APPENDIX 1
Foundation Treatment for Little Blue Run Dam

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FOUNDATION TREATMENT FOR LITTLE BLUE RUN DAM

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ABSTRACT

Methods of abutment and foundation treatment of high earth and rockfill dams are as many and diversified as are the dam sites themselves. The exploration, planning and execution of this work at the site of a 420-foot high sloping-core dam founded upon sedimentary rocks in western Pennsylvania are discussed, considering the influences of site topography, geology, and hydrology. Hydraulic pressure testing of diamond drill holes during the subsurface investigation was utilized to determine permeability variations. Settlement, sliding and special seepage analyses were carried out to guide the design of the foundation treatment. The grouting program was modified during construction to meet special conditions and was continually evaluated by pressure testing before and after grouting along centerline exploratory borings. Additional foundation preparations utilizing air and water jets, hand cleaning, dental concrete, slush grout, and a grout cap were used to insure a satisfactory bond of impervious core material to rock. Special consideration was given to the problems associated with the presence of "country bank" coal mines in the abutment areas. A variety of techniques and materials were used to backfill the mines. Backfill materials included fly ash-cement and sand-cement grout. The adequacy of the backfilling operations in critical areas was later checked using water pressure tests. An assessment of final performance of the dam must await its completion in 1976.

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Introduction

For a 420-foot high sloping-core, rock-fill dam, located on sedimentary rock strata in southwestern Pennsylvania, the following foundation conditions were encountered:

1. A shell of highly fractured rock formed by the valley walls and floor, resulting in a 20- to 30-foot deep weak zone with permeabilities of the order of 10^{-3} to 10^{-4} centimeters per second.

2. Order of magnitude variations in compressibility and permeability between adjacent horizontal rock strata.

3. A variable core-rock contact surface involving weatherable shale and fractured sandstone with existing rock slopes as steep as 0.5 horizontal to one vertical.

4. A series of country bank coal mines penetrating as far as 110 feet into the abutments.

Foundation exploration, analyses and treatment procedures required to accommodate the conditions encountered are described herein.

Location and General Situation

Particulate matter and sulfur dioxide in stack gases at the Bruce Mansfield Power Station, located near Shippingport, Pennsylvania, will be removed by wet scrubbers and treated with a solidifying additive (CALCILOX)*. The resulting sludge will be transported via pipe line (Fig. 1) and deposited in the reservoir formed by Little Blue Run Dam. As the solid portion of the sludge settles out, the supernatant remaining above the solid surface will be maintained at a minimum depth of 10 feet. On this basis, the reservoir has a 30-year capacity. At the final embankment crest elevation (1100 feet), the reservoir surface will be approximately 11,000 feet long and 8,200 feet wide (maximum width).

Elevations in Little Blue Run Valley vary from about 650 feet above sea level at the mouth of the stream to a maximum of 1,380 feet at the southern limit of the watershed. The lowest point along the downstream

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*CALCILOX is a proprietary additive developed by Dravo Corporation.

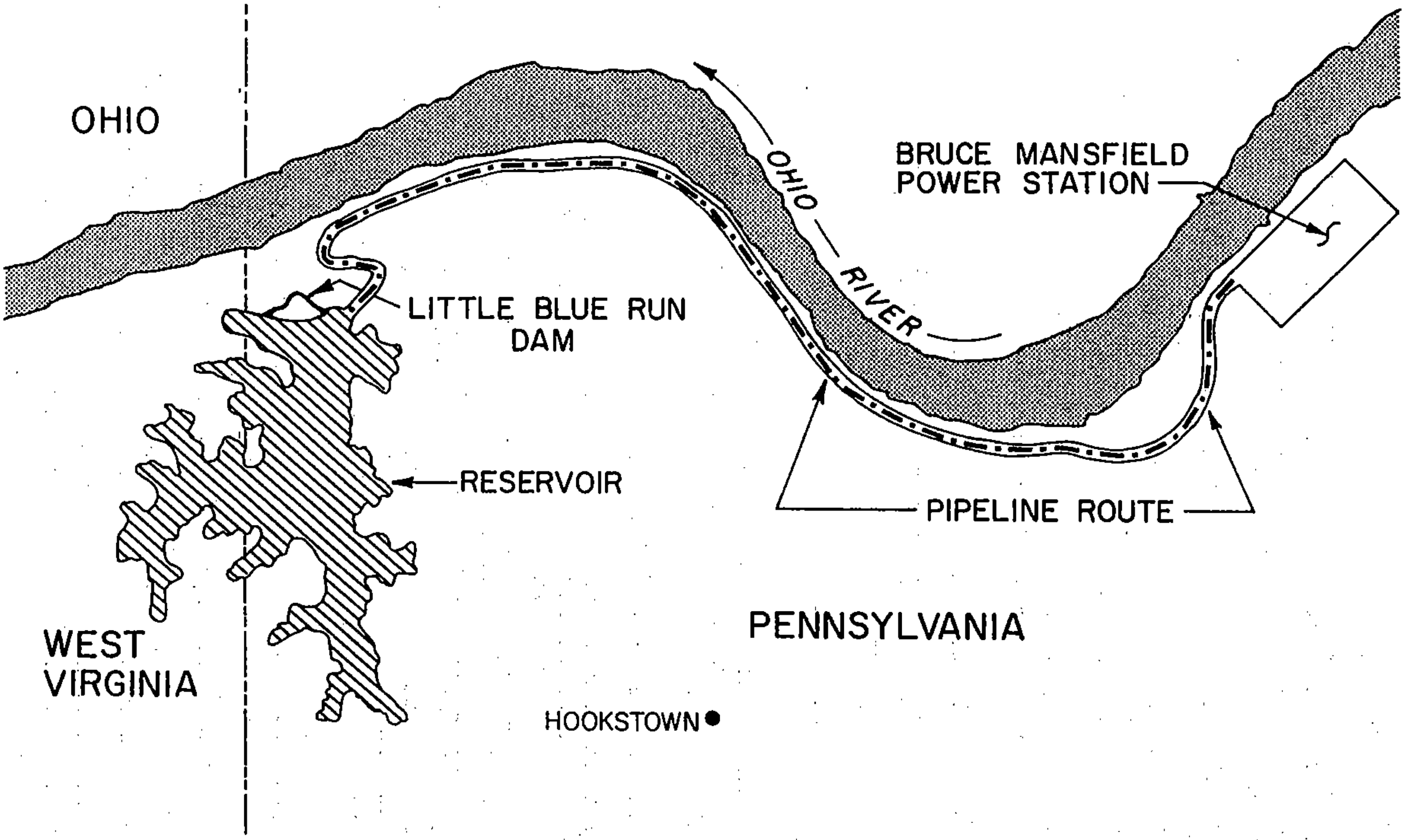


FIGURE 1 - LOCATION MAP

toe of the dam is at elevation 680; the final crest length will be approximately 2,200 feet. The 8,000,000 cubic yard embankment will be one of the highest earth- or rock-fill dams in the eastern United States. Construction began in April 1974, and by January 1976, approximately 4,000,000 cubic yards of material had been placed. Additional details regarding construction of the embankment have been presented elsewhere (5).

The particular location and geometry of the embankment centerline were selected to maximize the storage volume of the reservoir and to minimize the embankment volume (Fig. 2). (The design cross-section is shown in Fig. 3.) The design is based on the comprehensive subsurface investigation described below.

Subsurface Investigation

The subsurface investigation was designed to determine:

1. The types and distribution of soils at the location proposed for the embankment;

2. Strengths and permeabilities of the rock strata at the proposed dam site.

Investigative techniques included the following:

1. In soil: Machine dug test pits and borings;

2. In rock: Core-boring and field permeability tests in bore-holes (packer tests, Ref. 7).

In the preliminary phase of the investigation, borings were spaced approximately 400 feet apart along the initially proposed centerline and along the axis of the valley (Fig. 2). Because relatively impermeable rock strata were encountered above elevation 420, the deepest borings were terminated at approximately this elevation (270 feet below the top of rock in the valley floor).

Data from the 15 preliminary borings and the preliminary geologic study indicated that the proposed site was acceptable. Therefore, the detailed phase of the study was initiated. Additional borings were drilled in this phase, between the preliminary borings and at other locations, (as shown in Fig. 2), resulting in a total of 121 borings. Six borings in the valley walls were inclined at 60 degrees (roughly normal to the valley walls) to better evaluate joint spacing. A plot of joint strike frequency indicates that most jointing is from stress relief, i.e., normal to the valley walls (2,3).

