RESEARCH ON DEFORMATION AND
FAILURE OF EARTHEN DAMS AND EMBANKMENTS

A Senior Thesis

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Partial Fulfillment of the

Requirement for the Degree of

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ACKNOWLEDGEMENT

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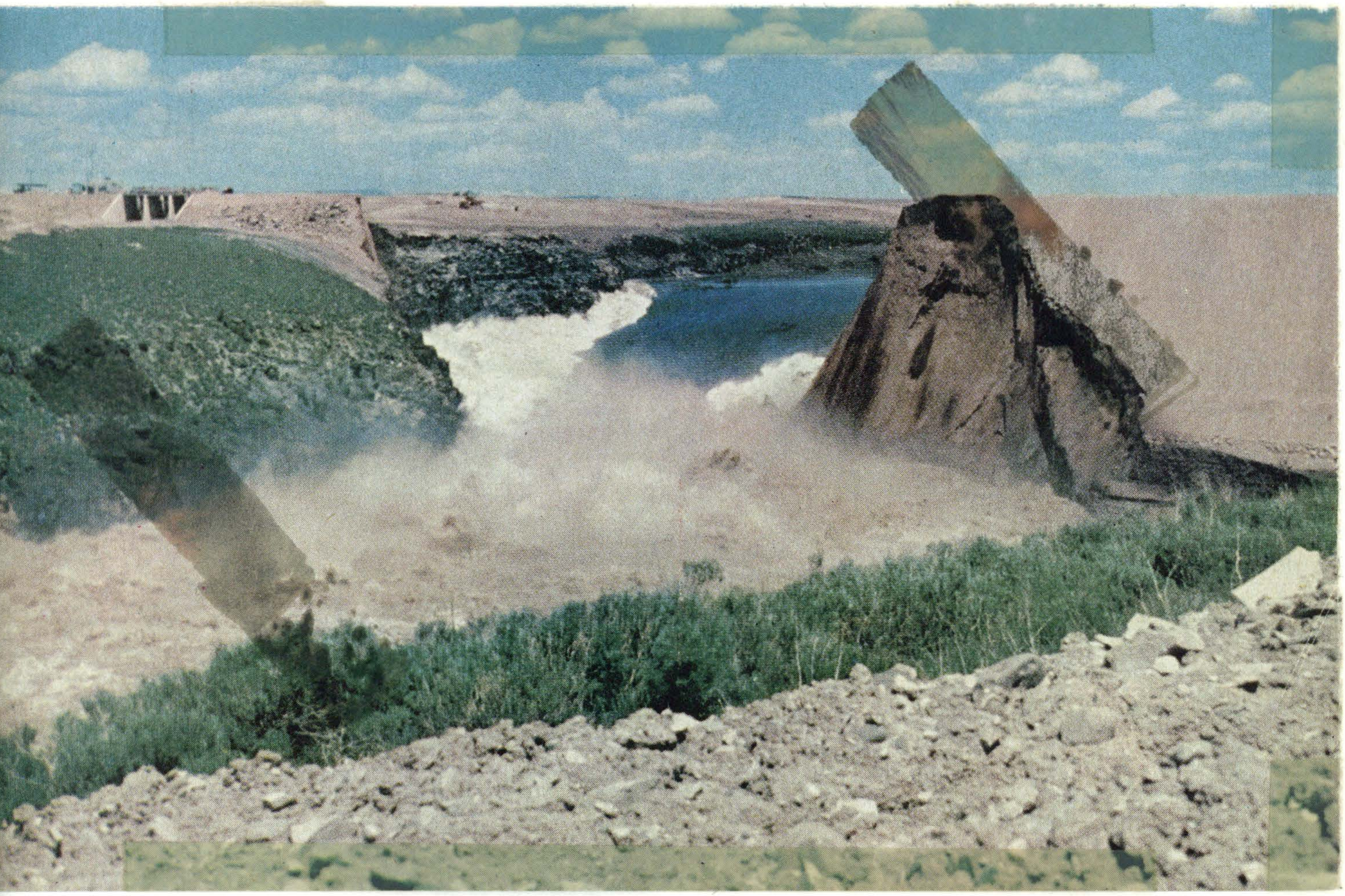
ABSTRACT

Stability of any structure, be it an earth dam, building, bridge or oil platform depends on the structure's material of composition and the foundation on which it rests. This thesis shall endeavor to reveal basic structural deformation in earth dams and earth embankments, as well as research foundation conditions which lead to either successful or nonsuccessful earth dams. The employment of both engineering and geology is necessary for such a study.

