO. OF SLICES= 8 SEISMIC COEFF.= 0.0

EMBANKMENT AND FOUNDATION PROFILE:
****

XTOEL	YTOEL	XTOPL	YTOPL	XTOER	YTOER	XTOPR	YTOPR
*****	*****	*****	*****	*****	*****	*****	*****
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

	X1(I)	Y1(I)	X2(I)	Y2(I)	TYPE
(1)	-5000.0	606.0	206.0	606.0	5
(2)	206.0	606.0	230.0	620.0	5
(3)	230.0	620.0	313.0	641.0	3
(4)	313.0	641.0	400.0	682.0	3
(5)	400.0	682.0	410.0	682.0	3
(6)	410.0	682.0	488.0	722.0	3
(7)	488.0	722.0	495.0	722.0	3
(8)	488.0	722.0	488.1	732.0	1
(9)	488.1	732.0	516.0	732.0	1
(10)	516.0	732.0	516.1	720.0	1
(11)	516.1	720.0	644.0	675.0	1
(12)	644.0	675.0	796.0	637.0	1
(13)	796.0	637.0	943.0	632.0	1
(14)	943.0	632.0	5000.0	632.0	4
(15)	230.0	620.0	382.0	608.0	5
(16)	382.0	608.0	495.0	722.0	2
(17)	382.0	608.0	506.0	611.0	5
(18)	495.0	722.0	630.0	614.0	2
(19)	506.0	611.0	630.0	614.0	4
(20)	506.0	611.0	634.0	590.0	5
(21)	630.0	614.0	916.0	597.0	4
(22)	634.0	590.0	5000.0	590.0	5
(23)	916.0	597.0	943.0	632.0	4
(24)	-5000.0	560.0	5000.0	560.0	6

SOIL CONSTANTS:
****
NUMBER OF SOIL TYPES= 6

	TYPE NO.	WT.MOIST	WT. SAT.	C(1)	PHI(1)	C(2)	PHI(2)
(1)	1	120.0	125.0	200.0	32.5	0.0	0.0
(2)	2	125.0	130.0	0.0	34.0	0.0	0.0
(3)	3	125.0	130.0	0.0	34.0	0.0	0.0
(4)	4	100.0	105.0	0.0	28.0	0.0	0.0
(5)	5	130.0	130.0	0.0	34.0	0.0	0.0
(6)	6	135.0	135.0	1000.0	30.0	0.0	0.0

```
PIEZOMETRIC SURFACE DATA;
*************************
NUMBER OF POINTS FOR PIEZOMETRIC SURFACE=  9

      XPIEZ(I)    YPIEZ(I)
( 1)   5000.0       721.0
( 2)    518.0       721.0
( 3)    517.0       710.0
( 4)    512.0       700.0
( 5)    420.0       670.0
( 6)    369.0       632.0
( 7)    326.0       620.0
( 8)    220.0       611.0
( 9)  -5000.0       611.0

DRAWDOWN SURFACE DATA;
**********************
NUMBER OF POINTS FOR DRAWDOWN SURFACE=  0
**************************************************************************
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MIDDLE CREEK DAM
FEBRUARY 9 1984