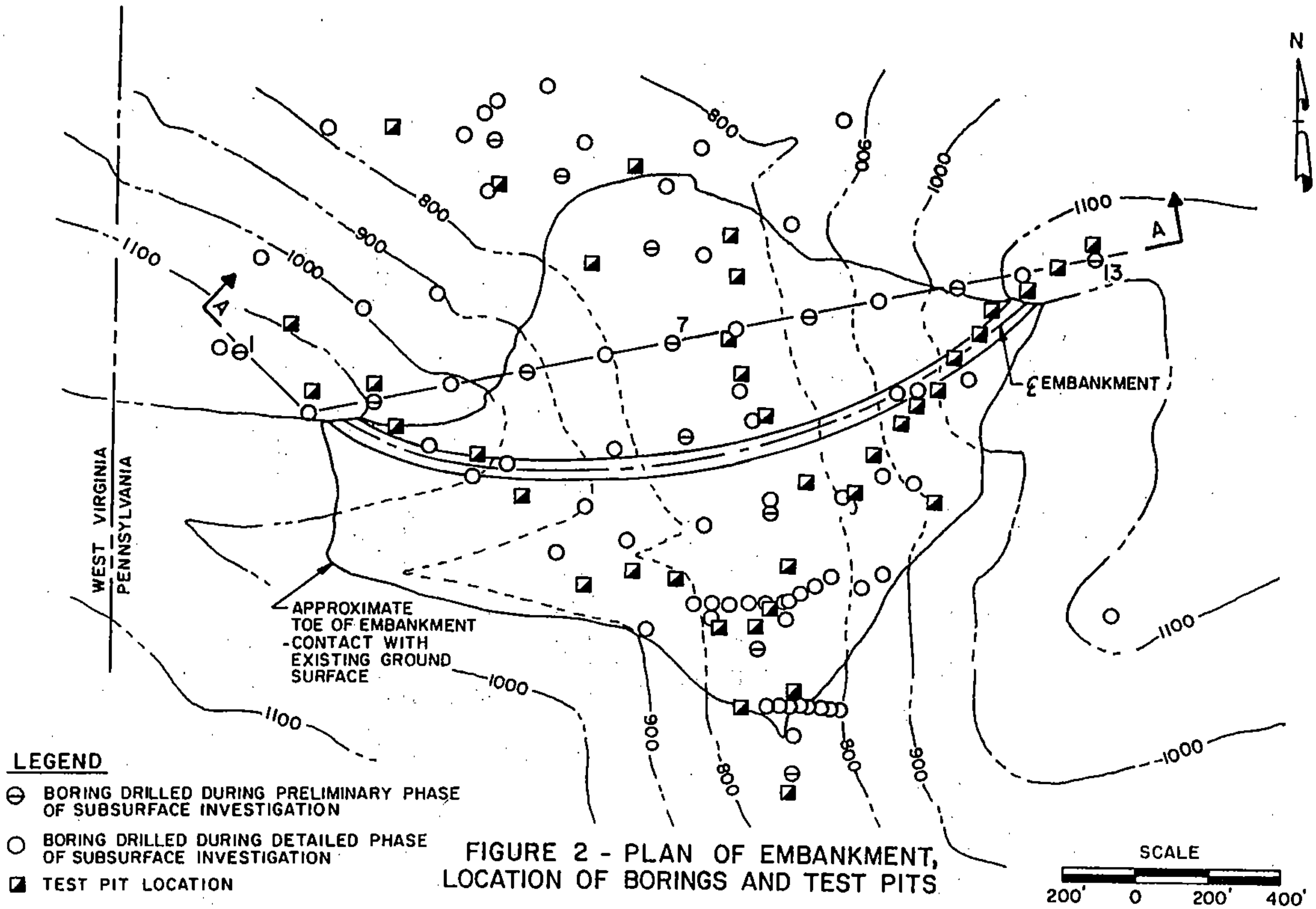


FIGURE 2 - PLAN OF EMBANKMENT, LOCATION OF BORINGS AND TEST PITS

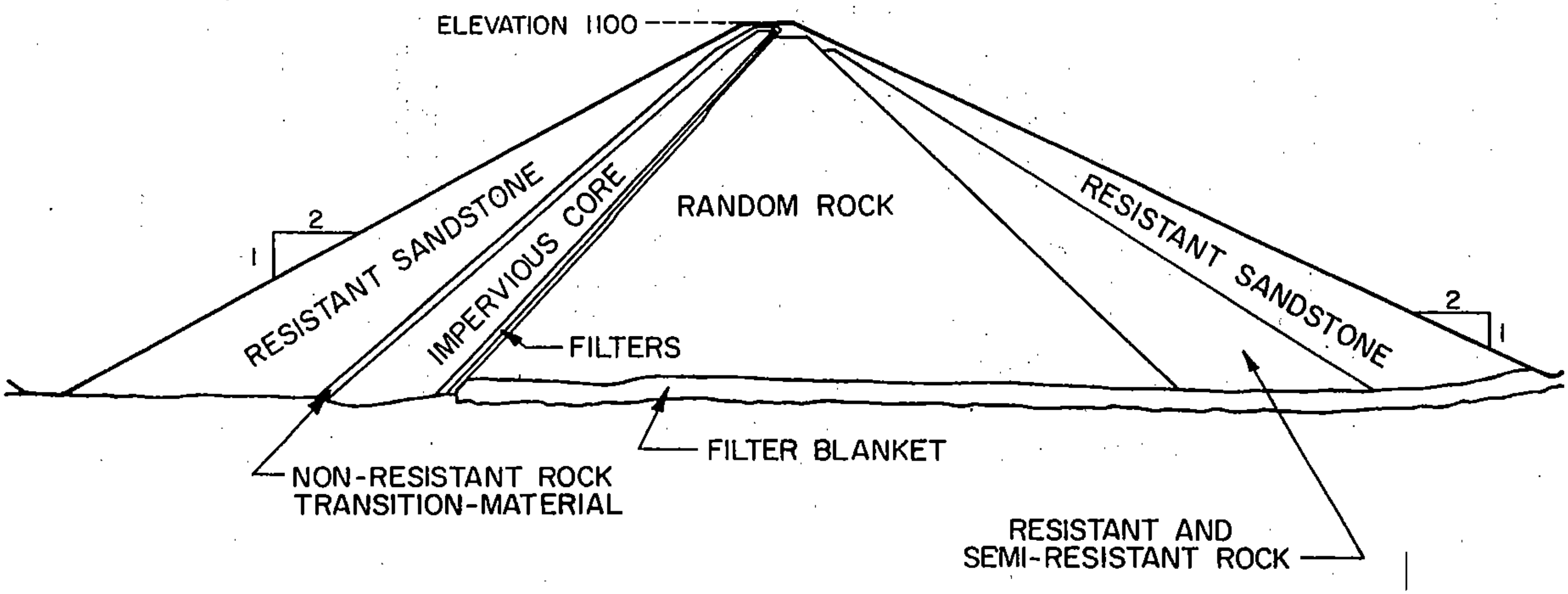


FIGURE 3 - EMBANKMENT CROSS SECTION

Soil depths ranging from 3 to 30 feet were encountered in the borings and test pits, as shown in Fig. 4. A relatively thin veneer of fine-grained colluvial soils (mostly sandy and clayey silt with some silty clay) was found on the valley walls. Thicker deposits of silty sand and gravel with occasional thin lenses of clay were present in the valley bottom. Because of the variability in thickness and composition of the natural soils throughout the embankment area, it was decided to eliminate the differential settlement and stability problems related to foundation soils by founding the dam directly on rock.

Essentially flat-lying sedimentary rocks (sandstone, siltstone, shale, coal, claystone, and occasionally limestone) varying in thickness from less than an inch to 60 feet were encountered (Fig. 4). Rock hardness varied from soft to hard, indicating that order of magnitude variations in modulus and compressive strength could be expected. Of particular interest were the claystone seams shown in Fig. 5. The compressibility and strength of these seams were considered as discussed below.

Packer permeability tests were carried out in 31 of the borings, generally in 5-foot intervals. A water pressure of 1 pound per square inch per foot of depth below the top of rock was used in the subsurface investigation. The packer test data indicate that rock permeability varies with the extent of jointing and brokenness of the rock more than with rock type. Because breakage decreases with depth below the top of rock, permeability also decreases with depth. For analysis purposes the rock strata were divided into the following three permeability zones as shown on Fig. 4:

Zone A: Upper 20 to 30 feet of rock; 10^{-3} to 10^{-4} centimeters per second permeability.