Introduction



Teton Dam June 5, 1976

The research of earth dam design, deformation and failure is necessitated by man-made catastrophes as pictured above. The Teton Dam in Idaho failed on June 5th of 1976 killing nine persons and leaving three-thousand homeless. The cost of clean up was over one billion dollars The blame for such a catastrophe may only be placed on society. Yes, the politicians were ignorant and neglectful as were the engineers, but the people elected the politicians and the institutions taught the engineers.

In this thesis I shall attempt to instill a basic understanding of earth dams.

Foundation Introduction

Often the weak point in earth dam construction is the interface between the embankment material and the foundation. Preliminary investigations are generally carried out for proposed dams to ascertain the probability of constructing a safe structure at selected sites and to furnish sufficient data for design.

Geologic structure often promotes foundation failure. Problems may arise from outstanding geologic features such as faulting, jointing, slump, or folded rocks containing weak layers, such as shale or evaporites presenting a potential for foundation failures. Foundation failure may also be a result of more subtle geologic phenomena as porosity, permeability, anisotropy*, settling or even extension fractures in a brittle sandstone bed.

* Anisotropy - Having physical properties that vary in different directions.

Foundation Research

Foundation research may be separated into three phases; reconnaissance, exploratory investigation and detailed investigation. This separation saves time and permits data to be compiled progressively for the planning and design of earth dams.

Reconnaissance work involves researching the geologic nature of the soil and rock formations and to estimate their structure and composition. These studies may begin with an examination of maps and reports published by the United States and State Geological Surveys. Some features to be examined may include "nature and boundaries of recent deposits by streams, lakes, wind and ice; the character, structure, dip and strike of beds, shape and magnitude of folds; location, dip and flow cleavage; and the direction, extent and width of crevices; the classification of rocks as to age and origin, composition of aggregate and cementing material, and geologic processes which may affect the rock or soil structure; and the relation of these geologic conditions to the permeability of the basin floor and the future stability and permanence of the dam, spillway and other structures."[1]

Exploratory investigation explores the depth, thickness and composition of soils and rock strata. Depth of ground water is noted during this state of the foundation research and estimations for the engineering properties begin. Methods of subsurface exploration include open test pits, bars, or rods driven down through shallow soils to rock surfaces, wash boring well drilling may be employed to obtain data at great depths, core drilling and geophysical prospecting.

[1]Subcommittee on Small Water Storage Projects of the National Resources Committee, Low Dams (Washington, D.C. 1938), p. 11

"Construction of accurate geologic cross-sections across and parallel to the axis of the dam generally require exploratory drilling and geophysical measurements. Where it is presumed that the dam will rest on unconsolidated deposits drive-samples are obtained for testing, and water tests are made to determine permeability. In situations where all or part of the dam will be constructed on bedrock, the foundation is explored by core drilling and water testing. Ordinarily, recovery of drive-samples and rock cores is preceded by geophysical investigation, the results of which are correlated with drill-hole data in construction of geologic cross-sections. However, under appropriate conditions, valuable information may be derived from geophysical logging of the bore holes."[2]

The exploratory or feasibility - state investigations are terminated when it is concluded that the dam and reservoir site is or is not acceptable for construction of the dam.

Assuming that the site is acceptable the detailed investigation continues to secure accurate data on the engineering properties.

"The emphasis is now directed toward obtaining data that will be required for adequate design and close estimation of quantities and construction costs, including a detailed assessment of the quality and quantity of available construction materials"[3].

[2]Ernest E. Wahlstrom, Dams, Dam Foundations and Reservoir Sites, (New York 1974) pp. 191, 192.

