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AMERICAN SOCIETY OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors should indicate the technical division to which the paper is referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles must have a length not exceeding 50 characters and spaces.

2. A summary of approximately 50 words must accompany the paper, a 300-word synopsis must precede it, and a set of conclusions must end it.

3. The manuscript (an original ribbon copy and two duplicate copies) should be double-spaced on one side of 8½-inch by 11-inch paper. Three copies of all illustrations, tables, etc., must be included.

4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

5. Mathematics are recomposed from the copy that is submitted. Because of this, it is necessary that letters be drawn carefully, and that special symbols be properly identified. The letter symbols used should be defined where they first appear, in the illustrations or in the text, and arranged alphabetically in an Appendix.

6. Tables should be typed (an original ribbon copy and two duplicate copies) on one side of 8½-inch by 11-inch paper. Specific illustrations and explanation must be made in the text for each table.

7. Illustrations must be drawn in black ink on one side of 8½-inch by 11-inch paper. Because illustrations will be reproduced with a width of between 3-inches and 4½-inches, the lettering must be large enough to be legible at this width. Photographs should be submitted as glossy prints. Explanations and descriptions must be made within the text for each illustration.

8. The desirable average length of a paper is about 12,000 words and the absolute maximum is 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

9. Technical papers intended for publication must be written in the third person.

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Journal of the SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONTENTS

March, 1963

Papers

(over)

Note.—Part 2 of this Journal is the 1963-14 Newsletter of the Soil Mechanics Division.

The three preceding issues of this Journal are dated October 1962, December 1962, and February 1963.

DISCUSSION

KEY WORDS: piles; load; bearing capacity; testing; steel; soil mechanics; foundations

ABSTRACT: Pile load tests on instrumented piles reveal that load transfer to surrounding soil is a significant factor in the capacity of point bearing piles. The piles tested were 14BP89 and 14BP117 steel H-piles driven approximately 40 ft through sand and gravel. The piles were point bearing in shale. Such piles are relatively short and stiff, but the tests revealed that they were of sufficient flexibility to transfer approximately one-third of the applied load to the surrounding soil. A theoretical calculation of load transfer is presented and is in good agreement with the observed results. The mechanism of pile failure, and significance of load transfer in the failure mechanism, is described and examined in terms of significance in design.

REFERENCE: "Load Transfer in End-Bearing Steel H-Piles," by E. D'Appolonia and J. P. Romualdi, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM2, Proc. Paper 3450, March, 1963, pp. 1-25.

KEY WORDS: compaction; soil; shear; effective stress; density; water

ABSTRACT: The application of a compaction pressure to an unsaturated soil results in the development of shearing stresses at the points of contact between soil particles until the contacts fail. The particles then slide over one another with an increase in density. The compaction characteristics of soil are thus controlled by the shearing resistance of these contacts. The shearing strength is a function of the effective normal stress. When the compaction pressure is applied to a dry soil, the effective stress is approximately equal to the total stress, because the χ-coefficient is small. The addition of water increases the degree of saturation and increases the pore pressures, thus weakening the soil. Hence, the soil deforms more under stress, and higher densities result. The density must increase during compaction until the total stress, the pore-air pressure, and the pore-water pressure combine to give the required effective stresses. The theory is qualitative because little is known of the dynamic stress-strain properties of unsaturated soils.

REFERENCE: "Effective Stress Theory of Soil Compaction," by Roy E. Olson, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM2, Proc. Paper 3457, March, 1963, pp. 27-45.

KEY WORDS: dams; earthfill; clay; compaction; settlement; plasticity; strains; testing

ABSTRACT: An approximate theory is formulated to calculate the critical tensile strains in an earth dam that result from differential settlements along the axis of the dam. Laboratory apparatus and procedures are developed to estimate the limiting tensile strain at which compacted clay will crack. Comparisons between predicted and observed behavior of five dams indicate that the theory and laboratory tests can be used to predict cracking potential with an accuracy that is satisfactory for practical purposes. The ratio of tensile strain at cracking to compressive strain at failure is a small fraction (approximately 0.01 to 0.1) that shows no evidence of any consistent pattern. Accordingly, it is hazardous to assess the flexibility of compacted earth dams on the basis of stress-strain relations obtained from compression tests. The effects of molding water content and compactive effort on the flexibility of clay was investigated. No consistent relationship between flexibility and plasticity characteristics of the clay was found.

REFERENCE: "Flexibility of Clay and Cracking of Earth Dams," by G. A. Leonards and J. Narain, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM2, Proc. Paper 3460, March, 1963, pp. 47-98.

E = Young's modulus;

$F(y)$ = function of y;

$F_n(y)$ = n^{th} derivative of F(y) with respect to y;

$f(x)$ = function of x, representing crest-settlement curve of dam;

$f_n(x)$ = n^{th} derivative of f(x) with respect to x;

G = modulus of rigidity (shear modulus);

H = height of dam;

R = radius of curvature;

u = displacement in x direction;

v = displacement in y direction;

$2L$ = length of dam;

α = $\frac{n\pi}{L}$, in which n represents 1, 2, 3, and so forth;

β = $\frac{\sinh \alpha H + \alpha H \cosh \alpha H}{\alpha H \sinh \alpha H}$;

γ_{xy} = shearing strain on x-y plane;

ϵ_x, ϵ_y = normal strains in x and y directions;

μ = Poisson's ratio;

σ_x, σ_y = normal stresses on planes perpendicular to x and y axes;

τ_{xy} = shear stress on x-y plane; and

ϕ = Airy stress function.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

OAHE DAM: GEOLOGY, EMBANKMENT, AND CUT SLOPES[a]

By Donald K. Knight,[1] F. ASCE

FOREWORD

This symposium is composed of three papers originally presented before the Society's first Water Resources Conference in May 1962 in Omaha, Nebr. The paper "An Unusual Spillway," by H. S. Kidd, E. R. Bloomquist, and C. E. Johnson is currently under review by the Soil Mechanics and Foundations Division. The paper "Influence of Shale on Oahe Power Structures Design," by E. A. Johns, R. G. Burnett, and C. L. Craig was published as Proc. Paper 3423 in the February 1963 Journal of the Soil Mechanics and Foundations Division.

SYNOPSIS

General data on the geology of the project are reviewed and the major soils problems of cut slope and embankment stability are examined. The principal

Note.—Discussion open until August 1, 1963. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 89, No. SM2, March, 1963.

a Presented at the May 1962 ASCE Conference in Omaha, Nebr.

1 Prof. Engr. and Chf., Embankment, Foundation and Speical Studies Sect., Foundation and Materials Br., Engrg. Div., Omaha Dist Office, U. S. Corps of Engrs., Omaha, Nebr.

features of the embankment design are noted. Experiences of slides during construction are related with reasonably complete details given regarding the left abutment of major slide. Stability studies are generally presented and vital design shear strengths are developed using a semi-empirical method. Conclusions are drawn that may assist in the future design of dams to be built under similar conditions.

INTRODUCTION

It is interesting to note that the sedimentary bedrock formations of the upper Missouri River Basin are generally classed as soft rocks. In 1935, prominent engineers and geologists, such as William Gerig, Warren J. Mead, and Frank E. Fahlquist, F. ASCE, [2] described weaknesses of these bedrocks and even expressed doubts on their adequacy by concluding that dams of approximately 200 ft heights could not be safely built. The occurrence of the Fort Peck slide in 1938 with the dam nearing completion added further doubts. Fortunately, investigation and reports[3,4] that followed explained the slide failure at Fort Peck and provided the profession with much invaluable data. In more recent years, during the Oahe Project design, many members of the Board of Consultants, hired to review and comment on the work of the United States Army Corps of Engineers, brought out the concern for the relatively soft bedrock at the Oahe Project. Joel D. Justin expressed concern over the use of shale as fill material in the upstream and downstream berm slopes of the embankment. Arthur Casagrande, F. ASCE, attended some fifteen meetings of the Oahe Board of Consultants from 1945 to 1957 before expressing complete optimism in regard to the successful design of this project. Because the Bearpaw shale at Fort Peck has been classified as a "twin brother" to the Pierre shale bedrock at the Oahe Dam, the engineering data and experiences from that project have been studied thoroughly in the design of this project.

Although many soils and foundation features of the Oahe Dam will be mentioned herein, only the design of the embankment and cut slope stability will be examined in detail.

The Oahe Dam is located a few miles north of Pierre, S. Dak. A general plan of the project is shown in Fig. 1. The reservoir is forming to the north or top side of this plan. Looking downstream, the power works are located in the left abutment, with the outlet works in the right abutment. The spillway is approximately a mile landward of the valley right abutment. Although the embankment or earthwork on the project was essentially finished, the

2 "The Missouri River '308' Report," published as House Document No. 238, 73rd Congress, 2nd Session, 1947, pp. 974-1025.

3 "Report on the Slide of a Portion of the Upstream Face of the Fort Peck Dam," by the War Dept., Corps of Engrs., U. S. Army, July, 1939.

4 "Fort Peck Slide," by T. A. Middlebrooks, Transactions, ASCE, Vol. 107, 1942, p. 723.

total project was approximately 88% complete as of Jan. 31, 1963. The contract for installation of the relief well system, which is to insure underseepage control at the toe of the dam, is 94% complete as of Jan. 31, 1963. As of Feb. 1963, the Oahe Reservoir storage was approximately 9,544,000 acre-ft, or 40% of the gross storage, and the pool is near El. 1567, or 27 ft above minimum operating pool level. The first four Power units are now operating on the line and the remaining three units will be producing by the end of 1963.