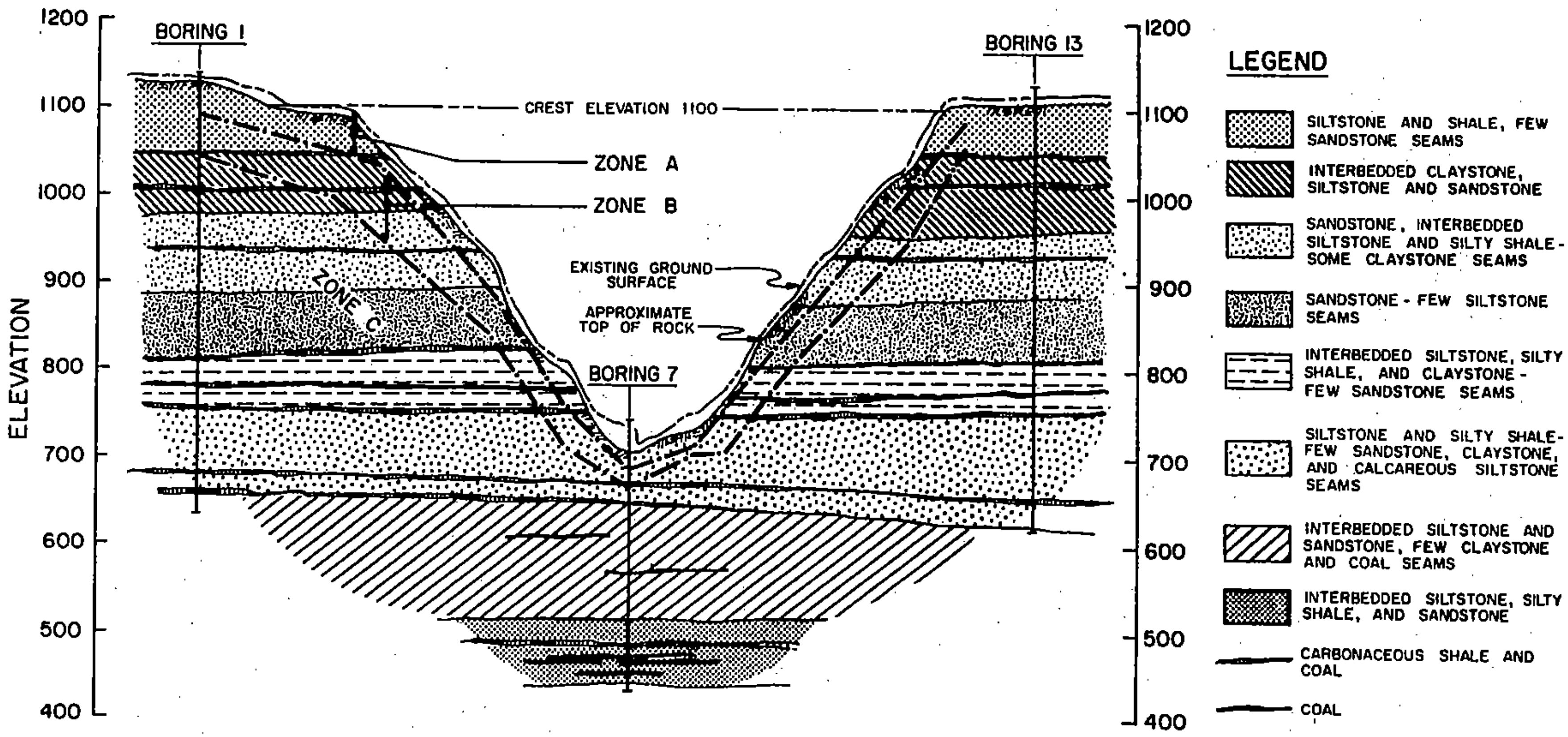
Zone B: 40- to 50-foot thick zone starting at the base of Zone A; 10^{-4} to 10^{-5} centimeters per second permeability.

Zone C: Lower zone starting at the base of Zone B; permeability less than 10^{-5} centimeters per second.

(Frequency of jointing determined from the inclined borings correlates well with these zones.) These permeabilities were considered in the analyses described below.

Settlement, Sliding, and Seepage Analyses

Settlement analyses indicated that foundation



SECTION A - A

GENERALIZED ZONES OF PERMEABILITY

ZONE A = 1×10^{-3} CENTIMETERS/SECOND

ZONE B = 1×10^{-4} CENTIMETERS/SECOND

ZONE C = 1×10^{-5} CENTIMETERS/SECOND OR LESS

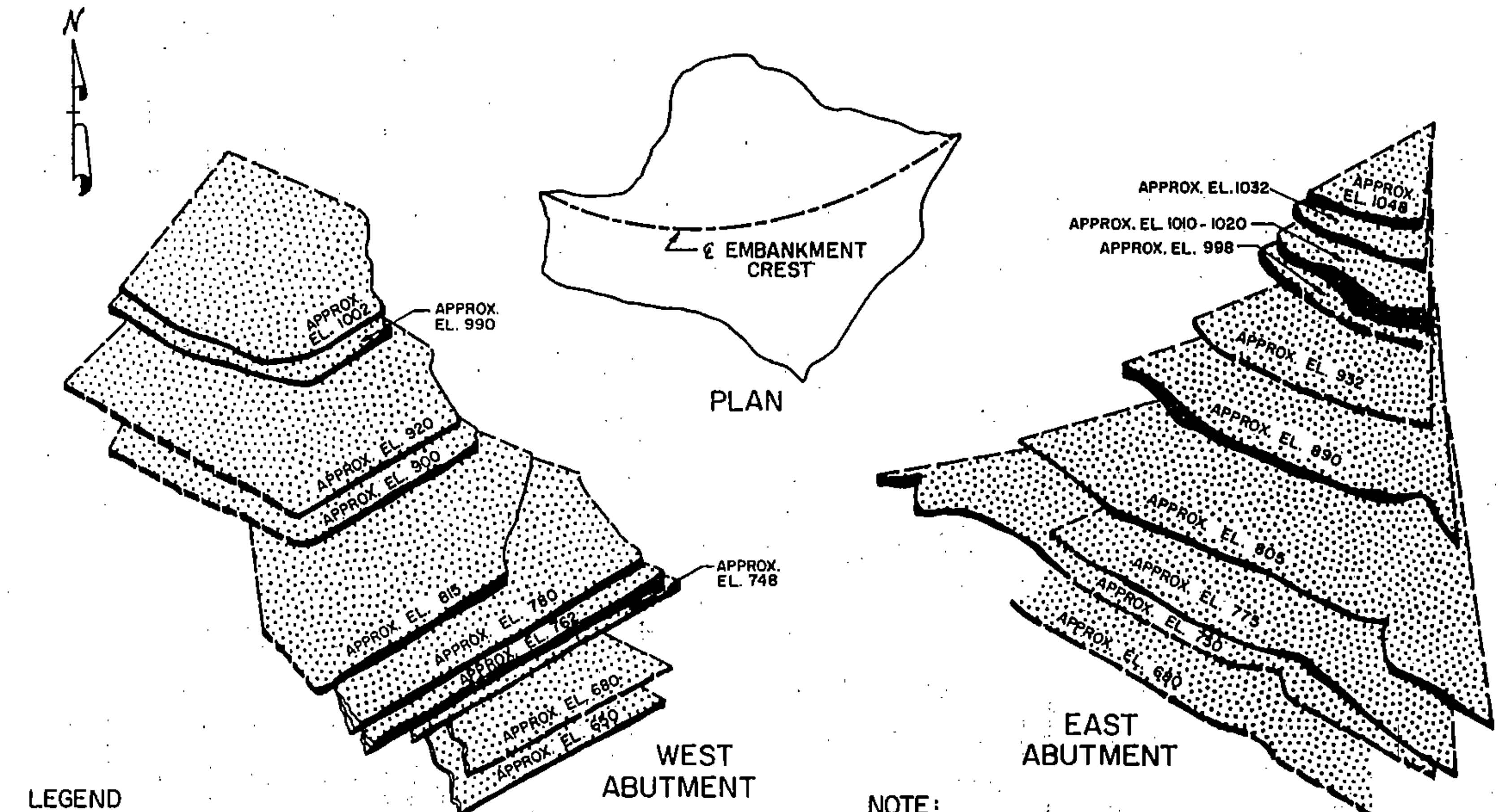
NOTE:

FOR LOCATION OF SECTION AND BORINGS SEE FIGURE 2

HORIZONTAL SCALE

200' 0 200' 400'

FIGURE 4 - GENERALIZED GEOLOGIC CROSS - SECTION



LEGEND

- - - INFERRED STRATA

▬ OUTCROP OF CLAYSTONE SEAMS

NOTE:

THE STRATA AS INDICATED ARE MAPPED FROM CLAYSTONE CORE RECOVERY, INDICATIONS OF SLICKENSIDED SURFACES IN SILTSTONE CORE RECOVERY, AND CLAYSTONE TRACES ASSOCIATED WITH CORE LOSSES.

FIGURE 5 - ISOMETRIC VIEW OF CLAYSTONE LAYERS

settlements would be less than 0.1 percent of the height of the embankment and therefore would be acceptable. Sliding of the embankment was checked on several of the claystone seams shown in Fig. 5. For preliminary analyses, low strength parameters were assumed for these layers ($c = 0$, $\phi = 12$ degrees) (4). On this basis, calculations indicate sliding will not be a problem unless unusually high pore pressures develop. Therefore, 7-inch diameter, 50-foot deep relief wells were installed on 50-foot centers, 100 feet downstream of the core-rock contact below elevation 750 (Fig. 10), to reduce pore pressures, and piezometers were installed at various locations to check on the pore pressures actually developed in the claystone seams. Unless excessive pressures are measured, additional foundation treatment related to the claystone seams is not required.