STEADY STATE SEEPAGE DOWNSTREAM SLOPE FOR REINFORCED EARTH ALTERNATIVE
COE PROGRAM WITH SEISMIC COEF, OF 0.0
FAILURE PLANE ELEVATION 630

CHECK DATA FOR THE BEFORE OR NON-DRAWDOWN CRITICAL ARC
****

FACTOR OF SAFETY= 1.581
LOCATION OF CENTER; X= 340.00 Y= 820.00
ARC RADIUS= 190.00

TABULATION OF SLICE DATA
****

SLICE NO.	SLICE WIDTH	X-COORD. OF SLICE	TOTAL WEIGHT	WATER FORCE	DIR. OF W. FORCE	C-FORCE DEVEL.	DIR. OF C-FORCE
1	2.39	507.20	0.65	0.0	0.0	0.64	61.64
2	5.80	503.10	6.59	0.0	0.0	1.43	59.14
3	5.20	497.60	11.46	0.0	0.0	0.0	56.04
4	6.90	491.55	22.60	0.0	0.0	0.0	52.90
5	0.10	483.05	0.32	0.0	0.0	0.0	51.19
6	21.84	477.08	76.36	0.25	108.06	0.0	46.18
7	21.84	455.24	100.37	18.00	108.06	0.0	37.34
8	21.84	433.40	109.89	28.42	108.06	0.0	29.44
9	12.48	416.24	62.52	20.62	126.69	0.0	23.66
10	10.00	405.00	51.54	13.51	126.69	0.0	20.01
11	18.00	391.00	92.80	16.00	126.69	0.0	15.57
12	21.00	371.50	94.47	2.02	126.69	0.0	9.54
13	21.00	350.50	74.50	0.0	0.0	0.0	3.17
14	13.50	333.25	34.46	0.0	0.0	0.0	-2.04
15	13.50	319.75	22.10	0.0	0.0	0.0	-6.12
16	20.07	302.97	12.08	0.0	0.0	0.0	-11.24

SLICE NO.	PHI DEVEL.	NORMAL STRESS	NORMAL FORCE	ALPHA TOP	ALPHA BOTTOM	SIDE FORCES E1	SIDE FORCES E2
1	21.95	0.06	0.31	0.0	0.0	0.0	0.0
2	21.95	0.55	6.24	0.0	0.0	0.0	3.33
3	23.11	1.35	12.56	0.0	0.0	3.33	10.76
4	23.11	0.71	8.08	0.0	44.71	10.76	21.28
5	23.11	-32.24	-5.14	44.71	58.29	21.28	23.77
6	23.11	2.34	73.76	58.29	27.15	23.77	49.44
7	23.11	2.46	67.51	27.15	27.15	49.44	75.99
8	23.11	2.74	68.60	27.15	27.15	75.99	95.14
9	23.11	4.44	60.55	27.15	13.57	95.14	100.41
10	23.11	3.56	37.83	13.57	12.62	100.41	106.01
11	23.11	2.64	49.34	12.62	25.23	106.01	117.14
12	23.11	4.63	98.52	25.23	25.23	117.14	90.70
13	23.11	4.18	87.93	25.23	25.23	90.70	54.66
14	23.11	3.33	44.96	25.23	25.23	54.66	31.69
15	23.11	2.46	33.41	25.23	19.72	31.69	11.62
16	23.11	0.87	17.83	19.72	14.20	11.62	-0.00

*** NOTES ***
1) ALL ANGLES MEASURED FROM THE (+) X AXIS.
2) ALL FORCES ARE MEASURED IN KIPS.

EXHIBIT 5

PROGRAM SLOPE - SLOPE STABILITY DATE: 2/ 9/84