GEOLOGY

The Oahe area is near the eastern edge of the Missouri Plateau region of the Great Plains physiographic province. Geological studies have shown that

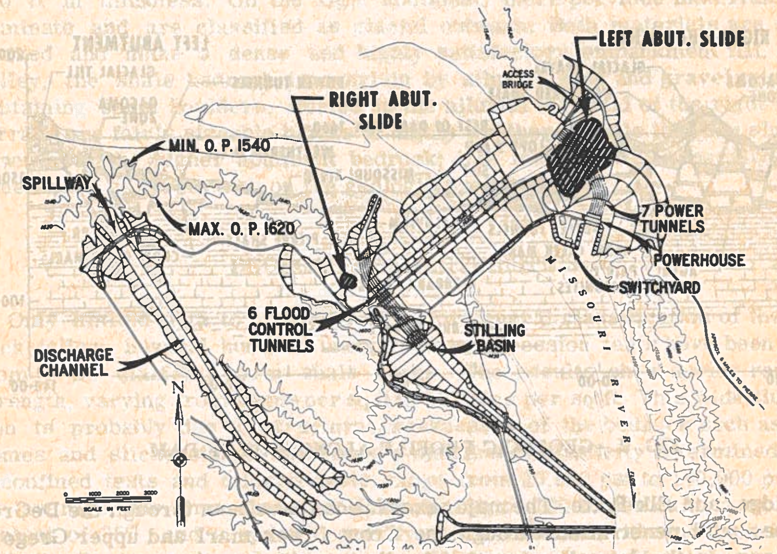

FIG. 1.—GENERAL PLAN

the area was drained by eastward-flowing streams prior to the Illinoian glacial stage. The preglacial streams are believed to have run eastward from Pierre, approximately 100 miles to the valley now occupied by the James River. The first glacier dammed the waters of these eastward-flowing streams, forming lakes that spilled over low divides and rapidly cut connecting channels to form the Missouri River trench. This trench reached a maximum depth below the present river level of approximately 75 ft during the Sangamon-Interglacial stage. This shallow depth of scour is an exception to the general rule for all other main stem dams on the Missouri River, where the maximum depth of the buried valley is approximately 150 ft. In the writer's opinion, the shallower scour at Oahe is due to the harder bed of Crow Creek marl (see Fig. 2). In past geological ages, this relatively weak, Pierre shale bedrock has been subjected to repeated loadings from ice and overburden. The entrenchment of the river provided free boundaries for slides and slumps to develop. The river valley bluffs, now remaining as nature's record of the turbulent geologic history, had to be utilized for founding the various structural features of the project.

The only bedrock formation exposed at the damsite is the Pierre shale formation, which was formed late in the Cretaceous geologic period. The Pierre shale is derived from clays and silts, which are pressure indurated by the weight of the overlying sediments and several thousand feet of glacial ice into thinly bedded, dark gray to black shales and marls. Many layers of bentonite occur throughout the formation, with occasional zones containing iron-manganese concretions. In the Missouri valley region, the Pierre formation has been subdivided into eight members, which are in ascending order: Sharon Springs, Gregory, Crow Creek, DeGrey, Verendrye, Virgin Creek,

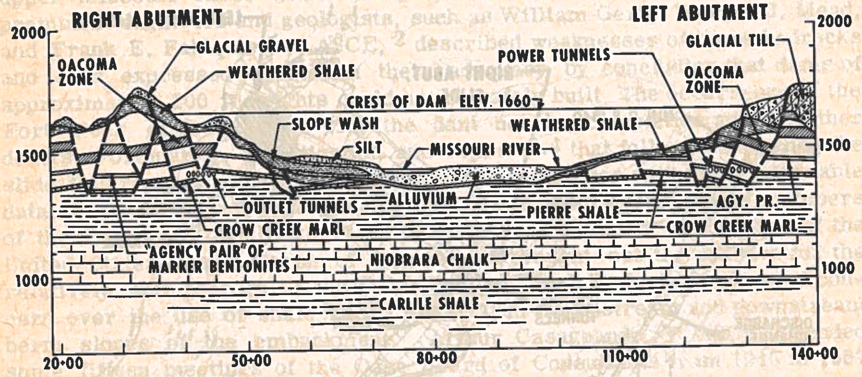

FIG. 2.—GEOLOGIC PROFILE ALONG AXIS OF DAM

Mobridge, and Elk Butte. The major excavations have been through the DeGrey member with penetration through the Crow Creek marl and upper Gregory members. In the lowest excavation for the powerhouse, there remains 276 ft of shale above the Niobrara chalk. Faults caused by regional disturbances are believed to be few or widely spaced because all deep borings at the dam site showed the Pierre-Niobrara contact to be within a few feet of the same elevation. Along the river bluffs and in other bedrock exposures, any surface effect of regional deformation is obscured by local slumping of Pierre shale. The many closely spaced slump faults in the shale near the surface show vertical displacements varying from a few to as many as 80 ft and have resulted in the typical "sugar-loaf" or "hummocky" topography. This is plainly evident along the right abutment immediately upstream of the dam and quite generally visible both upstream and downstream of the dam on the left abutment. The most recent slumping, which is still quite active, is approximately a mile upstream from the dam on the left abutment.

Geologic studies have revealed several points that substantiate the theory that the fault system shown on Fig. 2 is not caused by deep-seated, crustal

disturbance, but is the result of gravity faulting or slumps along slickensided zones developed much earlier by at least two cycles of compression and rebound from glaciation. Such slumping probably occurred at a time when the maximum depth of scour existed as the Missouri trench. The facts, first, that the drill hole logs show a concentration of slickensides in a 50-ft zone immediately above the deepest scour level, and second, that the faults show less vertical displacement with depth as they pass through this zone, are evidence substantiating this theory.

Even firm shale is jointed and contains numerous horizontal parting planes along the bentonite seams. In general, the Pierre formation is relatively impervious to water and seepage into deeper excavations has not been a problem. In highly weathered zones, the accumulation of seepage waters over a period of many years into pockets or joints in the formation was encountered.

Overlying the shale abutments are glacial deposits and minor amounts of loess. On the left abutment, the overburden material is glacial till to 200 ft in thickness. On the right abutment, more pervious materials predominate and are classified as glacial outwash. Both materials are easily worked and make a dense and highly satisfactory embankment fill. In the valley, the shale bedrock is overlain by alluvial sand and gravel mixtures containing some boulders. Buried in the alluvial material of the right valley terrace are fairly sizeable blocks of highly weathered shale that has slumped down from the higher abutment bedrock; this material lies generally in the zone shown as slope wash on the geologic profile of Fig. 2.

BEDROCK CHARACTERISTICS

Only limited data to provide a basis for general classification of the bedrock follow. Several hundred unconfined compression tests have been made from 6-in. diameter firm shale cores. The results show a wide range of strength, varying from 5 tons per sq ft to 182 tons per sq ft. This wide fluctuation is probably due to structural weaknesses of the bedrock such as joint planes and slickensides. The Young's modulus of elasticity determined from unconfined tests and triaxial tests range from 20,000 psi to 140,000 psi. On the average, the firm shale has a dry density of 100 lb per cu ft, a moisture content of 25%, and a specific gravity of 2.7. Load and rebound experiences with the Pierre shale and other similar Missouri valley bedrocks have led to the adoption of a restrained modulus of elasticity of 100,000 psi.

Freshly excavated firm shale deteriorates rapidly when exposed to dry air and, when subjected to repeated wetting and drying, decomposes to a sticky gumbo clay. Therefore, in excavating for foundations of structures, it is necessary immediately to protect the shale from air slaking by the use of a bituminous sealer, or to place protective concrete within a few hours.

The bentonite seams in the shale have a soapy feel, are fairly hard and rather brittle. They sometimes appear mottled, greenish-gray in color and some of the thicker seams generally seem to be divided horizontally into at least two sections, one having a shiny appearance, the other dull. Bentonite seam core samples sometimes separate rather easily along inclined planes which appear to be existing joints. Bentonite exposed to surface drying soon shows the appearance of fine hair cracks on the surface and small fragments bounded by such hair cracks can readily be detected.

A few hand-cut undisturbed samples from one of the thicker firm bentonite seams were tested under the supervision of Casagrande at Harvard University, Cambridge, Mass., as part of a research investigation sponsored by the United States Waterways Experiment Station of the Corps of Engineers in Vicksburg, Miss. From this project, a mixed series of unconfined and triaxial compression tests show the firm bentonite compressive strength to be from 12 tons per sq ft to 15 tons per sq ft. It is interesting to note that the same research project also tested a number of 2-in. core samples from the Cucaracha shale of the Panama Canal project. These were triaxial compression tests and showed a much wider range in compressive strength from approximately 4 tons per sq ft to as high as 15 tons per sq ft. Such variation is probably caused by the slickensided material.

The bentonite seams consist essentially of a crystalline clay-like material originally deposited as a volcanic ash and later altered to a clay mineral. Upon wetting, bentonite will swell several times its dry volume; however, this same swelling acts to seal off against the passage of water through the formation. For Atterberg tests on the bedrock, several changes in the standard test procedures were made as follows: (1) the sample was air-dried and pulverized sufficiently to pass a No. 40 sieve before testing; (2) the sample was mixed with water above the liquid limit and allowed to "temper" for 16 hr; and (3) the curve for liquid limit determination was developed by progressively reducing the moisture content. These tests show the liquid limit to generally be approximately 100 for the shale and 300 for the bentonite seams.

An interesting characteristic of the shale was noted during the Atterberg testing. In soaking the samples to obtain high moisture content, it was found that specimens that had the original moisture were slow to absorb moisture. On the other hand, an air-dried, or oven, specimen could easily be mixed with water into a homogeneous paste. This may be explained because, in the natural state, many of the highly colloidal particles are difficult to separate by simple mechanical mixing. However, once the material is dried, subsequent wetting produces thorough break-up of the colloidal agglomerations and additional water molecules can readily be absorbed into the lattice structure of the clay particles. This characteristic is considered to be quite important in regard to weathering action of this shale bedrock.