Seepage analyses involved the following considerations: All seepage from Little Blue Run Reservoir is to be collected and tested for water quality. If the water quality is objectionable, the water collected will be pumped back into the reservoir. To avoid excessive pumping costs the maximum acceptable seepage rate has been set at 1,000 gallons per minute.

For the impervious core at Little Blue Run, field permeability tests (well permeameter method, Ref. 7) gave permeabilities of the order of 10^{-6} centimeters per second. With this permeability seepage through the core will be approximately 35 gallons per minute at final pool level. Because this value is small compared to the acceptable limit, only seepage through the foundation has been considered in subsequent analyses.

Prediction of seepage rates through the foundation at Little Blue Run involves the following complicating factors:

1. Rock permeability depends both on rock integrity (jointing and brokenness) as well as rock porosity. Thus, a flow net based on the assumption of only 3 permeability zones is only approximate.

2. The foundation surface forms a valley with a narrow bottom and moderately steep sides. Thus, a flow net defined by a vertical section along the valley axis will not be representative of conditions encountered at most sections along the centerline of the dam.

Factor 1 could be accounted for by using available computer solutions. However, the available solutions are only two-dimensional, so that factor 2 cannot be accounted for accurately unless a three-dimensional solution is developed. Therefore, it was decided that hand-drawn two-dimensional flow nets would be used to provide an order-of-magnitude estimate of seepage rates, and computer solutions used only if further refinements were required. The effects of factors 1 and 2 above were approximately accounted for as follows:

1. Flows through the individual layers in Zone B were calculated using the permeabilities measured in the packer tests and neglecting the effects of three-dimensional flow.

2. Horizontal flow nets were drawn for several typical seams, each sandwiched between layers of low permeability. Following Twelker (6) revised flows were calculated and used to correct the flows calculated in 1, approximately accounting for the effects of three-dimensional flow.

3. The flow net defined by a vertical section along the valley axis was used with the total flow calculated in 2. to estimate the equivalent uniform permeability for Zone B (3×10^{-4} centimeters per second). This value was used in subsequent calculations.

4. Flow nets were drawn as if permeability were constant, and then modified in zones other than Zone A by increasing the number of head drops in a given zone by the ratio k_1/k_2, k_1 being the permeability of Zone A (10^{-3} centimeters per second) and k_2 being the permeability of the given zone.

Using the procedures outlined above, the seepage rate corresponding to the maximum pool with an untreated foundation was calculated to be 2,000 gallons per minute. Because this rate exceeds the acceptable limit, foundation treatment was necessary. Consideration of various treatment procedures is discussed below.

Consideration of Foundation Treatment Alternatives

The basic purposes of foundation treatment are the following:

1. To insure adequate foundation strength and stiffness.

2. To prevent the development of excessive seepage rates and excessive uplift pressures downstream from the core.

3. To prevent piping of fine-grained material.

4. To insure adequate bond between embankment and foundation.

Adequate bond was made possible by carefully cleaning the foundation surface. Strength and stiffness did not appear to be a problem for most of the foundation; however to insure strength and stiffness beneath the core, blanket grouting was carried out in this critical area.

Assuming blanket grouting to the bottom of Zone A, and a permeability of 10^{-5} centimeters per second for the grouted zone (the upper bound for values reported for grouted rock in the literature, Ref. 1), the estimated rate of seepage for the maximum pool level is 750 gallons per minute. This is less than the acceptable limit, indicating that additional foundation treatment (such as deep curtain grouting) is not absolutely necessary. As added insurance, however, the following procedures were also carried out:

1. Coal seams and large cracks in the bedrock surface below the upstream rock shell (between the core and the upstream toe of the dam) were sealed using clayey soil, slush grout and dental concrete. [The area upstream of the dam will be covered with low permeability sludge ($k = 10^{-6}$ centimeters per second), so that sealing permeable areas between the core and the upstream toe forces seepage from the reservoir to pass through the contained sludge mass, thereby reducing the total seepage which passes beneath the embankment.]

2. A number of the grout holes along the centerline of the core-rock contact zone were drilled to a depth of approximately 80 feet, packer tested to verify the depth of Zone A, and used to place grout in excessively permeable portions of Zone B. These holes were called exploratory grout holes. This procedure essentially guided the choice of the depth of blanket grouting, and to a lesser extent decreased the permeability of Zone B.

In addition to reducing seepage to an acceptable level, the procedures outlined above will tend to prevent the development of excessive uplift pressures downstream of the core. For added insurance a filter blanket is being installed over all portions of the foundation downstream of the core (Fig. 3). This will tend to relieve any uplift pressures developed in this area. Prevention of piping of fine-grained material should be accomplished by the blanket grouting and dental work.

Implementation of Foundation Treatment Procedures

Using dozers and scrapers, the bulk of the soil was removed from beneath all portions of the dam. The core-rock contact area required additional cleaning in the following two stages:

Before grouting, hand cleaning with picks and shovels was used to remove enough of the rock rubble and soil left by the dozers and scrapers to permit subsurface grouting.

After grouting had been completed, a second more careful cleaning of the core trench was performed in small increments. The second cleaning involved primarily additional hand-cleaning (Fig. 6) and the use of light machinery and air or water jets (Fig. 7), depending on rock type-air for shale, water for sandstone. Each increment extended no more than five to seven feet above the adjacent fill surface. Each interval of core trench surface treatment was followed by a corresponding interval of fill placement.

In cleaning a particular interval of core trench, a group of workers, commonly three or four but sometimes as many as a dozen, removed any soil and loose rock left after the first cleaning using picks and shovels. A front-end loader often worked in conjunction with the workers, scraping the rock surface with a "back-dragging" motion of its bucket. The loader was particularly effective in preparing shale slopes but was less effective in sandstone, owing to its harder and more irregular surface. Scraping shale with the front-end loader resulted in a rather smooth surface that required only a minimal amount of sweeping with a straight broom to render it acceptable for placement of clay. Preparation of sandstone was generally more involved than that of shale and varied markedly with the character of the sandstone itself. The sandstone ranged from thin bedded to massive, and jointing varied from hairline cracks to openings twelve or more inches wide. In the lower half of the valley walls, the degree of jointing tended to diminish with distance from the valley centerline. For example, sandstone some distance removed from the centerline could often be prepared merely by hosing with water, while elsewhere extensive hand cleaning was often required. A loader, and less frequently a dozer was employed to remove large, loose blocks, while workers with picks and shovels removed the smaller ones. Presplit blasting techniques were used in one 40-foot interval of