MIDDLE CREEK DAM
FEBRUARY 9 1984
STEADY STATE SEEPAGE DOWNSTREAM SLOPE FOR EXISTING STRUCTURE
COE PROGRAM WITH SEISMIC COEF, OF 0.18
FAILURE PLANE ELEVATION 630

****

INPUT DATA

MINIMUM ELEVATION OF CIRCLE= 630.0 X-START= 330.0 Y-START= 810.0
SEARCH INCREMENT= 5.0 F.S. MIN.= 0.0

NUMBER OF LINES= 22 MIN. NO. OF SLICES= 8 SEISMIC COEFF.= 0.180

EMBANKMENT AND FOUNDATION PROFILE:
****

XTOEL	YTOEL	XTOPL	YTOPL	XTOER	YTOER	XTOPR	YTOPR
*****	*****	*****	*****	*****	*****	*****	*****
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

	X1(I)	Y1(I)	X2(I)	Y2(I)	TYPE
(1)	-5000.0	606.0	206.0	606.0	5
(2)	206.0	606.0	230.0	620.0	5
(3)	230.0	620.0	313.0	641.0	3
(4)	313.0	641.0	400.0	682.0	3
(5)	400.0	682.0	410.0	682.0	3
(6)	410.0	682.0	490.0	722.0	3
(7)	490.0	722.0	495.0	722.0	3
(8)	495.0	722.0	510.0	722.0	1
(9)	510.0	722.0	644.0	675.0	1
(10)	644.0	675.0	796.0	637.0	1
(11)	796.0	637.0	943.0	632.0	1
(12)	943.0	632.0	5000.0	632.0	4
(13)	230.0	620.0	382.0	608.0	5
(14)	382.0	608.0	495.0	722.0	2
(15)	382.0	608.0	506.0	611.0	5
(16)	495.0	722.0	630.0	614.0	2
(17)	506.0	611.0	630.0	614.0	4
(13)	506.0	611.0	634.0	590.0	5
(19)	630.0	614.0	916.0	597.0	4
(20)	634.0	590.0	5000.0	590.0	5
(21)	916.0	597.0	943.0	632.0	4
(22)	-5000.0	560.0	5000.0	560.0	6

SOIL CONSTANTS;
****
NUMBER OF SOIL TYPES= 6

	TYPE NO.	WT.MOIST	WT. SAT.	C(1)	PHI(1)	C(2)	PHI(2)
(1)	1	120.0	125.0	200.0	32.5	0.0	0.0
(2)	2	125.0	130.0	0.0	34.0	0.0	0.0
(3)	3	125.0	130.0	0.0	34.0	0.0	0.0
(4)	4	100.0	105.0	0.0	28.0	0.0	0.0
(5)	5	130.0	130.0	0.0	34.0	0.0	0.0
(6)	6	135.0	135.0	1000.0	30.0	0.0	0.0

```
PIEZOMETRIC SURFACE DATA;
*************************
NUMBER OF POINTS FOR PIEZOMETRIC SURFACE=  9

      XPIEZ(I)    YPIEZ(I)
( 1)   5000.0       716.0
( 2)    524.0       716.0
( 3)    514.0       699.0
( 4)    416.0       665.0
( 5)    369.0       632.0
( 6)    326.0       620.0
( 7)    220.0       611.0
( 8)    206.0       606.0
( 9)  -5000.0       606.0

DRAWDOWN SURFACE DATA;