Quite extensive mineralogical analyses of the Pierre shale have been carried out by the Corps of Engineers[5] partly through the aid of Ralph E. Grim, geologist-mineralogist at the University of Illinois, Urbana, Ill. These analyses show that the shale itself contains from 40% to 60% montmorillonite, with c-axis spacing of approximately 17 angstroms (indicating a mixture of two and three molecular water layers). The shale also contains approximately 30% quartz and cristoballite, generally less than 5 microns in diameter, and a small amount of illite. The bentonite seams, however, indicate approximately 90% montmorillonite clay minerals with a dominate exchangeable base of sodium, and vary from two molecular water layers for the firm seams to as many as five layers for fault gouge or soft weathered bentonite material. The analysis then appears to indicate a material that is highly unstable; that is, it would change in volume readily with additional moisture and be-

[5] Unpublished draft of "Report of Investigation of Expansive Characteristics of Shale and Weak Rocks," by Missouri River Division, Corps of Engrs., U. S. Army, May, 1954.

come highly plastic and weaken in strength proportionally to the amount of absorbed water.

EMBANKMENT DESCRIPTION

A typical section of the valley embankment is shown in Fig. 3. The embankment has been designed with a central section comprised of rolled overburden fill. This central section is buttressed by large upstream and downstream berms constructed of shale spread in 18-in. layers, broken up by a minimum of four passes with a large-diameter, spike-tooth roller and compacted by the hauling and spreading equipment. The embankment essentially used all the material excavated for the power and outlet works structures and provided adequate stability and long seepage paths for control of reservoir water flowing through the alluvial foundation under the embankment. A sheet pile cutoff through the pervious alluvium to bedrock is located approxi-

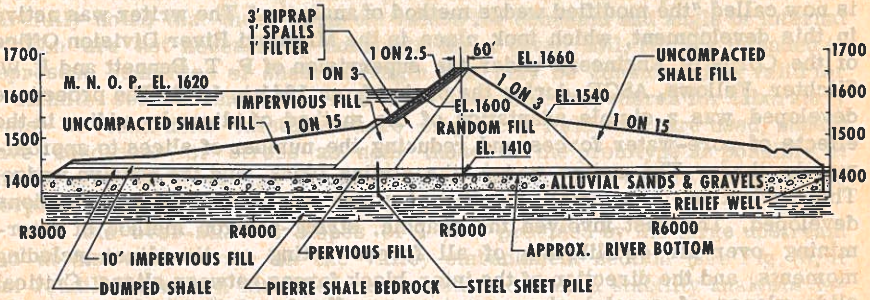

FIG. 3.—MAXIMUM EMBANKMENT SECTION

mately 400 ft upstream of the axis of the dam. A relief well system will be installed at the downstream toe. The central rolled fill zone of the embankment is separated into two sections, the upstream third of select impervious fill and the downstream remaining section of random fill material. The fill materials for the central zone right of the closure section, or approximately station 80 + 00, were excavated entirely from the right abutment where select impervious clays were in short supply from structural excavation areas. This resulted in some borrow of silty clay material from the right valley terrace immediately upstream of the dam. Due to the over-abundance of glacial outwash sands and gravels on the right abutment, the more downstream part of the random fill is of a pervious material. On the other side, however, all of the central zone left of closure is generally impervious and came from the glacial till deposits on the left abutment. A minor percentage of highly weathered shale, which had been worked into the till by glacial action, was mixed and included into the rolled fill.

With the flat excavation slopes designed around the power and outlet works structures in each of the abutments and the generous upstream and downstream berms that are flared and raised to higher elevations in transitioning the embankment into each abutment, a general balance of earthwork is obtained, exclusive of the excavation for the remote spillway landward of the right abutment area. Shale excavation for the spillway structure of approximately 30 million cu yd was wasted into large berms along each side of the channel and upstream of the approach. The embankment for the dam totaled approximately 92 million cu yd.

EMBANKMENT STABILITY

In the modern soil mechanics approach for an embankment design, as is often done in other fields, the engineer must rely heavily on empirical knowledge. Such knowledge gained from the experiences on the Fort Peck Dam project were of vital importance at Oahe and will be examined in some detail in regard to the stability analyses.

The method of analysis used for all of the large dams in the Missouri River basin was developed from the Swedish method of slices originated largely through the work of Wolmar Fellenius in the 1920's.[6] This technique is now called "the modified wedge method of analysis." The writer was active in this development, which took place in the Missouri River Division Office of the Corps of Engineers under the supervision of P. T. Bennett and F. W. Slichter, Fellows, ASCE, during the period from 1941 to 1943. The procedure developed was a simple adaptation of the method of slices by adding in the effects of pore-water forces and reducing the number of slices to approximately three: The active wedge, the sliding block, and the passive wedge. This reduction to a small number of slices was part of the modifications developed. The rest involved the graphic, string-polygon method of determining over-all equilibrium of all forces acting on the slices including moments, and the direction of the inter-block forces between slices. Critical slide planes of special shape were generally found that had long, nearly horizontal sections through the weak and weathered bedrock foundations. These critical slide planes were steeply sloping through the active thrust area, nearly horizontal in the large central part and moderately sloped upward for the passive part at the toe. Within reasonable limitations, the greater the percentage of the slide plane passing through the weak shale the more critical or lower safety factor would result. The final developmental phase showed that practically the same results as obtained with the complicated curve and many slices could be realized in much less time by the modified wedge method. For a close balance of moments, as well as vertical and horizontal forces on all slices, it was necessary to assume the direction of the inter-block forces generally parallel to the surface of the embankment. These inter-block forces are now generally assumed to act horizontally for convenience in computation, which gives a slight additional conservatism in the results. This method has been generally adopted by the Corps of Engineers for use under similar conditions and is described in practical form in Appendix IV to the "Engineering Manual."[7] It should be noted that this method of analysis is most applicable with dams founded on thinly stratified formations containing one or more weak layers, or on foundations that logically would force the failure plane to be on a nearly straight plane, such as at the Oahe project.

6 "Calculation of the Stability of Earth Dams," by Wolmar Fellemius, Transactions, 2nd Congress on Large Dams, Vol. 4, Washington, 1936.

7 "Engineering Manual," EM 1110-2-1902, parts may be purchased from Super. of Documents, Washington, D. C.

Obviously, a dam of this size and damage potential upon failure should have an embankment designed with a reserve of strength. This extra stability then provides a margin of safety against the unexpected and the unknown. The factor of safety for embankment stability is expressed as the ratio of the available shear strength to the average strength necessary to maintain equilibrium. This ratio is applied uniformly along all parts of the slide plane. For the total stress approach, it is also assumed that the shearing strength may be represented by an expression in the form of Coulomb's empirical Law:

$$S = c = \sigma \tan \phi \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which c and ϕ are the apparent cohesion and friction angle, respectively, that apply for each soil under the existing conditions. The normal pressure, σ, is the effective intergranular pressure on the failure plane. It must be carefully noted that these apparent values for cohesion and angle of internal friction are not necessarily constants for a given soil, but are intended to represent quantities of shear strength that may be considered as valid for a definite set of conditions. It follows then that procedures for laboratory tests, or conditions existing where empirical field data are used, should have conditions of stress as near those in the prototype as possible. It should also be noted that the total stress approach is basic throughout the Corps of Engineers.

With the over-all flat slopes of 1 on 7 provided by the large upstream and downstream berms on the valley embankment section (see Fig. 3), it can readily be shown that the results obtained from a stability analysis depend largely on the foundation strength, which is governed by two factors: First, the seepage and excess pore-water pressures that can develop in the embankment and the foundation; and second, the shear strength in the top or weathered zone of the shale bedrock.

A semi-empirical and conservative approach was deemed advisable on both of these important factors. For excess pore-water pressures in the foundation shale due to the added embankment load, a study comparison was made between the theoretical maximum pressure, equal to the full embankment load, and the actual excess foundation pressures that were observed under the Fort Peck Dam. These were reported by T. A. Middlebrooks.[4] A curve was adopted for the Oahe design that was approximately midway between the two sets of data. The seepage-water pressures in the dam and alluvial foundation materials were taken from equi-pressure lines determined by electrical analogy. The analogy was made on a sheet of cardboard using an "aqua-gel" conductive medium.

On the second factor, the shear strength of the shale foundation, early project studies recognized that, as a result of the occurrence of bentonite seams at almost any elevation, they would be the weakest and, thus, the most critical surface for slide planes. Although direct shear and triaxial compression tests were run on core samples of the shale seams, the results were quite variable and were not considered sufficiently reliable. Strength values adopted for shear along bentonite seams were largely based on judgment and experiences at other projects, such as Fort Randall, Harlan County and, particularly, Fort Peck. Even though firm seams show a high cohesive strength, the adopted value for cohesion was 0.20 tons per sq ft. The adopted frictional resistance was a tangent $\phi = 0.20$, or $\phi = 11.4°$. This value was closely checked by a stability analysis of the Fort Peck slide section immediately before failure, assuming an appropriate strength for the fill material, core pool seepage forces, and a factor of safety of 1.00.