[3]Ibid., p. 192

Design of Earth and Rockfill Dams

In the design of an earth dam "the designer must plan a structure which meets basic requirements and which at the same time must be economical. There are so many variables involved that an analytical solution for the best dam exactly suitable to a given site is impossible. Instead the designer must rely on his intuition, experience, and iniative, guided by a scientific analysis of the technical factors where that is possible."[4]

Technical design and discussion of variables relevant to design of earth and rockfill dams may be found extensively covered in the following books:

Earth and Earth Rock Dam by James L. Sherard

Embankment - Dam Engineering by Arthur Casagrande

Earth and Rockfill Dam Engineering by George F. Sowers

Dams, Dam Foundations and Reservoir Sites by Ernest E. Wahlstrom

Embankment Engineering

Materials used in embankments have different compressibility charactistics. Well compacted and well graded materials containing silty and sandy fines are known to be materials of poor compressibility. Optimum water content in fine grained soils will increase the compressibility of that material. As the water content is increased the compressibility of the fine grained material increases. For rockfill compressibility the best materials are low quality rocks, such as weathered micaceous schist. Size and uniformity of such materials are also factors of compressibility. As size and uniformity increases so does the compressibility. Plastic materials such as clay are poor compressible materials because of their rebound characteristics.

[4] George F. Sowers, Earth and Rockfill Dam Engineering, (New York, N.Y., 1962), p. 183.

For embankment engineering one should understand that stress strain curves are not linear, tensil strengths are generally negligible and the compressive strength of the embankment may be of equal magnitude as in calculations. Compressive stresses which lie on parallel planes to tensile stresses often rule out opening of cracks in those locations. If tensile stress is greater than or equal to the compressive stresses then cracks will occur. "For example vertical cracks are frequently observed near the surface of many dam embankments. The maximum depth at which vertical cracks can remain open can be determined from the equation: $\sigma_h = h\gamma - q_u = 0$

Where h = depth at which the cracks are closed

γ = unit weight of fill

q_u = unconfined compressive strength

σ_h = stress in horizontal direction

If $h \geq q_u/\gamma$ then the horizontal stresses are positive and cracks will not develop unless there is local arching. At the crest itself where the vertical stress is zero, horizontal extension strains must be resisted by tensile stresses in the soil or else cracks will open up."[5]

Upon filling an earth dam, deflection occurs to the downstream side because of the water load. $\varepsilon = \frac{d}{2R}$ is an expression used by Casagrande to explain maximum deflection near or at the center of the earth dam.

Where ε = maximum deflection

d = downstream deflection

R = radius of curvature

For most earth dams R will approximately equal L (the length of crest). Therefore $\varepsilon = \frac{d}{2L}$ and the strain which is developed from the water load is compressive in nature; thereby reducing the tendency to crack.

[5]Arthur Casagrande, <u>Embankment Dam Engineering</u> (New York, N.Y., 1973), p.413

Internal Dam Deformation

Internal deformation takes place within dams and embankments from the time of construction and thereafter throughout the history of each dam. Some causes for this deformation are "due to changes in total stresses and pore pressures and due to creep and secondary time effects."[6] Foundation movement along with settlement also plays a part in deformation. The first filling of a reservoir may cause significant movement of the crest, but thereafter deformation will slack off. Some areas of deformation are vertical (settlement), upstream and downstream (normal to the dam axis) and cross-valley (parallel to the dam axis).

Essential to the detection and computation involved in understanding deformation of dams and embankments is the instrumentation. Some instruments and devices used are surface monuments, USBR crossarms, inclinometers, strain meters and horizonal displacement devices. Placement of such instruments is done during construction of the dams. By installation of instruments and devices during construction a complete record of deformation and stress can be utilized for research in this field of work.

Deformation in the Mammoth Pool Dam

In 1958-59 Mammoth Pool Dam on the San Joaquin River in California was the first such project to be undertaken. Mammoth Pool Dam is an earthfill dam which was constructed mainly from a disintegrated granite, medium fine grained and "essentially nonplastic with 0-20% passing the No. 200 sieve and 90-100% passing the No. 4 sieve."[7] To minimize the tendency of any seepage along the contact between the smooth rock abutments and the rolled embankments, concrete cutoff walls were constructed on the upstream side. The crest length of the dam is

[6] Arthur Casagrande, Embankment Dam Engineering (New York, N.Y. 1973), p.366
[7] Ibid., p. 368

about 820 feet and the width from toe to toe is about 1,900 feet. The height of the dam is approximately 320 feet above the original streambed and another 40 feet of excavated material was replaced by material more suited for dam construction.