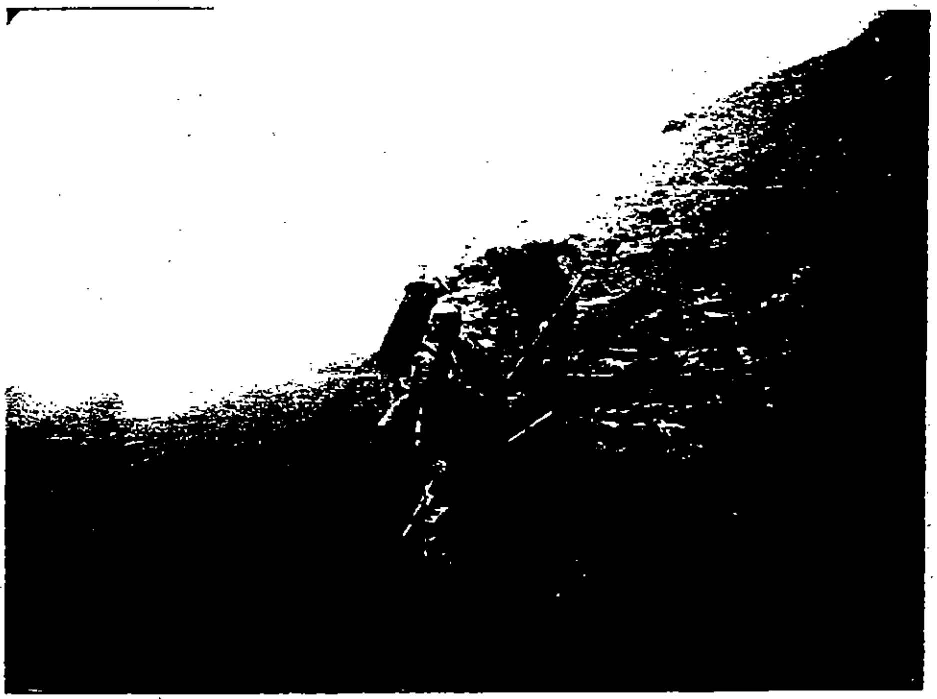


FIGURE 6 - CLEANING CORE AREA WITH BROOMS

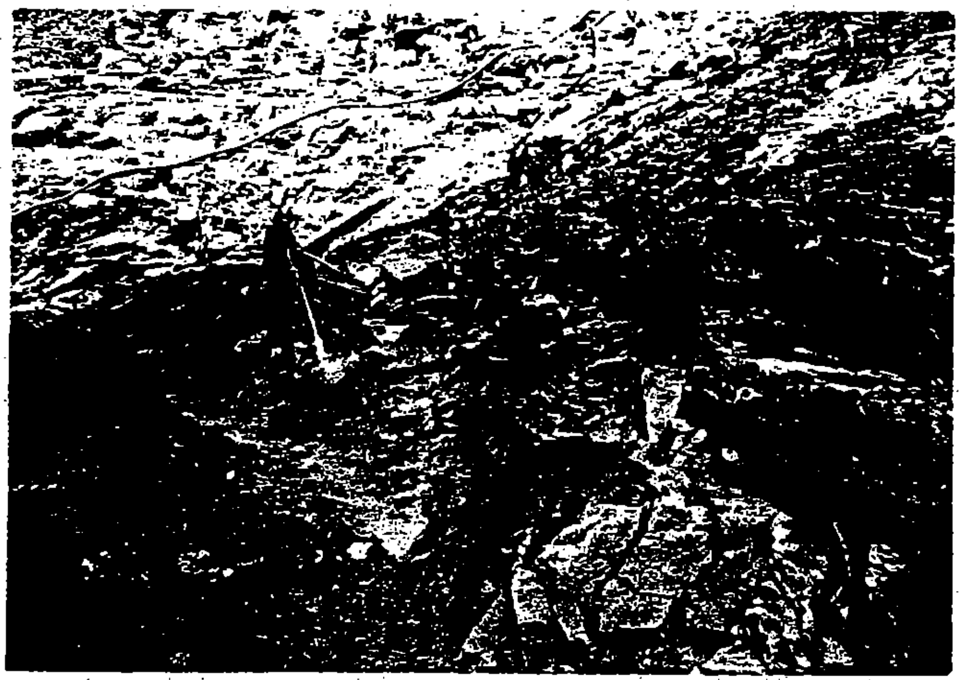


FIGURE 7 - HOSING JOINTS IN CORE AREA

sandstone, reducing the quantities of machine and hand labor required to prepare the rock surface.

Once a sandstone surface was essentially free of loose rock, the rock was hosed off with water, thereby removing any remaining soil and chips of rock (Fig. 7). At the same time, joints were cleaned out to a depth on the order of three times their width. Joints open several inches were cleaned out with picks and shovels and then flushed out with water (Fig. 7). Joint-filling invariably consisted of silt, many times along with broken rock. Joints too narrow to clean with tools were merely flushed out with water.

Water was not used to clean shale or shaley siltstone owing to the rather high tendency for these rocks to break down when wetted. In a few instances, final cleaning of rock surfaces was successfully accomplished by use of a blowpipe operated from an air compressor.

The rock in the valley bottom was primarily weatherable shale and siltstone. To protect this material from weather and vehicular traffic in the core area, an 8-inch (minimum) thick concrete grout cap was placed on the freshly exposed rock from station 9+00 to 13+00 (Figs. 8 and 9). The grout cap was placed before grouting began and served as a platform for subsequent drilling and grouting in the valley bottom. Foundation treatment outside the valley bottom is described below.

"Dental concrete," a four-sack, sand-cement mixture supplied by a local redi-mix distributor and brought to the site in mixer trucks, (Fig. 10), was used to fill large open joints, to seal jointed rock and erodible seams (coal and underclay) (Fig. 10), and to fill overhangs in which compaction of fill would be difficult. (Filling of overhangs was resorted to only when the overhang could not be eliminated by trimming.) The mixture was transported from the mixer truck to the rock face in the truck chute (Fig. 10) or a shallow pan suspended from a mobile crane. Laborers spread the material on the rock surface, smoothing it with the backs of their shovels. Where a rock surface was highly jointed, slush grout, formed by diluting the dental mixture to the consistency of milkshake, was poured over the surface from the carrying pan (Fig. 11). Laborers spread the slush grout with shovels and brooms, developing a coating nominally one-eighth to one-quarter inch thick.

As soon as possible after the second cleaning and the placement of any required dental concrete or slush grout, a layer of highly plastic contact clay was placed against the prepared foundation and compacted with heavy, rubber-tired construction equipment. Prompt placement of clay served to protect the exposed rock,