In all stability analyses on the Oahe Project, the horizontal part of the slide surface is assumed to be along a bentonite seam when passing through the shale formation. For the central rolled fill zone of the embankment, the adopted shear strength factors were cohesion = 0.35 tons per sq ft and a tangent $\phi = 0.35$ (or $\phi = 19.3°$). The adopted strengths for the shale berm fill material were taken from ultimate strength plots of direct shear tests on weathered shale samples (tan $\phi = 0.21$, $c = 0.26$ tons per sq ft). In these tests, the undisturbed core samples were trimmed to fit the shear box, immersed and immediately loaded so as to consolidate overnight, and then sheared at a rate of from 0.01 in. per min to 0.02 in. per min. The weathered samples of shale had a dry density of 80 lb to 85 lb and a natural moisture content of approximately 35%. The strength used for the ultimate plots is that remaining after the maximum is passed and the strain has reached several tenths of an inch.

This procedure resulted in a reduction of the apparent cohesion obtained from the plots, which was considered appropriate to allow for long-time effects of saturation and related phenomenon. Using the preceding data, the embankment slopes were designed so as to obtain a minimum safety factor of 1.50 both upstream and downstream. Because of the large reservoir storage of over 23 million acre-ft at maximum normal pool (El. 1620), it was not considered possible to have a rapid drawdown condition on the upstream slope.

As the embankment approaches and is carried up the abutment slopes, it was found necessary to raise the stabilizing berms by several transitions to obtain the required factor of safety (The slopes in Fig. 1, are not detailed enough to show the transitions and raised berms.) This was necessary because of the gradual reduction of the high-strength, alluvial sand-gravel layer and the thickening and rising level of the weathered surface zone of the shale bedrock as shown in Fig. 2. These higher berm slopes take off from the central zone as high as El. 1610 at the right abutment both upstream and downstream.

At the left abutment, the treatment was somewhat different due to the large foundation excavation treatment resulting from the slide, to be examined subsequently. The embankment section is transitioned to an entirely select overburden fill (no shale fill) landward of station 120+00. This station is approximately 700 ft landward of the left river bank or, generally, the toe area of the abutment. Landward of this point, the downstream embankment slope has been warped to a 1 on 8 from crest to the surge tank-powerhouse area, and this slope flares on into the abutment 1 on 9 cut slope landward of the powerhouse.

At the toe of the dam and the left abutment on the upstream side, a large stabilizing berm at El. 1520 was placed to insure stability of these critical slopes, which nearly surround the important power intake area. After the first transition from the valley section, the embankment slopes above the 1520 intake level flatten from a combination 1 on 3 to 1 on 4.5, through a combination 1 on 3 to 1 on 7, on to the flatter 1 on 10 cut slope landward up the abutment.

For intercepting and controlling the flow of seepage through the abutment sections, a french drain with pipe outlet in the right abutment and an inclined sand drain through the left abutment fill were constructed.

In summarizing the over-all stability of the dam, it is believed that the most critical section probably lies immediately riverward of where the stripping of the surface weathered shale stopped near the left river bank. This stripping of the shale was carried as far as dewatering was practical and stopped at station 110+50 (see Fig. 9). The head above the shale acting in the valley sands at this point was approximately 30 ft. This critical zone is fairly narrow, as within a few hundred feet riverward the shale bedrock drops to the typical valley level. An interesting comparison of over-all slopes on Oahe versus those of the Fort Peck Dam shows, respectively, 1 on 7 and 1 on 7.8 downstream, and 1 on 8 and 1 on 7 upstream. Other factors for comparison, such as foundation conditions, and so forth, lead to too many complications to be included herein, but at least the height of the two dams and the over-all slopes are quite similar.

CUT SLOPE PROBLEM

The cut slope problem of design for adequate stability is considered to be a much more complex problem than that of similar design for the embankment slopes, because the effective shearing strength of the shale formation as a whole, against the possibility of slides founded in the shale bedrock, is governed by many factors: (1) The wild and closely spaced pattern of faults; (2) the long-time effect of reservoir seepage waters; (3) the zones of weakening evidenced by closely spaced slickensides; (4) variation in the depth of weathering effects on the fault planes and the bentonite beds; and (5) the wide range of strength resulting from the effects of water on the reactive clay minerals in the formation. In the case of the embankment design, the principal zone of weakest strength is undoubtedly in the surface weathered shale zone, and for the major or valley part of the embankment this will be overlain by considerable thickness of cohesionless sands and gravels with rather high frictional strength. On the other hand, in the cut slope problem, the complex condition of the bedrock formation poses the difficult problem of determining the composite strength along any given slide plane, or combination of slide planes, for the evaluation of slope stability.

From early design studies, made in 1948, the Corps of Engineers realized that the stability of the deep cut slopes in the shale would depend on the complex factors outlined and thought it wise to consider also a study of the existing slopes in the Pierre shale along the valley near the project. The height and inclination of the existing cuts and natural slopes in the vicinity of Pierre were measured and the experience with cut slopes in the similar Bearpaw shale of Fort Peck Dam was also reviewed. Included with these data were slope chart studies reported in 1942 by D. F. MacDonald on the Cucaracha shale of the Panama Canal project.[8] The principle factor inherent in any study of existing

[8] "Panama Canal Slides," by D. F. MacDonald, Dept. of Operations and Maintenance, Balboa Heights, Canal Zone, September, 1942.

or natural slopes is the unknown condition of the particular slope being studied. Such facts as the depth of weathering, the extent of weakening of the potential slide surfaces in the formation, the length of time that this slope has been exposed, the factor of safety against sliding, and all other related factors are entirely unknown from a mere superficial examination and measurement of the ground surface. It is recognized that the slope chart approach is one of the handy "tools" to be used by the designer; however, it is hoped that, in the examination and examples to follow, some of the shortcomings of this tool in giving sufficient data for finalizing and particular slope design of a given project with a foundation bedrock similar to the Pierre shale will be noted.

A supplemented version of the slope chart developed in the 1948 project studies at Oahe is shown in Fig. 4. Various data incorporated in this slope

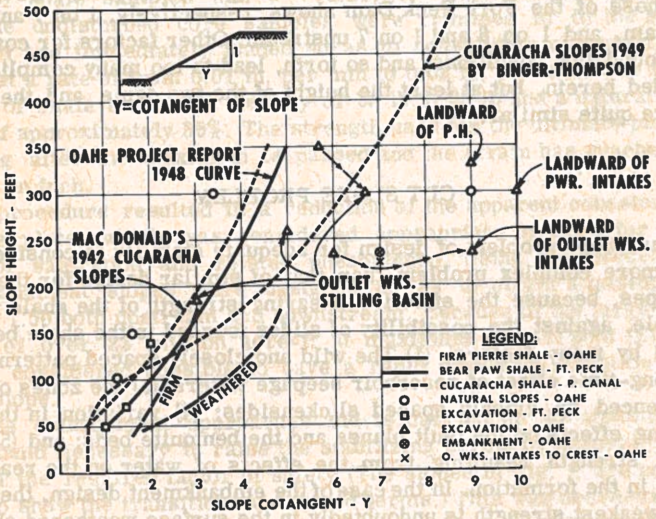

FIG. 4.—SLOPE CHART

chart are apparent on examination and study. It may be noted that the recommended curve for the Oahe project design is quite similar, in fact parallel to, the 1942 Panama Canal approved slope curve to be used for the Cucaracha shale at that time. MacDonald, resident geologist at the project for a number of years, based his recommended slope curve[8] on wide experience with the Cucaracha formation and on qualitative studies of existing slopes and slides in this material. Another slope chart curve for the Cucaracha shale has been added. This shows the 1948 design curve reported by Wilson V. Binger, F. ASCE, and Thomas F. Thompson, Aff. ASCE.[9] Comparison of the two design curves on the Cucaracha shale clearly indicate a trend toward additional conservatism for the higher depth of cuts in the intervening period of about

9 "Panama Canal—The Sea-Level Project: A Symposium: Excavation Slopes," by Wilson V. Binger and Thomas F. Thompson, Transactions, ASCE, Vol. 114, 1949, pp. 734-754.

6 yr. One other set of data for cut slope design, which is for the Fort Peck Bearpaw shale, is plotted in the dashed line symbol as reported by K. S. Lane, F. ASCE.[10] These curves for safety factor of unity were developed during design of the second powerhouse in 1953-1956. Analysis of the individual data and the total picture that may be obtained from Fig. 4 follows.

First, note the two circle points shown at a height of 300 ft for natural Oahe slopes. These points represent the extremes, with an indefinite number of points between, for the natural valley slopes that now exist along the Missouri River in central South Dakota. Consider any one slope, for example, the 1 on 3.4 slope 300 ft high. (All references to slopes herein, such as 1 on 3, means a 1 vertical to 3 horizontal.) At the present time, this slope is standing and has been for at least 10 yr by observation. How long this slope will remain stable is unknown, but it can be predicted with assurance that it will slide and will probably do so within the next 50 yr due to the effect of the rising reservoir. On the other side of the natural valley picture, there are 300-ft high slopes at a slope of 1 on 9. On these also, the safety factor against slide action is unknown. Many of these flatter slopes have undoubtedly had slide activity in the past, while some may still be in the process of a slow-creep type of slide action. It seems clearly apparent that with any natural slope in the Pierre shale formation there are always so many unknown subsurface conditions that a design chart based on such data is at best only an indication of the extremes to be considered. Other slope experiences at Oahe confirm this statement.

Note the points in Fig. 4 shown by a triangle symbol for existing excavation slopes around the outlet works stilling basin. The highest, at 350 ft, stood for several years as the slope landward from the basin. This slope was later reduced in height by borrow of overburden material overlying shale in the higher ground, as the material could be usefully incorporated into the embankment fill. Two other cuts of lesser heights are shown that have been stable for several years. It is apparent that these slopes fairly closely corroborate the assumed 1948 design curve. Unfortunately, or fortunately, depending on the viewpoint, these are not all the facts.