**********************
NUMBER OF POINTS FOR DRAWDOWN SURFACE=  0
************************************************************************
```

MIDDLE CREEK DAM
FEBRUARY 9 1984
STEADY STATE SEEPAGE DOWNSTREAM SLOPE FOR EXISTING STRUCTURE
COE PROGRAM WITH SEISMIC COEF, OF 0.18
FAILURE PLANE ELEVATION 630

CHECK DATA FOR THE BEFORE OR NON-DRAWDOWN CRITICAL ARC

FACTOR OF SAFETY= 1.089
LOCATION OF CENTER; X= 340.78 Y= 320.43
ARC RADIUS= 190.43

TABULATION OF SLICE DATA

SLICE NO.	SLICE WIDTH	X-COORD. OF SLICE	TOTAL WEIGHT	WATER FORCE	DIR. OF W. FORCE	C-FORCE DEVEL.	DIR. OF C-FORCE
1	2.93	502.34	0.84	0.0	0.0	1.02	58.04
2	5.87	497.94	6.62	0.0	0.0	0.0	55.62
3	5.00	492.50	10.41	0.0	0.0	0.0	52.82
4	22.77	478.62	77.63	0.0	0.0	0.0	46.37
5	22.77	455.85	104.26	15.23	109.13	0.0	37.18
6	22.77	433.08	114.87	25.66	109.13	0.0	28.99
7	11.69	415.85	58.88	17.36	125.07	0.0	23.22
8	10.00	405.00	51.86	12.29	125.07	0.0	19.71
9	18.00	391.00	93.25	14.69	125.07	0.0	15.29
10	20.61	371.69	93.24	2.15	125.07	0.0	9.34
11	20.61	351.08	73.86	0.0	0.0	0.0	3.10
12	13.89	333.83	35.93	0.0	0.0	0.0	-2.09
13	13.89	319.94	22.80	0.0	0.0	0.0	-6.28
14	19.71	303.14	11.71	0.0	0.0	0.0	-11.40

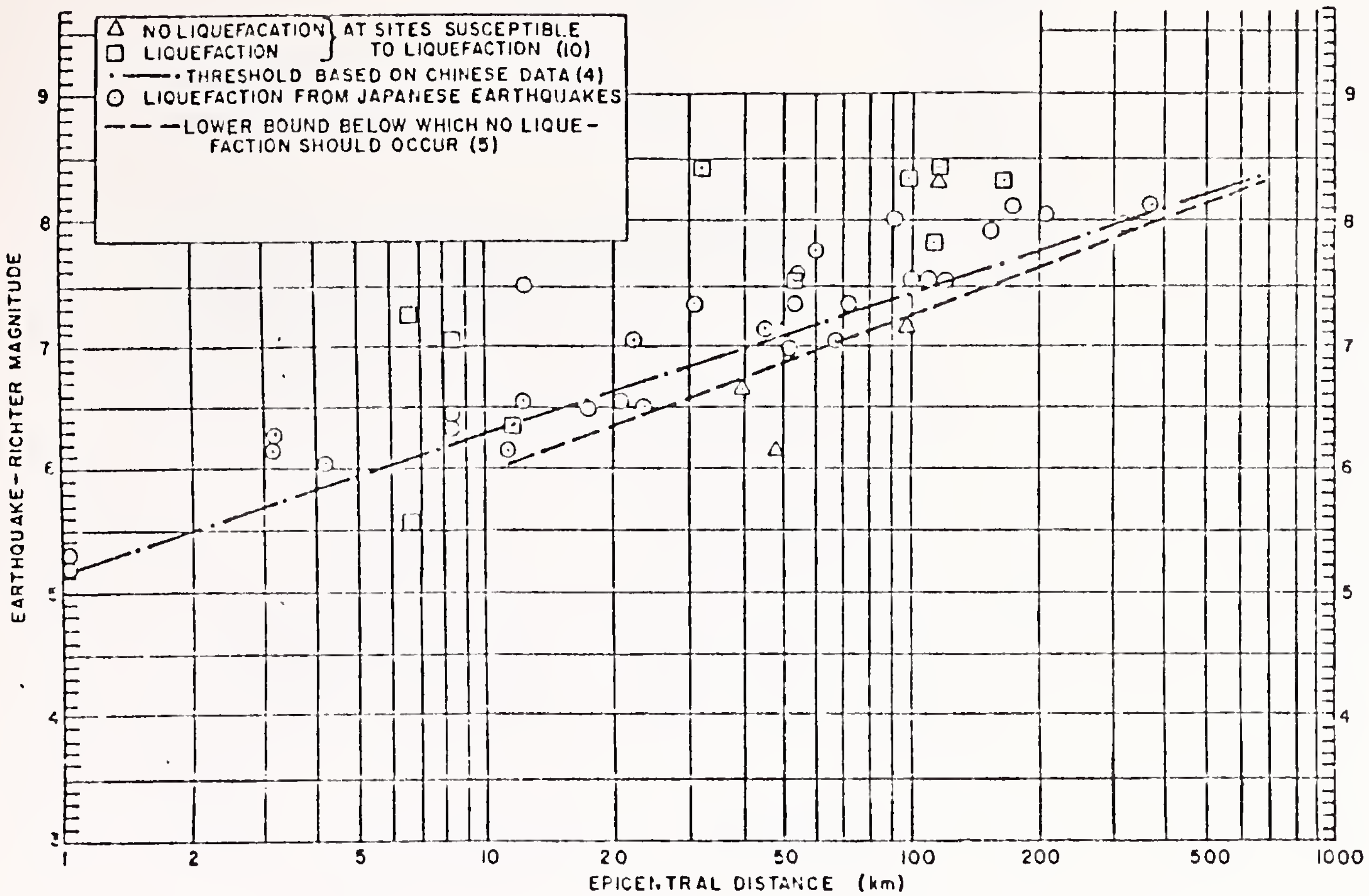
SLICE NO.	PHI DEVEL.	NORMAL STRESS	NORMAL FORCE	ALPHA TOP	ALPHA BOTTOM	SIDE FORCES E1	SIDE FORCES E2
1	30.33	0.08	0.44	0.0	0.0	0.0	0.0
2	31.78	0.59	6.15	0.0	0.0	0.0	4.12
3	31.78	0.91	7.51	0.0	13.28	4.12	9.41
4	31.78	1.61	53.02	13.28	26.57	9.41	43.43
5	31.78	2.23	63.59	26.57	26.57	43.43	77.86
6	31.78	2.55	66.37	26.57	26.57	77.86	106.13
7	31.78	4.54	57.82	26.57	13.28	106.13	108.27
8	31.78	3.39	35.99	13.28	12.62	108.27	115.71
9	31.78	2.29	42.65	12.62	25.23	115.71	136.98
10	31.78	4.54	94.76	25.23	25.23	136.98	109.86
11	31.78	4.23	88.24	25.23	25.23	109.86	69.48
12	31.78	3.54	49.24	25.23	25.23	69.48	40.94
13	31.78	2.74	38.28	25.23	19.72	40.94	14.21
14	31.78	0.96	19.24	19.72	14.20	14.21	-0.00

*** NOTES ***
1) ALL ANGLES MEASURED FROM THE (+) X AXIS.