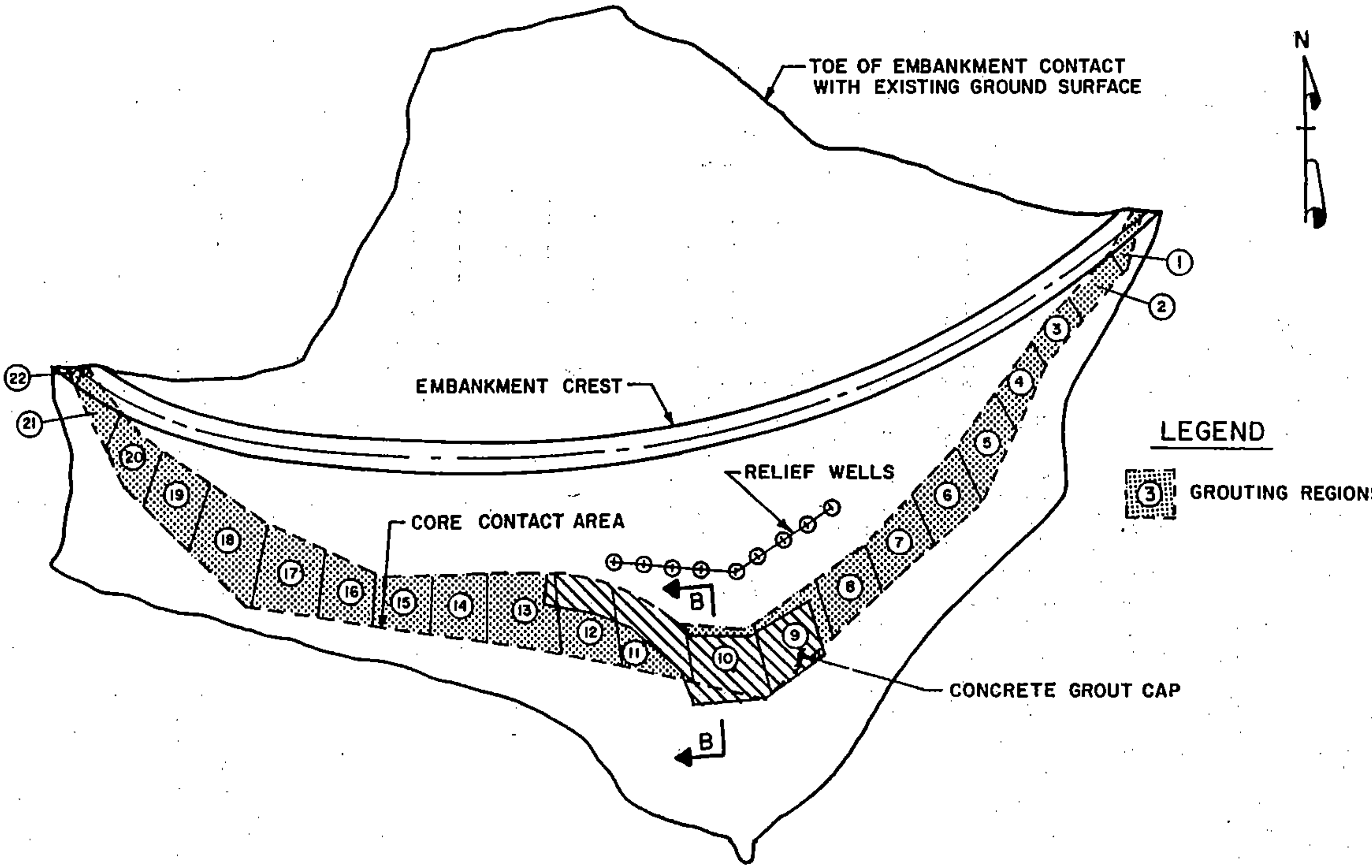


FIGURE 8 - CORE CONTACT AREA, GROUT CAP, RELIEF WELLS AND GROUTING REGIONS

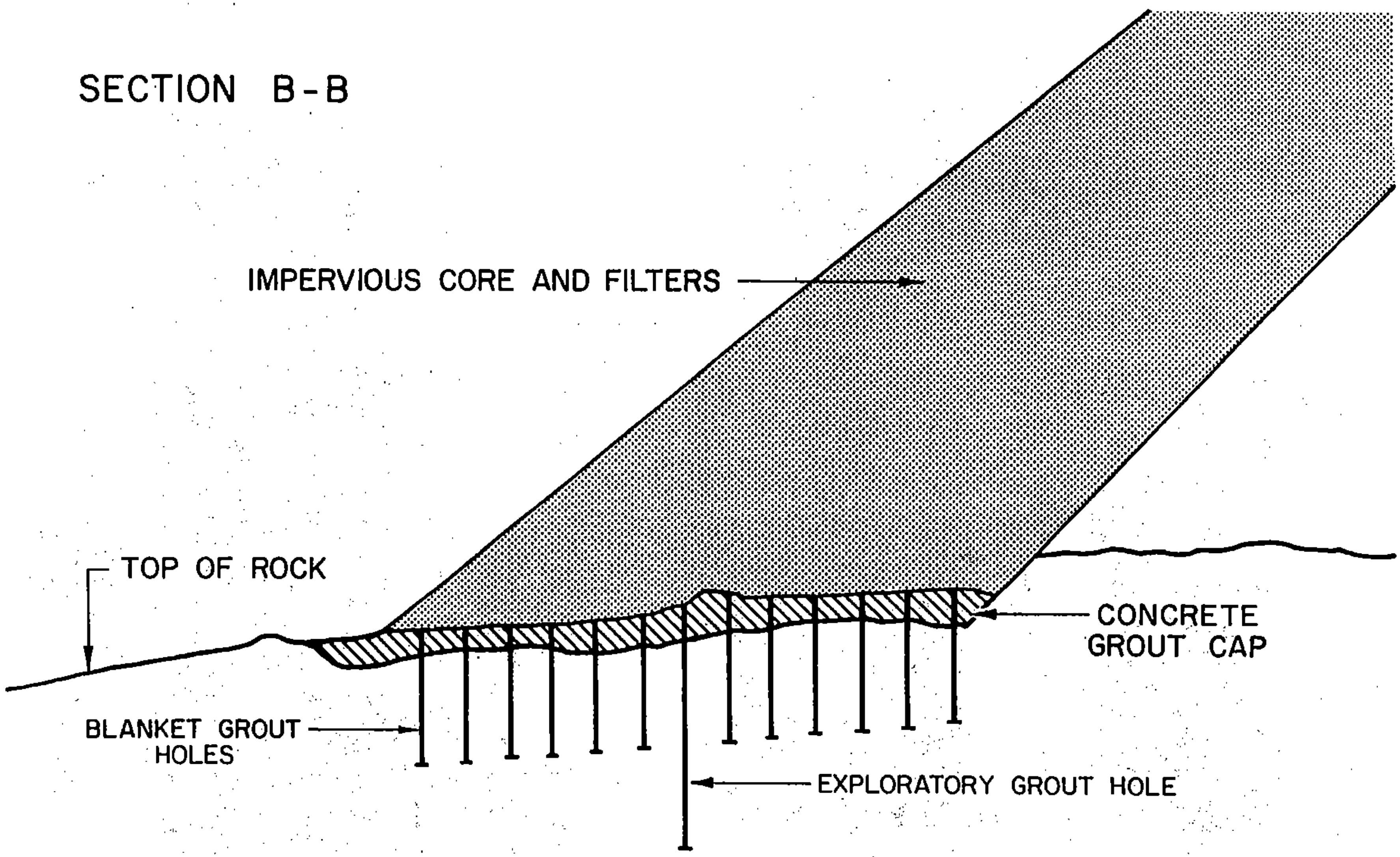


FIGURE 9 - SECTION VIEW OF GROUTING

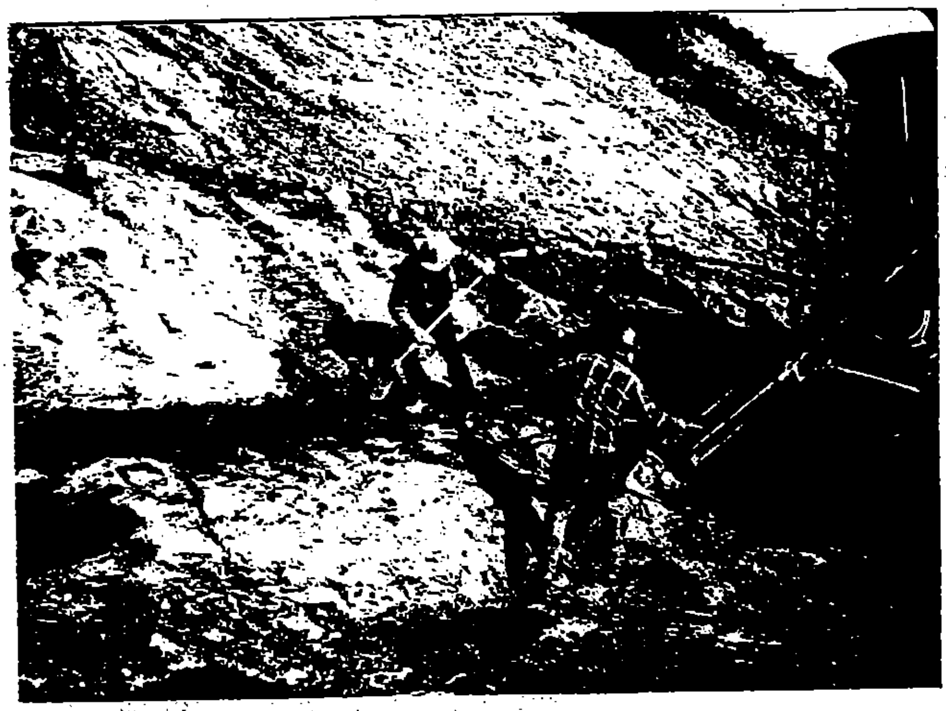


FIGURE 10 - SEALING COAL SEAM WITH DENTAL CONCRETE

FIGURE 11 - SEALING JOINTS WITH SLUSH GROUT

eliminating unnecessary recleaning. Compaction with rubber-tired equipment kneaded the plastic clay into any remaining surface irregularities.

During construction a number of country bank coal mines were discovered in the abutments. Most of these extended only 25 to 30 feet into the valley walls. Mines downstream of the core were filled with filter material to insure good drainage. Small mines upstream of the core were backfilled with concrete and in some cases pressure grouted. The largest mine, located in the right abutment, extended 110 feet into the hillside in the core area. The mine was entered and surveyed visually. Several seven-inch-diameter vertical holes were then drilled from the ground surface into the roof of the mine, the entrances were sealed, and the mine was backfilled by injecting a mixture of fly ash, cement and water through the holes. After backfilling, the entire area was pressure grouted and packer tested. The packer tests indicate that the mine was successfully backfilled and that rock permeability has been reduced to acceptable levels.

Blanket grouting beneath the core (Figs. 10 and 11) followed the pattern shown in Fig. 12. The primary exploratory holes in a given area were drilled first (50-foot centers along the centerline of the core-contact area), and packer tested to check on the local depth of Zone A, ($k > 10^{-4}$ centimeters/second), the zone to be blanket grouted. (In general, the depth of Zone A was 20 to 40 feet.) Because of the critical nature of the core zone, pressures were limited to a maximum of 0.75 pounds per square inch per foot of depth, and survey points were initially monitored to detect any tendency for foundation uplift. The exploratory holes were extended to a depth of approximately 80 feet to be certain no important permeable zones were missed. Pressure grouting was then carried out below Zone A in each exploratory hole. (Zone A was left ungrouted in these holes until after additional testing as described below.) (Pressures were limited as noted above for packer testing.) When the grout take in a given hole exceeded 0.15 bags per lineal foot of grouted hole, the adjacent secondary exploratory holes were also drilled, tested, and grouted (grouting in Zone B only). However, grout takes in the lower portions of the primary exploratory holes were generally less than 0.15 bags per foot, so that few secondary and tertiary exploratory holes were drilled.

After the lower portion of a given exploratory hole had been grouted, the surrounding primary blanket holes were drilled (to the bottom of Zone A) and grouted. If the grout take in a given hole exceeded 0.15 bags per lineal foot of grouted hole, the surrounding