Additional data are shown on Fig. 4. Two points are shown for the slope landward of the outlet works tunnel intakes. Actually, the original cut tried for the intakes was a 1 on 3 and resulted in the first slide experience on the project. A more detailed examination of the first slide is given subsequently. The 1 on 6 slope 235 ft high represented by another triangle symbol was excavated and stood stable for several years; the flatter 1 on 9 slope was obtained later in the project development solely for added factor of safety. This slope is considered one of the more vital slopes in regard to safety of the project; in addition, the future effects of the rising reservoir pool were kept in mind.

It is noted also on Fig. 4 that the flattest slopes of 1 on 9 and 1 on 10 have been used for the excavation landward from the important power intakes and powerhouse. There are two remaining points for 1 on 7 slopes: The encircled "X" point to bring the over-all embankment slope into the picture; and the plain "X" for the over-all slope from the dam crest to the outlet works approach channel at the intakes.

10 "Field Slope Charts for Stability Studies," by K. S. Lane, Proceedings, 5th Internatl. Conf. on Soil Mechanics and Foundation Engrg., Paris, 1961.

It seems pertinent at this point to relate some of the data and experiences with the Cucaracha shale formation, because in many ways it is quite similar to the Pierre shale. Binger and Thompson based the standard design curve for the Cucaracha, which has been replotted on Fig. 4, on the cohesive strength of the material derived from analytical studies of a stable bank cut, combined with the results of a simple laboratory test to measure the friction along slickensides. The strength values used were a friction angle of 10° (tangent $\phi = 0.176$) and cohesion equal to 16 psi (1.15 ton per sq ft). Their Cucaracha design curve for cuts above 200 ft was developed from data published by D. W. Taylor.[11] The part of the curve for cuts less than 200 ft is empirical and is based on recommendations made by MacDonald on the basis of his broad experience with the Cucaracha formation.

It is noted that considerable comment, both agreeing and disagreeing with Binger and Thompson's results for the slope curve, is given in the various discussions of their paper.[9] Realizing an ever present truth that any stability analysis approach is no better than the various assumptions incorporated into the analysis, it is not surprising to note that most of the disagreements expressed are on one or more of the assumptions. In his discussion,[12] E. Montford Fucik presents good arguments to show that, for the higher slopes, the curve is unduly conservative. He points out (1) that the assumption of sudden drawdown was applied twice in the analysis, and (2) that the stable bank was assumed to have only a safety factor of unity while it could easily have been near a 1.20 safety factor. In general, Fucik appears to agree with a curve near the earlier curve of MacDonald's.

Casagrande's discussion[13] also finds vulnerable spots in the Binger-Thompson approach, which casts doubts on the validity of their results. He first points out that the sequence of various layers of the Cucaracha formation in the model stable slope is not identical to that in the slopes that failed in the Culebra slides, and summarizes by saying that, in his opinion, the tacit assumption that for the purpose of analysis these slopes have similar strength characteristics is not tenable . Casagrande acknowledges, nevertheless, that their design represents the best that could be accomplished with the tools and knowledge available.

In the writer's opinion, the important point to be noted from the these discussions is one which was inferred by several discussers and mentioned by Binger and Thompson in their closing remarks[14] on the discussions; that is, the future action that could add the most significant data for improving the slope design would be detailed exploration of critical areas, so that more accurate data on the natural strength and structural geology of the foundation would be obtained.

SLIDE EXPERIENCES

In 1950, during the first earthwork contract at Oahe for the excavation of the outlet works approach channel, a small slide occurred in a 1 on 3 cut

11 "Stability of Earth Slopes," by D. W. Taylor, Journal, Boston Soc. of Civ. Engrs., July, 1937, p. 197.

12 Discussion by E. Montford Fucik of "Panama Canal—The Sea-Level Project: A Symposium," Transactions, ASCE, Vol. 114, 1949, pp. 831-834.

13 Ibid., by A. Casagrande, pp. 870-874.

14 Ibid., by Wilson V. Binger and Thomas F. Thompson, near bottom p. 899.

landward of the intake area. Its location is shown on Fig. 1 by the small cross-hatched area in the right abutment. At that time, the outlet tunnels were straight and the intakes were located several hundred feet landward and toward the crest of the dam from their final location. A true scale section looking upstream, as in Fig. 5, shows the original ground, the dashed line of the original excavation for the stage I earthwork contract with outline of intake area, the cross-hatched slide zone, and the heavy solid lines for the cut as it is now.

Explorations following the beginning of the slide showed the slide block was bounded in the rear by an old fault plane and was based along weathered bentonite seams in the Oacoma facies of the DeGray member of the shale formation. The slide surface along the steeply dipping fault was partially smeared with yellow, weathered bentonite and the depth of weathering extended below the base of the slide zone. Because the slide was moving at a slow creep (approximately 1.3 in. per day for first 10 days), the safety factor for the slide was taken equal to unity and, by stability analysis using the modified wedge method, the effective shear strength along the slide plane was computed.

It is important to note that, in this type of analysis, assumptions such as pore-water forces should not be included in the study if the more conserva-

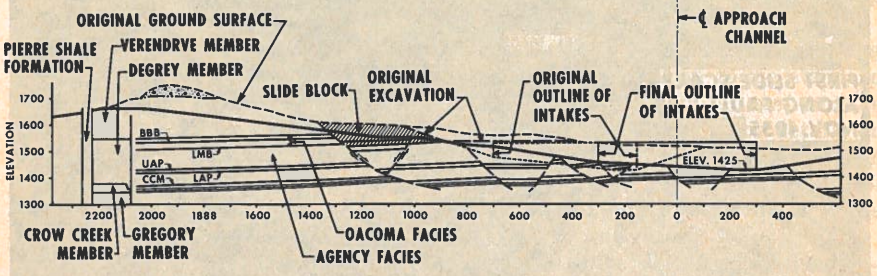

FIG. 5.—SLOPE LANDWARD FROM OUTLET WORKS INTAKES

tive or lower shear strength values are desired. Although several combinations of cohesion and friction were determined, the adopted values were cohesion equal to 0.15 ton per sq. ft and a tangent $\phi = 0.15$ ($0 = 8.5°$). Undisturbed box samples were obtained from the bentonite bed along the slide plane. These samples were tested by both direct shear and squeeze tests for shear strength determination. The results from direct shear tests checked the adopted values closely; the squeeze tests showed slightly less strength.

Excavation at the toe of the cut was stopped immediately following the slide and some material at the top of the slide block was removed to lessen the driving force. This started another slow moving slide above the first one. The total area involved then was still only a part of the entire slope to the top of the abutment. In general, the remedial treatment of the slide area was simply to excavate enough of the weathered slide mass to reach a stable slope. The decision to move the intakes out from the toe of the slope facilitated the flattening and thus the stabilization of the slope.

Slide movement of the lower slide was stopped completely in November 1950, but resumed again the following spring. This was believed due to considerable snow melt and rainfall, so the logical remedy was to try drainage by nearly horizontal drain holes located at the toe of the slide. Although the slide had practically stopped by the time the horizontal drains were installed in 1953, it was possible by alternating plugging and opening the drain outlets to determine that a definite beneficial effect on the stability could be realized from the drainage system. This effect would not have been apparent had not the safety factor of the slope been near unity.

As was noted in the analysis of the slope chart, Fig. 4, the first completely excavated cut was a 1 on 6. The later flattening to an over-all of 1 on 9 was

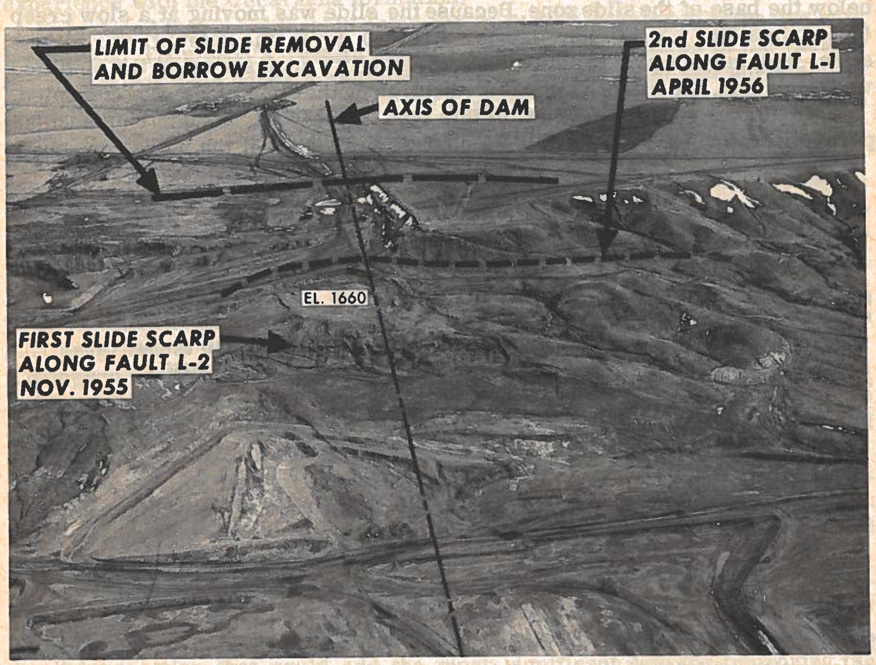

FIG. 6.—AERIAL VIEW OF SLIDE, LEFT ABUTMENT, MARCH 20, 1956

done by cutting back the top part of the 1 on 10 slope in 1959 and 1960, under stage VII earthwork contract.