Average settlement and compression measured a total of 5.3 feet during the construction of Mammoth Pool Dam. Different materials used in the dam show different rates and degrees of deformation of vertical settlements. The 78 foot layer of river sands and gravels only compressed 0.7% of their original thickness (about 6 inches). A relatively large difference from this was the embankment itself which compressed 3.0% of the 300 foot thick fill used. This compression of the lower levels is not surprising when it is understood that approximately 20 tons/ft^2 of pressure is located in the lower reaches of the fill. These kinds of deformation were detected by the crossarms located at various locations on and penetrating the dam.

Inclinometers were the main devices used to measure horizonal movements in the dam during construction. An inclinometer consists of a plastic or aluminum casing which is telescopic. The bottom of the tube is attached into the bedrock base and as construction goes on the tube is raised. The slope of the tube is an essential factor which indicates different horizontal movement at the location. Small valley strains were detected within the Mammoth Pool Dam by the use of inclinometers. The three directions of stress are of major concern in earth dams and embankments. The vertical, stresses normal to the axis of the dam and stresses normal to the axis of the river. "Large extention strains can occur even when all three principal streses are compressive, . . . although large extension strains may develop in the lower portions of embankments, the high confining pressure from the weight of the embankment may result in all principal stresses being compressive, and cracking will not occur unless the effective stresses across the crack is zero or a tension."[8] Such was the case of the Mammoth Pool Dam,

[8]Ibid., p. 371

with a compressive stress of about 20 $tons/ft^2$. The small strains which were detected were practically irrelevent with respect to the vertical movement. Often times a ratio of vertical movement vs. ratio of horizontal movement is concluded, but in the Mammoth Pool Dam the horizontal movements were too little to make any reliable ratio.

The large width of the toe was quite a conservative act in the design of the Mammoth Pool Dam, where the toe width was much greater than the length of the crest. During and after construction the amounts of upstream-downstream deformation were so negligible that they were not even reported, and horizontal movement of the shells were found to be inconsequential.

Small postconstructional movements have been detected and are considered normal in earth dams. Typical for most dams is the decreasing rate of deformation as time goes on, as was the case of the Mammoth Pool Dam.

Deformation in the Oroville Dam

In 1972 the undertaking of the world's highest embankment dam, up to that time was completed on the Feather River in Northern California. The Oroville Dam towered 770 feet with a crest length of 5,600 feet and a toe width of 3,500 feet and was located on a hard amphibolite. Instrumented extensively the dam provided a steady and reliable source of information throughout construction. "Two crossarm devices were installed in the downstream shell and 35 fluid devices were installed at 11 elevations in the core, transition and upstream shell."[9] Sensitivity of these devices provided a good source in the areas of fluid pressures and settlement. The greatest amounts of deformation occurred near the center of the dam and lessened rapidly in the transition zones and shells. Near the center of the core deformation values were found to be in slight excess of 4.5 feet. The deformation of the assumed ridged concrete core was slightly greater than they had calculated, but was still negligible with respect to the

[9] Anonymous, "Movement in Oroville", Am. Soc. C.E. Proc. SM (Sept. 1972) p. 657.

earthfill material. Settlement of the upstream shell was affected by the settlement of the upstream transition zone due to the location and orientation of the core block.

Horizontal movements were measured by installing instruments in the dam during construction and by surface markers. "These devices consist of horizontal conduits containing a number of steel aircraft cables, each of which is attached to an anchor embedded within the embankment. Horizontal displacements are determined by measuring movements of the cables in the instruments housed at the downstream slope."[10] On the upstream side of the embankment the largest movements occurred. Movements within the transition zone exceeded 1.0 feet downstream, movements of the shell on the upstream were in excess of 1.2 feet towards the upstream side.