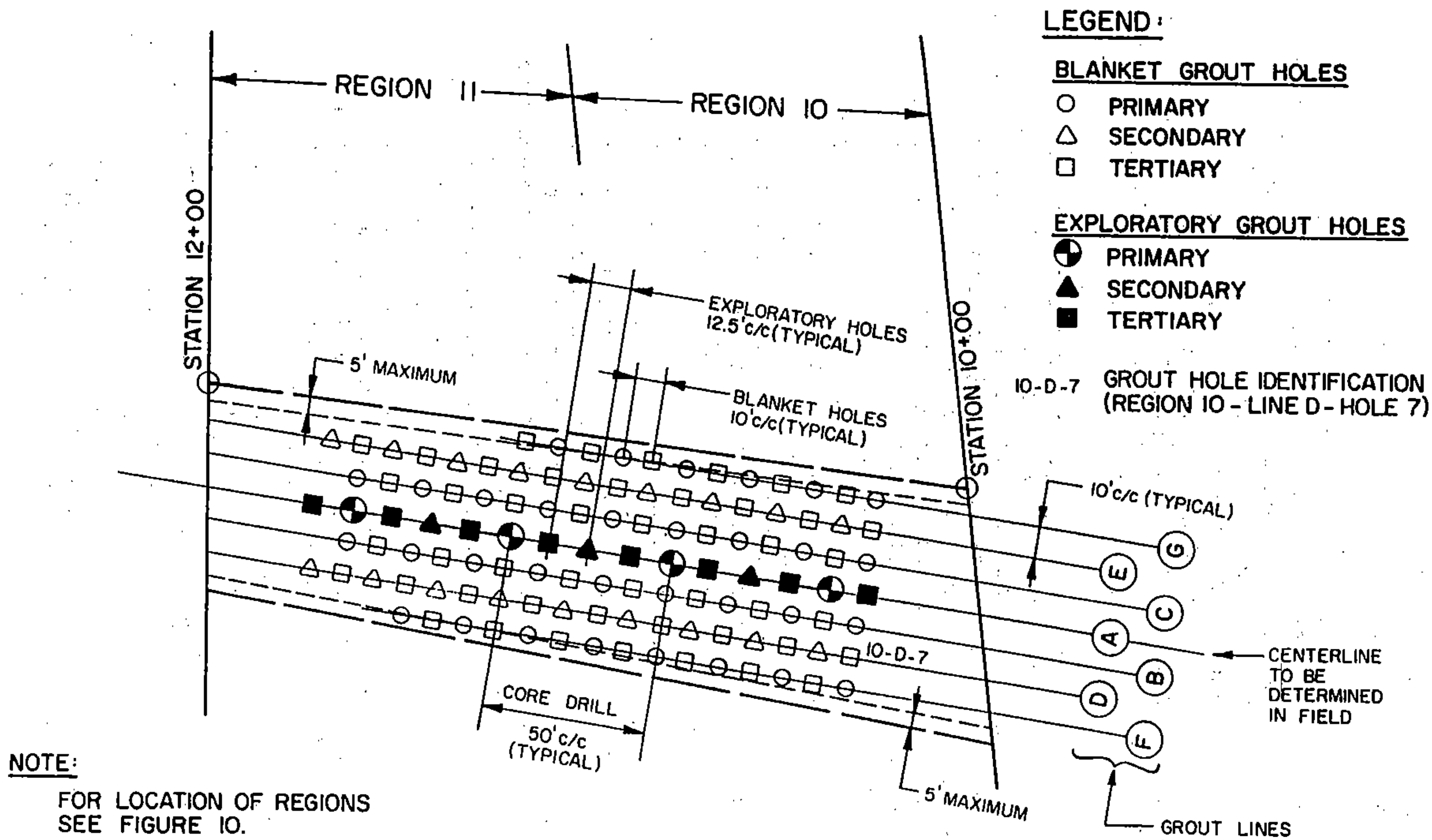


FIGURE 12 - TYPICAL GROUT HOLE PATTERN

secondary holes were also drilled and grouted, etc. Once acceptable takes were observed around a given exploratory hole, the hole was packer tested again. (In some cases grout from other holes had entered the exploratory hole and it was necessary to redrill the hole using a slightly larger bit.) The packer tests confirmed the adequacy of the blanket grouting as described subsequently. After a hole had been retested, the upper portion of the hole was pressure grouted as a final step.

Evaluation of Grouting

As outlined above, after completion of grouting around a given exploratory hole, the hole was retested to evaluate the adequacy of the blanket grouting. The results of packer tests carried out before and after grouting are presented in Figs. 13-15. In general, permeabilities of the order of 10^{-3} to 10^{-4} centimeters per second were reduced to 10^{-5} to 10^{-6} centimeters per second. For a more detailed evaluation, a line corresponding to $k = 10^{-5}$ centimeters per second has been drawn on each plot, defining zones of $k < 10^{-5}$ centimeters per second and $k > 10^{-5}$ centimeters per second. (A permeability of 10^{-5} centimeters per second was determined to be suitable for the grouted zones.) The total depth of rock having $k < 10^{-5}$ centimeters per second exceeds the total depth having $k > 10^{-5}$ centimeters per second. Thus, the permeability assumption for grouted rock used in the seepage analyses ($k = 10^{-5}$ centimeters per second) is conservative, and the grouting program has accomplished its intended goal. A more critical assessment of the grouting program in conjunction with the other foundation treatment procedures will be made as the reservoir is filled and final seepage rates and pore pressure levels are measured.

Summary and Conclusions

To provide a reservoir for containment and solidification of sludge collected by a wet scrubber system, a 420-foot high, sloping-core, rock-fill dam is being constructed in southwestern Pennsylvania. A comprehensive subsurface investigation involving 121 borings, 35 test pits and a large number of bore-hole (packer) permeability tests was carried out to determine the soil and rock characteristics involved. Due to the extent of jointing and rock breakage, permeability decreased with depth below the top of rock as well as with rock type. Settlement and sliding analyses indicated that the foundation would be acceptable, provided unusual pore pressures did not develop. Therefore, piezometers and relief wells were installed. To insure strength and stiffness beneath the core,