Major Slide Activity began on November 18, 1955, in the central area of the left abutment. An aerial photograph, Fig. 6, shows the area the next spring before any slide excavation operations had begun. When the slide began, the earthwork stage IV contractor had done the first summer's work on the left side of the channel closure section. Among other things, this included the dewatering of an area approximately 400 ft out from the left bank of the river and 2,500 ft upstream and downstream. This was to permit removal of weathered shale foundation material and inspection to insure that firm shale bedrock was reached in the stripping operations. This area is shown on Fig. 9.

The excavation of unsuitable weathered shale had also begun on the lower abutment slope. Some 20 ft of material removal is evident in the lower part of the photograph in Fig. 6 at the toe of the abutment. A little excavation about three-fourths the way up the abutment had been done to the crest of dam level at El. 1660 and several million yards of overburden excavation, at the upstream top portion of the abutment, had been excavated and placed in the embankment fill over the valley.

Based on observations of field geologists, the initial displacement was an instantaneous rupture, even though a rather large movement continued for several days. The rapid displacement was approximately 5 ft horizontally and 22 ft vertically and covered an area one-fifth of a mile parallel to the abutment slope and one-quarter of a mile wide. The rate of movement immediately following the initial slump was approximately 4 ft per day. The rate decreased gradually until, a month later, it was less than 1 in. per day and, by late January of 1956, almost all appreciable movement had ceased.

The slide was attributed to the stripping excavation at the toe of the abutment and to the previously unsuspected highly weathered condition of the abutment bedrock. There was also an unknown adverse effect from extensive overburden prewetting operations on top of the abutment, which had been going on for 7 months prior to the slide. It is noted that no prewetting of excavation areas was permitted in subsequent operations. Moreover, more stringent controls with regard to the requirement of excavating from the top down were put into effect.

A large additional exploration program was fully launched by the following spring, involving many forms of investigation. In addition to the standard 6-in. drive and core holes, Wilson-Shannon tiltmeter installations, surface reference points and gages, and calyx holes were used to develop details on the bedrock. This program was to take several months; consequently, it was decided to try an initial cut slide removal in April 1956. A 1 on 3 cut of 800,000 cu yd of shale, beginning at El. 1660 about 200 ft back of the top slide scarp and going down to a level bottom at El. 1540 was made.

Before this excavation was completed, the slide activity expanded approximately three times to a total area of approximately 65 acres, or one-tenth of a square mile. This area is shown by the solid and short dashed lines on Fig. 7. The investigational program, except for drill holes, is also shown on Fig. 7, together with the preslide and final alinements of the power tunnels and locations for intakes and powerhouse. Surface reference points to measure movements during the spring and summer of 1956 were established on three lines, E, F and G lines and points numbered as shown on Fig. 7.

Study of these points and plots of movements as shown on Fig. 8(a) and 8(b) shows that the most active sliding had occurred in a rather narrow band along a center line passing through point 5 in line G and a point midway between points 10 and 11 in line E. This zone of major sliding shows a maximum horizontal movement of 30 ft during the 4 summer months observed.

The determination of the base of slide movements was considered important, but was found to be quite difficult until slide movements had slowed down sufficiently to allow safe use of drilling equipment on the slide area. Depth to the slide plane was found in a few of the open 6-in., uncased drill holes by observing displacement through the aid of sunlight reflected from a mirror into the hole. Other indications of slide plane level were obtained

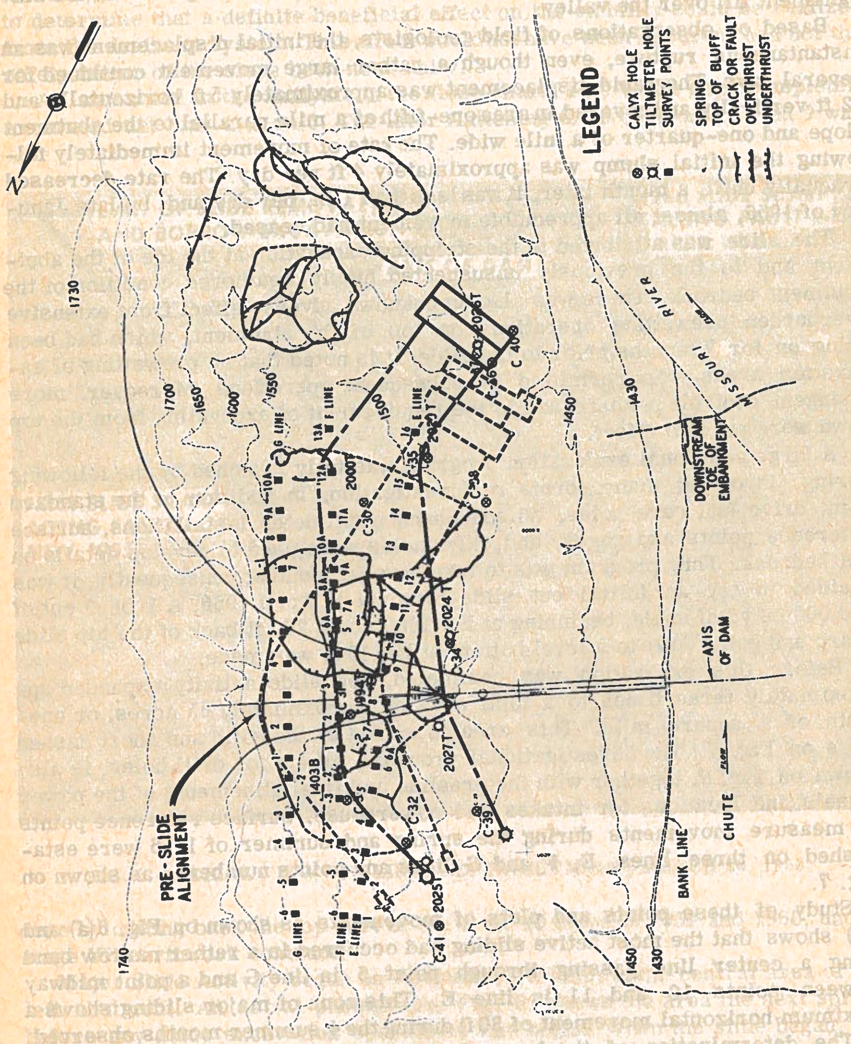

FIG. 7.—SLIDE AREA, LEFT ABUTMENT

from some of the tiltmeter installations. A small amount of shearing action would distort the plastic casing in the tiltmeter hole and thus stop the meter when it was lowered to that level. Because this would prohibit subsequent tiltmeter readings below such levels, the use of such installations in areas of considerable slide activity is not recommended. They have compensating advantages, however, in being sensitive to slight movements such as creep and are surprisingly accurate. Their use in checking early symptoms of weakness or overstress on some of the critical final cut slopes at the Oahe project should prove important.

A third method for determining the level of slide action in the shale was through the use of 30-in. diameter calyx holes. Early inspection of these holes in the slide area was accomplished by lowering a man to a depth of as much as 60 ft inside a specially designed, light steel cage, for overhead fallout protection, down into the calyx holes. Excellent data were obtained by this procedure, but toward late summer shortly after an inspection trip down one of the holes near the middle of the slide area by the writer, the District office decided it was not a safe operation in uncased holes.

The average slope of the abutment when the slide began was a 1 on 5.5 and involved a vertical horizon of approximately 150 ft. As in the right abutment slide, the shale formation member for the zone of movement was the DeGray, with the Oacoma facies being the primary stratum of the slide.

The Oacoma bed is generally 37 ft thick and averages one bentonite seam per foot. The thickness of bentonite seams varies from 0.6 ft to thin partings. In this facies are logged three of the thickest and most prominent marker bentonite seams: The Upper Micaceous Bentonite at the top, or UMB; the Big Bentonite Bed near the center, or BBB; and the Lower Micaceous Bentonite at the bottom, LMB. The general condition of the Oacoma is shown on the geologic profile, Fig. 2, and the marker bentonite seams in the more detailed sections of Fig. 5 for the right abutment and Fig. 10 for the left abutment.

The next lower marker beds in the DeGray member are the Agency Pair Bentonites, usually approximately 10 ft apart and shown on the sections as UAP and LAP. The lower marker beds are in the Crow Creek member of the shale and are the CCM for a bentonite seam and the marl bed approximately 8 ft thick. Most of the preceding marker bentonite seams are from 0.3 ft to 0.6 ft thick. There are also approximately twenty other seams of bentonite from the marl to the Oacoma facies that very in thickness from 0.01 ft to 0.07 ft. Below the marl, the bentonite seams are more numerous again, at least for 50 ft of thickness.

To give an interesting total picture of the exploration program of the left abutment area, the following facts are noted. Prior to the slide, the total number of borings was 350, spaced on a general grid pattern at from 200-ft to 300-ft intervals. At the conclusion of explorations after the slide, an additional 350 borings were made, spaced at half the original interval and closer in special areas of complicated faulting. The time spent in study of the exploration data was also increased several times.