2) ALL FORCES ARE MEASURED IN KIPS.
3) HORIZONTAL EARTHQUAKE FORCE MAY BE CALCULATED BY MULTIPLYING THE TOTAL SLICE WT. BY THE EARTHQUAKE COEFFICIENT

EXHIBIT 6

BUREAU OF RECLAMATION'S SEISMIC REEVALUATION OF EMBANKMENT DAMS CRITERIA

1. Liquifaction Criteria



2. If the embankment and foundation soils suffer a considerable loss in strength under cyclic loading, the above chart is used to determine the likelihood of liquifaction. If the chart indicates that liquifaction is a possibility, a liquifaction analysis is performed.

3. Deformation Criteria

For a dam and foundation not subject to liquefaction, deformation should not be a problem if the following conditions are satisfied: (1) the dam is a well built (densely compacted) dam and peak accelerations are 0.2g or less, or the dam is constructed on clay or rock foundations and peak accelerations are 0.4g or less [13]; (2) the slopes of the dam are 3 horizontal to 1 vertical or flatter; (3) the static factors of safety of the critical failure surfaces involving the crest (other than the infinite slope case) are greater than 1.5 under loading conditions expected prior to an earthquake; and (4) the freeboard at the time of an earthquake is 2 to 3 percent of the embankment height but not less than 5 feet (1.5 meters).

4. Deformation Analysis

If the above criteria is not met, a deformation analysis is required.

Source: see reference 19

EXHIBIT 7

CALIFORNIA'S SEISMIC REEVALUATION OF DAMS CRITERIA

1. Comparison Procedure

A. Assign the soil type(s) using the Unified Soil Classification System.

B. Determine the relative density or compaction based on construction testing and where available subsequent exploration and testing. Place it in one of the following four categories listed.

	Relative Compaction (ASTM 1557)	Relative Density
Very Dense	100-95.0	100-80
Dense	91.3-94.9	79-65
Medium Dense	88.7-91.2	64-55
Loose	88.6 or less	55 or less

In many cases the relative compaction testing was done using a standard other than ASTM 1557. However, Safety of Dams routinely faces this problem so conversion curves for various soils were included in the procedure.

C. Classify the level of acceleration.

	Peak Ground Acceleration
Low	0.2g or less
Medium	0.21 to 0.39g
High	0.40g or greater

It was noted that the duration of shaking was not included but should be in deciding borderline cases particularily for loose soils.

D. Determine the soil group and possible behavior.

Soil Group	Classification	Behavior
I	GW, GP, GM, SW, SP	Liquefaction
II	SM, ML	Liquefaction and/or settlement-slip circle
III	GC, SC, CL, OL, MH, CH, OH	Settlement-slip circle

E. Predict behavior using the following chart.

	Acceleration: Low 0-0.2	Medium .21-.39	High .40+
Loose	1	2	4
Medium Dense		3	5
Dense			6
Very Dense		7	

Zones 1, 3, & 6 — Borderline Zones - Cases that fall in these zones may or may not present a problem. A small investigative program is desirable to determine if there is a problem. Group III soils (clayey) might experience 0-5 percent settlement. There is some possibility for liquefaction of Groups I and II soils.

Zones 2 & 5 — Problem Zones - Cases that fall in these zones will usually present some type of problem. An investigative program would be very desirable. Settlement for Group II soils might range from 5-10 percent. Liquefaction for Groups I and II is very possible.