Stress meters were also employed in the research of the deformation at the Oroville Dam. Forty two of the meters were placed at various locations of the embankment. "The distribution of stresses throughout the dam is shown by the contours of calculated values of maximum principle stress (σ) shown in Fig. 8. The values within the core are only about two-thirds as large as those in the downstream transition and shell at the same elevation, indicating an appreciable degree of low transfer from the relatively softer core to the stiffer transition and shell. In addition, there is a zone of high values of σ in the transition zone above the core block parapet and a zone of low values of σ in the soft zone upstream from the core block, from which it may be concluded that the stiff core block and the adjacent soft zone had a large influence on the stresses in the neighboring portions of the dam. The calculated values of minimum principal stress and maximum shear stress were found to vary in a similar manner.

The severity of the stress conditions may be evaluated by contours of the

[10] Ibid., p. 660

type shown in Fig. 9, which indicate computed percentages of strength mobilized throughout the dam. The zones in which the greatest portion of the shear strength are mobilized are at the downstream edge of the sloping core and within the stress concentration above the upstream edge of the core block. The values of percentage of strength mobilized in these zones are about 80%, which corresponds to a factor of safety against local failure of about 1.25. The factor of safety against overall shear failure is considerably higher."[11]

Case Study of the Waco Dam Failure

The study of deformation in the Waco Dam was one of necessity. A 1,500 foot section of Pepper shale formation, which is a heavily overconsolidated, stiff-fissured clay, gave way under the weight of 700 feet of embankment downstream from the axis of the dam. "The Pepper shale had been geologically uplifted to the surface and was bounded laterally by two faults crossing the axis of the embankment. The slide was confined to the length of the embankment founded on the Pepper shale, and no significant movements were observed beyond the fault boundaries."[12] The conventional unconsolidated undrained triaxial compression test did not reveal any complication in the preconstructional field research involving stress analysis. Stress-strain curves were made up from the data collected in the field. Four curves for Pepper shale were looked at, a horizontal, a 30°, a 45°, and a vertical. These stress-strain curves reveal some reduction of shear strength in the horizontal and the vertical but no reduction of shear strength was detected in the 30° and 45° tests. Therefore, the negligible significance of results from the tests negated any further studies of failure during the construction phase of Waco Dam.

[11] Ibid., p. 661.

[12] Anonymous, "Analysis of Waco Dam Slide", AM. Soc. C.E. Proc. SM 7 (July 1972) p. 869

After the slide, the Pepper shale was studied in more detail. The Pepper shale was found to be "a highly plastic, dark gray compaction shale . . . with a high degree of closely spaced, horizontal fissuring . . . This investigation showed that the undrained shear strength of Pepper shale is highly anisotropic. The strength along a horizontal plane was found to be only about 40% as large as the strength in conventional unconsolidated-undrained compression tests on vertical specimens. While total stress analysis based on isotropic strengths from vertical specimens of Pepper shale indicated stable conditions at the time of the Waco Dam slide, the use of anisotropic strengths produced results in agreement with the observed failure and with the development of a pronounced noncircular failure surface.

The significant influence of the anisotropy of Pepper shale on the failure of Waco Dam shows the importance of considering anisotropy in analysis for the stability of embankments founded on heavily overconsolidated, stiff-fissured clays and clay shales. In addition to anisotropy, several other factors, including specimens size and rate of loading, are known to influence the strength of stiff-fissured clays and clay shales, and their effects should also be considered in investigations of embankment stability on stiff-fissured clays and clay shales."[13] Upon the conclusion of the studies of failure and stress-strain calculations there was a revision in the building specifications and the Waco Dam was completed.

[13] Ibid., pp. 876, 877.

Personal Experience on the Chippewa Watershed Involving Dam VII-C

In May of 1974, I became employed by the United States Department of Agriculture, Soil Conservation Service. I held a position as instrument man on a survey party, working on the Chippewa Watershed Project of Wayne County, Ohio. The project is part of the Muskingum Watershed and was designed to control sheet flooding of the source waters. The Chippewa Watershed Project involved thirty-two miles of channelling on the Chippewa River and eight earth dams. Five of the earth dams were dry dams and would not sustain a reservoir unless in flood condition.

Earth Dam VII-C was nearing completion in the early part of August, 1973 when structural failure was observed. The Report of Investigation of Structural Deficiency Site VII-C Chippewa Watershed, describes the structural failure as observed and understood by the engineers and geologists of the United States Department of Agriculture.