SLIDE TREATMENT—LEFT ABUTMENT

After extensive studies on various power structure layouts in both abutments, it was decided to leave them in the left abutment. Regardless of where

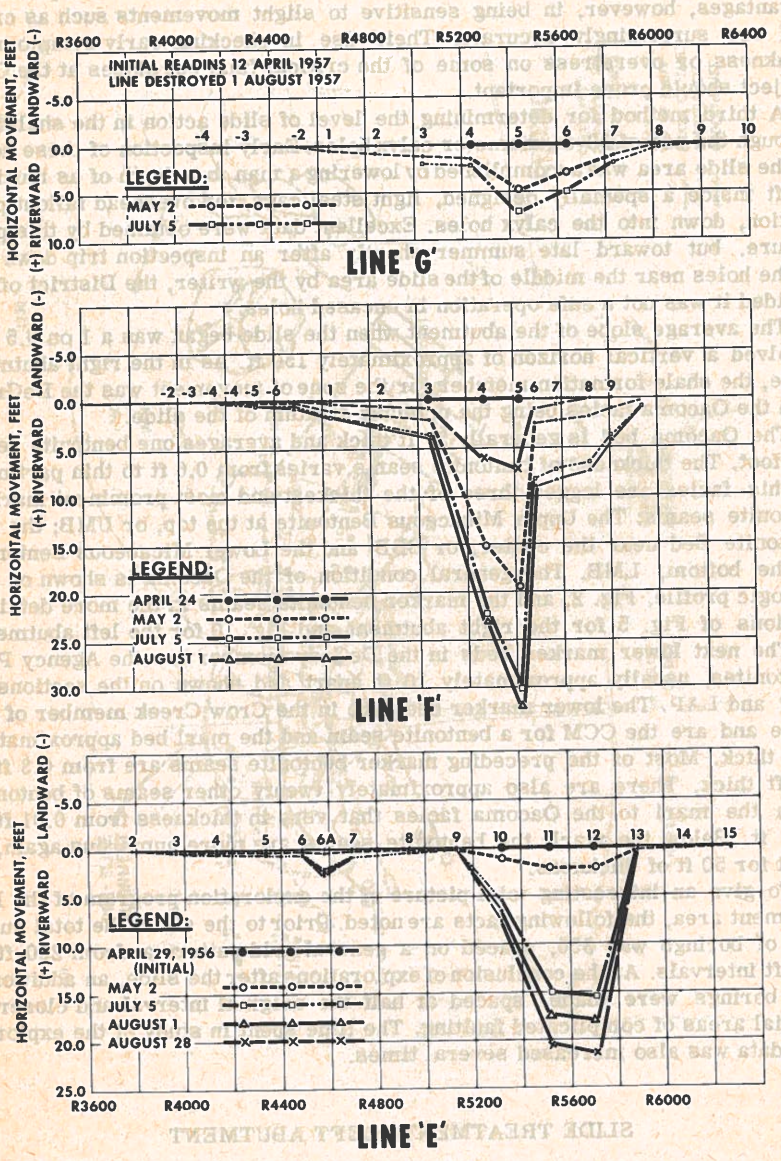

FIG. 8a.—PLOTS OF SURFACE MOVEMENTS

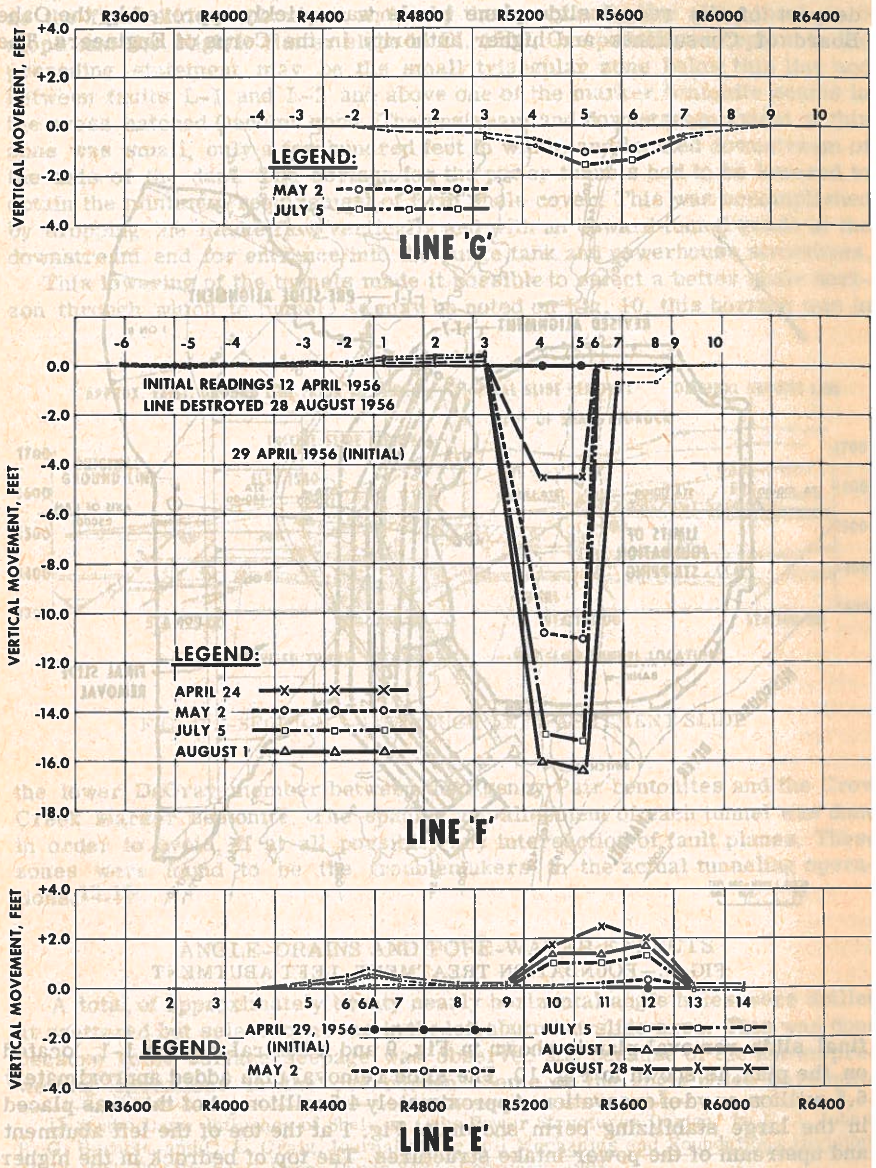

FIG. 8b.—PLOTS OF SURFACE MOVEMENTS

the power structures were to be located, the remedial treatment of this abutment would have been the same, because the over-all safety of the dam was involved. It was obvious that the most positive and foolproof corrective measures had to be taken. The District's decision to excavate completely the entire left abutment foundation area to firm, unweathered shale below the deepest of the recent slide plane levels was quickly approved by the Oahe Board of Consultants and higher authority in the Corps of Engineers. The

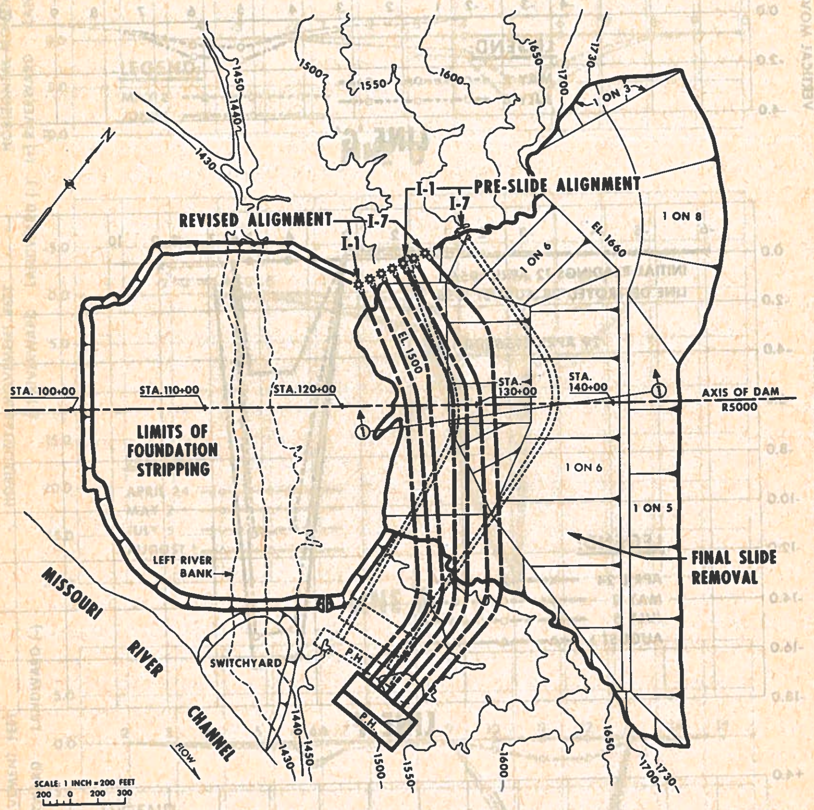

FIG. 9.—FOUNDATION TREATMENT, LEFT ABUTMENT

final slide removal plan is shown in Fig. 9 and a central section 1-1, located on the plan, is shown in Fig. 10. The slide removal plan added approximately 6.5 million cu yd of excavation. Approximately 4.5 million yd of this was placed in the large stabilizing berm shown in Fig. 1 at the toe of the left abutment and upstream of the power intake structures. The top of bedrock in the higher part of the left abutment varied considerably. Immediately downstream of the dam it is lowest at about El. 1550 or 1570, whereas, upstream of the dam, it

is near El. 1630. Section 1-1 is approximately parallel to and along the axis of the dam.

Attention is directed to Fig. 10. The initial slide moved downward along fault L-2 and the later enlargement of the slide had its most landward scarp along fault L-1. The depth of sliding was generally not as deep into the shale as the final slide removal cut shown by the heavy solid line along the 1 on 6 slope and the slightly flatter slope to El. 1500. One possible exception to the preceding statement may be the small triangular zone below this line and between faults L-1 and L-2 and above one of the marker bentonite seams in the cross-hatched Oacoma zone. The upstream and downstream extent of this zone was small, only a few hundred feet in width, and located downstream of the axis of the dam. The horizon for the power tunnels had to be lowered to obtain the minimum requirement of firm shale cover. This was accomplished by dropping the intake flow vertically and with an upward tunnel grade at the downstream end for entrance into the surge tank and powerhouse structures.