Zone 4 — Real Problem Zone - An investigative program should be initiated immediately. Settlement for Group III soils might range from 10-20 percent. Probability of liquefaction for soil Groups I and II is very high.

Zone 7 — No Problem - Cases that fall in this zone will normally not present any problems.

2. Liquifaction Analysis

3. Deformation Analysis

Source: see reference 19

INCLINOMETER DATA SHEET

SHEET 1 OF 1

HKM ASSOCIATES

Project Name: Middle Creek Dam
Job No.: 8M057-113C
Readings By: D. Dyer / R.A. Clark
Calculated By: D. Dyer

Observation Well No.: DH-5
Date: 10/6/83
Well Location: downstream face
Instrument No. 1711

Total Depth Of Well: 59.0'
Depth To Groundwater: 22.5'
Casing Size: 2.25"

Sign Convention

DEPTH	DIFF.	A N	A S	DIFF.	CHANGE	DIFF.	B N	B S	DIFF.	CHANGE
56		-117 -114 -116	-120 -121 -120	+4			-98 -98 -98	+96 +92 +96	-194	
54		+1 -2 +0	-4 -5 -4	+4			-11 -9 -10	+11 +9 +10	-20	
52		-83 -86 +84	+83 +81 +82	+2			-82 -80 -81	+75 +73 +74	-155	
50		-12 -15 -14	+0 +0 +0	-14			-156 -153 -154	+147 +145 +146	-300	
48		+22 +18 +20	-29 -30 -30	+50			-223 -221 -222	+215 +213 +214	-436	
46		+80 +76 +78	-85 -85 -85	+163			-264 -262 -263	+255 +253 +254	-517	
44		+86 +83 +84	-92 -93 -92	+176			-228 -228 -228	+223 +221 +222	-450	
42		+55 +52 +54	-59 -60 -60	+114			-154 -153 -154	+146 +144 +145	-299	
40		+49 +47 +48	-52 -53 -52	+100			-182 -179 -180	+176 +174 +175	-355	
38		-5 -9 -7	+1 +0 +0	-7			-113 -111 -112	+108 +106 +107	-219	
36		-113 -115 -114	+110 +108 +109	-223			-122 -120 -121	+115 +113 +114	-235	
34		-198 -201 -200	+195 +194 +194	-394			-149 -147 -148	+142 +140 +141	-289	
32		-201 -204 -202	+198 +197 +198	-400			-148 -146 -147	+141 +140 +140	-287	
30		~~-155~~ -156 -156	+150 +149 +150	-306			-113 -111 -112	+105 +105 +105	-217	
28		-111 -114 -112	+106 +106 +106	-218			-289 -287 -288	+286 +284 +285	-573	
26		-26 -30 -28	+21 +20 +120	-148			-376 -374 -375	+373 +371 +372	-747	
24		-29 -32 -30	+25 +24 +24	-54			-368 -366 -367	+365 +361 +363	-730	
22		-34 -39 -36	+31 +32 +32	-68			-255 -255 -255	+250 +251 +250	-505	
20		-18 -22 -20	+10 +13 +12	-32			-89 -87 -88	+86 +84 +85	-173	
18		-36 -38 -37	+26 +24 +25	-62			-40 -38 -39	+29 +31 +30	-69	
16		-41 -45 -43	+37 +38 +38	-81			+92 +94 +93	-100 -100 -100	+193	
14		-19 -23 -21	+16 +15 +16	-37			+127 +131 +129	-136 -138 -137	+266	
12		+9 +6 +8	-14 -15 +14	-6			+76 +78 +77	-85 -87 -86	+163	
10		-11 -14 -12	+5 +4 +4	-16			+29 +31 +30	-38 -40 -39	+69	
8		-4 -7 -6	-1 -2 -2	-4			+21 +23 +22	-29 -30 -30	+52	
6		-76 -79 -78	+73 +72 +72	-150			+1 +2 +2	-11 -13 -12	+14	
4		-46 -49 -47	+40 +40 +40	-87			+61 +63 +62	-70 -72 -71	+133	
2		-66 -67 -66	+63 +61 +62	-128			+78 +82 +80	-81 -86 -84	+164	

Note: Two readings were taken. The average between the two readings is used to calculate the difference. This difference establishes the baseline to be used with future monitoring.