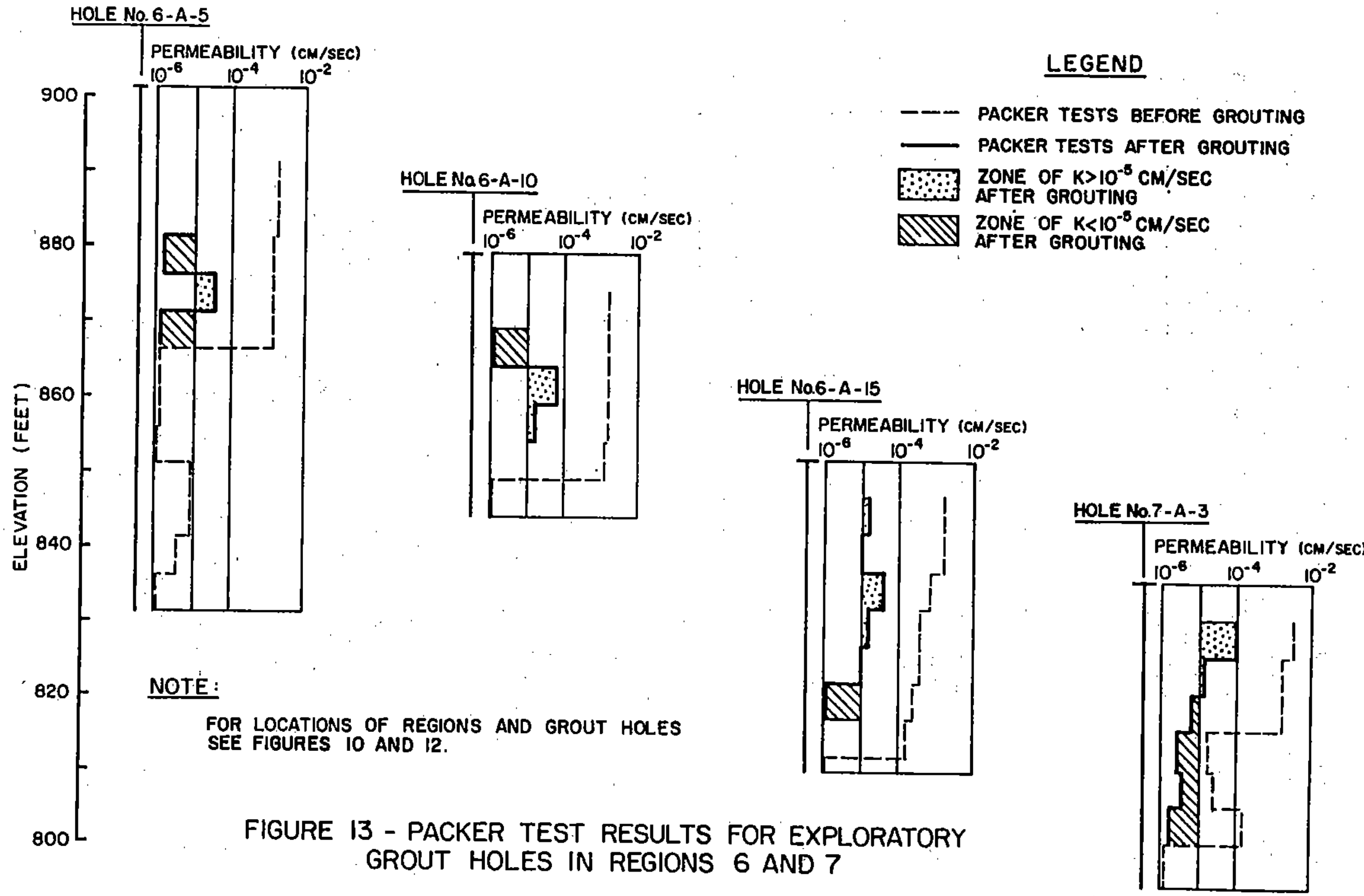


FIGURE 13 - PACKER TEST RESULTS FOR EXPLORATORY GROUT HOLES IN REGIONS 6 AND 7

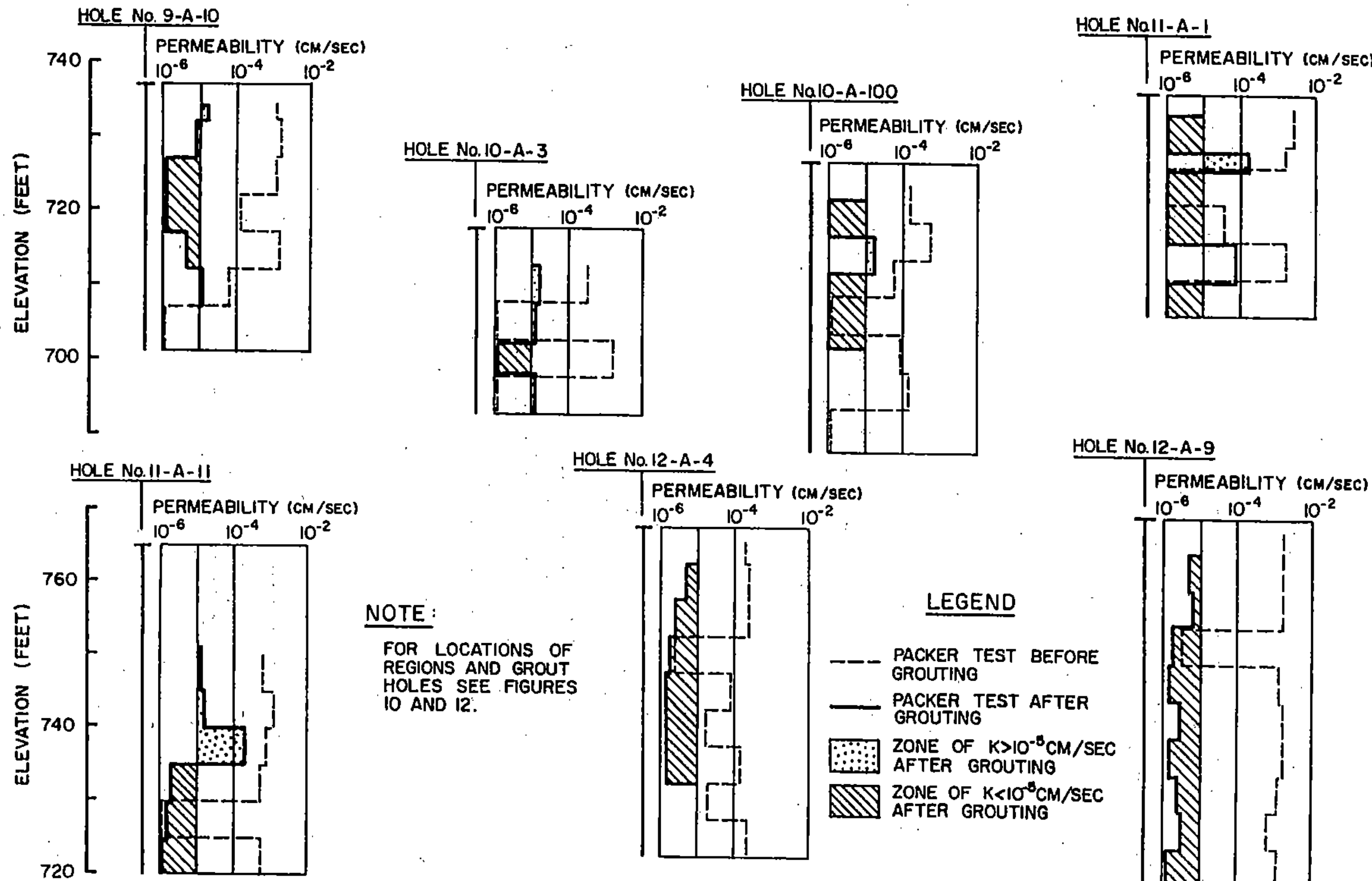


FIGURE 14 - PACKER TEST RESULTS FOR EXPLORATORY GROUT HOLES IN REGIONS 9 THRU 12

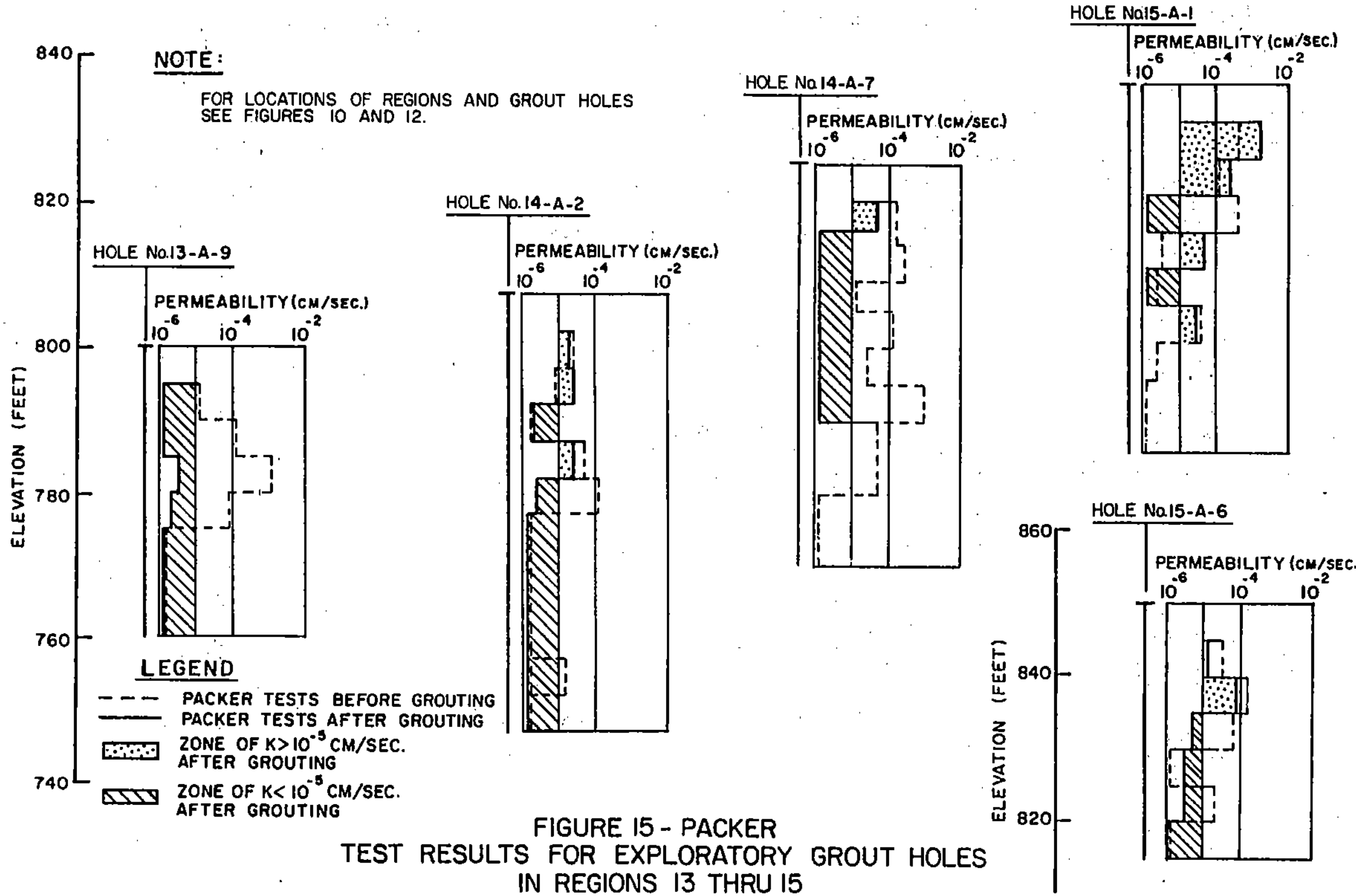


FIGURE 15 - PACKER TEST RESULTS FOR EXPLORATORY GROUT HOLES IN REGIONS 13 THRU 15

blanket grouting was conducted in this critical zone. Special seepage analyses were carried out accounting for variations in rock permeability and the three dimensional nature of the flow. The analyses indicated that the blanket grouting would reduce seepage to an acceptable level.

Soil was removed from beneath all portions of the dam, and the bedrock surfaces below the core and below the upstream rock shell were subjected to additional cleaning and sealing. A number of country bank coal mines in the abutments were also sealed. Additional exploratory holes were drilled and packer tested as part of the grouting program. The results of these tests and observation of grout takes guided the final selection of grouting depths and hole spacings. A preliminary evaluation of the grouting program was developed using packer tests carried out after completion of the work in a given area. In general, permeabilities exceeding 10^{-4} centimeters per second were reduced below 10^{-5} centimeters per second, the value assumed in the analysis of grouted zones. Thus, the foundation treatment implemented appears to be adequate. Pore pressures and seepage rates will be monitored during and after construction, as a further check on the adequacy of the foundation treatment.

Acknowledgements

Little Blue Run Dam is part of the Scrubber Sludge Disposal System for the Bruce Mansfield Power Station. The overall system was designed by Gibbs and Hill, Inc., and will be owned and operated by the Pennsylvania Power Company for the CAPCO group. Mr. Thomas M. Leps served as a consultant on the design of the dam.

APPENDIX I - REFERENCES

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