The essence of the failure was a slight rotation and downstream movement of the impact basin and a rise of the end sill.

My job in the summer of 1974 was two-fold. First, I was responsible for surveying and calculating volumes of earth excavated from Dam VII-C. Second, revised plans of VII-C called for moving the principal spillway approximately seven hundred feet along the axis of the old dam. I was responsible for the layout and inspection of Dam VII-C revised. Also employed on the survey crew at that time was Bob First, Gary Bennet and Ralph Bullis. Ralph Bullis was Party Chief and Head Inspector. My employment terminated in late September of 1974 when I returned to school at the University of Akron to finish a program of surveying and constructing and to start a program of geology.

UNITED STATES DEPARTMENT OF AGRICULTURE

Soil Conservation Service

311 Old Federal Building

Columbus, Ohio 43215

August 17, 1973

REPORT OF INVESTIGATION OF STRUCTURAL DEFICIENCY

SITE VII-C CHIPPEWA WATERSHED

WAYNE COUNTY, OHIO

Authority

Robert E. Quilliam, State Conservationist, appointed the investigating committee August 14, 1973, according to the authority of SCS Engineering Memorandum-53 (Rev. 1) dated April 19, 1968.

Committee Members

Kyle L. Moran, Assistant State Conservation Engineer, Columbus, Ohio - Chairman
Leonard L. Myers, Geologist, Columbus, Ohio
Clinton W. Liezert, Civil Engineer, Medina, Ohio
Robert F. Fonner, Geologist, Upper Darby, Pennsylvania
Joseph M. Zurlo, Jr., Civil Engineer, Upper Darby, Pennsylvania

Scope of Investigation

The investigating committee assignment is to inspect the structure, examine the deficiency, gather necessary records and physical data, interviews, photographs and other significant information to locate and describe the nature of deficiency failure and damage. The committee's findings are reported to the Director of the Engineering Division, SCS, Washington, D. C.

Deficiency

Foundation movement under principal spillway causing impact basin to move downstream and principal spillway pipe joint separation.

LOCATION AND DESCRIPTION OF STRUCTURE

Site VII-C of Chippewa Watershed is located approximately five miles northeast of Wooster, Ohio. The structure was designed as Class "B" with a maximum fill height of 25 feet. It is approximately 1300 feet in length and contains 41,000 cubic yards of embankment. It is a single purpose, dry flood control structure.

The principal spillway consists of seven 20-foot lengths of 48-inch diameter RC pipe.

INVESTIGATION OF DEFICIENCY

Movement of the principal spillway conduit was noticed for the first time on Friday, August 10, 1973. At this time the embankment was nearing completion. A survey on August 9, 1973 indicated normal conditions around the impact basin.

The proper authorities were notified. The investigating committee was appointed by Robert E. Quilliam, State Conservationist, on August 14, 1973.

On August 15, 1973, two members of the committee made the initial investigation.

On August 17, 1973, all five members of the team were present at the site for field observation and the recording of much of the data in this report.

DESCRIPTION OF DEFICIENCY

The foundation has moved in the downstream portion of the embankment in the vicinity of the principal spillway. The impact basin has moved downstream approximately 2.4 feet. Pipe section Number 5, starting with pipe section Number 1 at the upstream end, has separated from the other sections. The upstream end of the pipe section has a 1.0-foot joint separation. The downstream end of the section has a 1.5-foot joint separation. The conduit in the vicinity of pipe section 5 has settled approximately 15 inches below the outlet.

The bottom of the newly constructed channel just below the impact basin has heaved.

GENERAL GEOLOGY OF THE SITE

George M. White, "Geology of Wayne County, Ohio," 1947, describes the geologic history of the Little Chippewa Creek valley as a series of glacial advances and retreats during the Wisconsin Stage of the Pleistocene Epoch. The original glacial deposits, assumed to be outwash or kame deposits of layered sand, silt and clay, have been modified by subsequent glacial activity.

A glacial geology map is included as Attachment 2.