This lowering of the tunnels made it possible to select a better shale horizon through which to tunnel. As may be noted on Fig. 10, this horizon was in

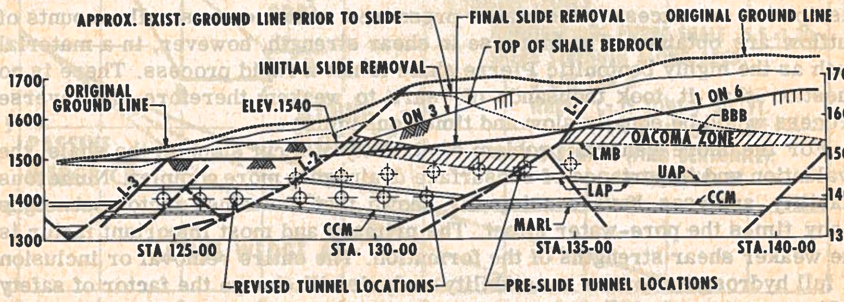

FIG. 10.—SECTION 1-1 THROUGH LEFT ABUTMENT SLIDE

the lower DeGray member between the Agency Pair bentonites and the Crow Creek Marker bentonite. The spacing and alinement of each tunnel was done in order to avoid, if at all possible, any intersection of fault planes. These zones were found to be the troublemakers in the actual tunneling operations.[15,16]

ANGLE-DRAINS AND PORE-WATER EFFECTS

A total of approximately twenty nearly horizontal angle holes were drilled at scattered but select locations in the left abutment slide area. This was done because some surface seepage was observed and because of the known prewetting operations in the overburden. Some of the angle holes were quite

15 "Oahe Dam: Influence of Shale on Oahe Power Structures Design" by E. A. Johns, R. G. Burnett, and C. L. Craig, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM1, Proc. Paper 3423, February, 1963.

16 Tunneling by Mechanical Miners in Faulted Shale at Oahe Dam" by L. B. Underwood, 7th International Congress on Large Dams, Rome, 1961, paper R-55.

effective in draining water from the shale, others were not and proved to be dry or nearly so.

One of the most effective groups drilled by use of a McCarthy auger rig was done in early January of 1956. This group of four holes hit a sizeable pocket of water. As each hole was drilled, the original flow was measured at approximately 2,000 gal per day. It was also noted that initial flow in some of the new drain holes in this group would cause flow in adjacent holes to stop completely. The total drainage from these four holes over a period of 24 days was estimated at 27,000 gal. The drainage from these most successful holes reduced to approximately 1/10 the initial flow in a period of several weeks. Study of the extensive system of piezometers covering the major slopes above the outlet works stilling basin shows that free water is generally in pockets or zones, each being apparently independent of the others.

The importance placed on subsurface drainage in a slide area, such as this case in the left abutment at Oahe, is somewhat controversial. The writer's views follow, including some comments of a more general scope. In the slide area, the only practical use of angle drains was to slow down and stop the sliding action. For this purpose, they are excellent and the effectiveness is proportional to the number of drain holes. They will accelerate the natural dissipation of excess pore-water pressures where even small amounts of outflow are obtained. The increase in shear strength, however, in a material such as the highly bentonitic Pierre shale is not a rapid process. There is no question that it took thousands of years to weaken; therefore, the reverse process would be equally slow and time consuming.

For the more general problem of stability of a cut slope in the shale, the evaluation and importance of subsurface drainage is more complex. Numerous stability analyses have consistently shown that one other factor outweighs many times the pore-water effect. The primary and most important factor is the weaker shear strengths of the formation. The entire removal or inclusion of full hydrostatic forces in a stability analysis will change the factor of safety only approximately 30% higher or lower, while a reasonably small error in the critical shear strength along a bentonite seam or fault plane can cause a 100% variation in the safety factor. An example showing how strength variation can affect slope design is given subsequently.

ADDITIONAL STABILITY ANALYSES

From the left abutment slide data and subsequent studies that developed the geologic structures of the movement zones, it was again possible to compute by stability analyses the effective shear strength values along the slide planes. The values obtained from the left abutment slide were roughly equal to those from the earlier slide in the right abutment, that is, cohesion equal to 0.15 ton per sq ft and a tangent $\phi = 0.15$ would give a safety factor of 0.95 for the slide mass.

For those who advocate slope design based on minimum strength values and stability analyses, a simple but revealing study has been devised. The modified wedge method of analysis has been used, although any conventional method would serve as well. The assumptions and results are given in Fig. 11. The factor of safety is held constant at 1.30 for a shale slope 300 ft high. The schematic section on this figure shows the slide mass ABCD, divided for

analysis into the active wedge AB, the sliding block BC, and the passive wedge CD. Along the ordinate scale, note that the intercept shows the 300-ft cut could be a 1 on 6.2 slope assuming all of the slide plane strength to be the low values derived from the Oahe slides.

Suppose that somewhere along the slide plane ABCD there were a small irregularity or jog in the surface that mobilized a small zone of high strength. For the present study, assume this high strength to equal that determined by Casagrande for firm bentonite. By following the solid curve to the point corresponding to only 5% of the total slide plane having this high strength somewhere under the sliding block BC, the 300-ft cut now is equally safe at a 1 on 4 slope. This is believed to be a good illustration of what actually happens in the Pierre shale. The deeper the slide plane is into the firm shale, the more appropriate it is to assume a large percentage of high strength to be acting. Because of the complex condition existing in the Pierre shale, it is believed that the study is significant. It first leads to the conclusion (stated previously) that is similar to that stated by Binger and Thompson.[14] It also

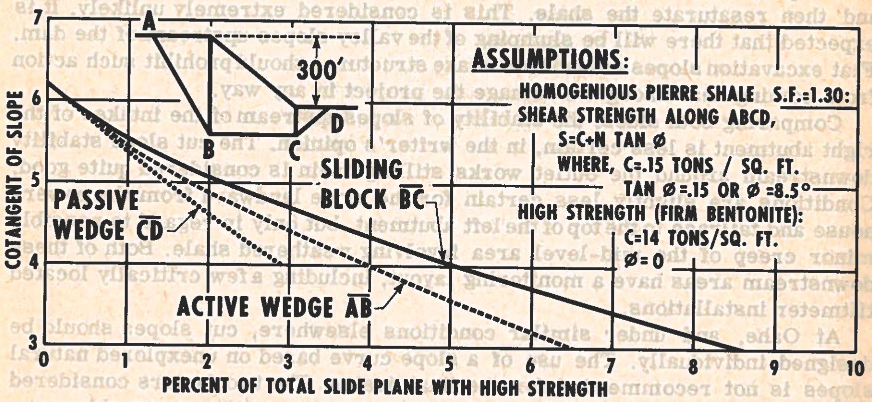

FIG. 11.—STABILITY STUDY

reveals the possible range of error in trying to establish a slope chart curve for the design of cut slopes in a formation similar to the Pierre shale.

CONCLUSIONS

It is assumed that only the passing of years can prove how successfully the problems of embankment and cut slope stability at the Oahe Dam have been handled. The concluding remarks will therefore deal with specific aspects of the two problems examined.

The embankment stability has been analyzed by use of semi-empirical methods based principally on the processes of the soil mechanics science and on data from the Fort Peck slide. The total stress approach used may not be as current theoretically as the effective stress analysis, but the conservatism resulting from inconsistencies inherent in this approach are not considered excessive for a project of such importance. One such inconsistency might be said to exist in using seepage forces or excess hydrostatic pressures in the foundation while also using apparent shear strengths, which have allowance for the same phenomenon built into the values adopted. However, because the same approach was used both on studies of the Fort Peck slide and on the Oahe embankment, the empirical aspect of these procedures would tend to correct for any error herein involved.

General conclusions on the cut slope design are considered to be less certain than for the embankment. The effects of time and the rising reservoir on the upstream cut slopes of the projects pose the major question. This question was the one so often raised by Casagrande in the early meetings of the Board of Consultants. The characteristics of the shale formation to seal itself off against the flow of seepage water is considered important. There is no evidence contrary to this statement from the experiences and records of the Fort Peck project to date.

It is believed that extreme weakening of the shale back into firm areas of the slopes, could only result if condtions develop alternatrely to desiccate and then resaturate the shale. This is considered extremely unlikely. It is expected that there will be slumping of the valley slopes upstream of the dam. Flat excavation slopes around both intake structures should prohibit such action from coming near enough to damage the project in any way.

Comparing both sides, the stability of slopes upstream of the intakes of the right abutment is less certain, in the writer's opinion. The cut slope stability downstream around the outlet works stilling basin is considered quite good. Conditions are slightly less certain for the slope landward from the powerhouse and tailrace to the top of the left abutment, but only in regard to possible minor creep of the mid-level area involving weathered shale. Both of these downstream areas have a monitoring layout, including a few critically located tiltmeter installations.

At Oahe, and under similar conditions elsewhere, cut slopes should be designed individually. The use of a slope curve based on unexplored natural slopes is not recommended for the final design. The two factors considered most important for a stability analysis approach are (1) a reasonably complete and accurate picture of the geologic structure of the bedrock, and (2) the effective over-all shear strength of the formation in the slope. The low shear adopted for cut slope stability studies at Oahe is believed to be a conservative but appropriate value. The final design of each slope should be based on judgment and experience considering all of the following:

1. The geologic structure of the foundation;
2. the depth of surface weathering or weakening, including the effect of ground-water seepage;
3. the shear strength of the weaker strata or zones along which sliding action would probably occur;
4. pore water and seepage forces;
5. results of stability analyses;
6. experiences with similar materials at other projects; and
7. the importance of this particular slope in regard to the over-all safety of the project, that is, the permissible risk factor.

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