The low terraces on the valley abutments contain a mixture of partly stratified silt, sand and clay, with local pockets of sand, gravel and heterogenous glacial tills. The recent alluvium in the flood plain blankets a complex variety of glacial deposits ranging from admixtures of silts, sands and gravels to thick deposits of lacustrine silt and clay with some organics.

Evaluation and interpretation of data from bore holes, road cuts, surface exposures and aerial photography plus thorough research of other work done in this area leads to the conclusion that correlation of deposits is very difficult over any significant distance, especially in the valley bottoms which are suspected of having been reworked and reformed to an undetermined extent and number of times.

The detailed geology is contained in site investigation reports of 12/5/62 by Lester J. Matthes, and 10/8/70 by Henry H. Fisher, SCS geologists. (See Geology Section of Attachment 12.)

CONSTRUCTION HISTORY

The dam is being built under a federally assisted contract by H. B. Lockhart, Construction Company, Akron, Ohio. The contract date was July 31, 1972, for $188,006.03. The contract is for two structures. Structure IV-A has been completed. The original contract price for VII-C, the structure in question, was $112,159.19. The embankment is approximately 1300 feet in length. The maximum fill height is 25 feet from creek bottom. The fill height from creek bank is approximately 18 feet. The embankment contains approximately 41,000 cubic yards of earthfill. The following data was taken from the Inspector's Diary.

> "The first equipment was moved on the VII-C site July 21, 1972. Core trench excavation started August 30, 1972. R. C. conduit delivered on site September 18, 1972.

"Foundation drain started same day. Impact basin excavation started September 20, 1972. At required depth the material was soft gray silt. Overexcavated 1.0 ft. and placed layer of #46 material (AASHO Coarse Aggregate). Approximately 100 ft. of principal spillway dug September 22, 1972. Silty material on bottom. Seems to firm up when exposed to air and dried somewhat. First pour of concrete on impact basin October 5, 1972.

"First pipe length of principal spillway laid October 10, 1972. Two inches of concrete placed under floor of impact basin for working purposes. Completed laying of pipe October 12, 1972. Twenty-seven yards of concrete poured in impact basin October 18, 1972. Concrete bedding under pipe poured on October 20, 1972. Impact basin completed November 10, 1972. Backfilling along principal spillway November 22, 1972. Placing drain fill around impact basin. Winter shutdown Friday, December 1, 1972.

"Started cleaning up site June 13, 1973. Riprap placed in outlet June 14, 1973. Outlet excavation being carried on. Old channel excavation completed and backfill and embankment started June 29, 1973. Embankment getting a good start July 13, 1973.

"Structural failure discovered August 10, 1973. At this point the fill was being topped out."

The embankment was placed in approximately 28 days using three Terex T-18 twin engine drive earthmovers.

OBSERVATIONS AND FACTS RELATING TO STRUCTURAL DEFICIENCY

The photos included in this report were taken by Robert Dush, District Conservationist at Wooster, on August 15, 1973. The statements of people on the job were taken at this time. A survey grid was set up on 10 ft. centers starting from fill centerline and extending down below impact basin and to 50 ft. either side of the principal spillway. The purpose is to have survey data available for future use. Sheets 1 through 6 of Attachment 10 are the survey data of the area on August 20, 1973.

Unless stated otherwise, all observations will be referenced looking downstream. The pipe sections and joints are numbered from the riser (see Attachment 7).

Upon inspecting the site in question, it was observed that the riprap downstream from the impact basin had been disturbed. Photo numbers 1 through 4 and Sheet 8 of the plans show the general nature and location of this disturbance. The riprap has heaved approximately 1 to 1-1/2 feet.

The impact basin has moved downstream, the end sill raised, and the basin rotated slightly to the right. See Attachment 9 for actual measurements along centerline of impact basin. See Project Engineer's statement for rotation of end sill.

There was no observed cracking of the embankment or in the soil surrounding open pipe joints. There are cracks on the slope below the berm and to the right of the impact basin. See Photo No. 5 and Sheet 8 of Construction Drawings.

There has been no observed settlement of the fill. To check for future movement of the fill a survey has been made of the areas as previously stated.

No separation was noted between impact basin and surrounding soil.

No cracks were observed in the impact basin structure.

No seepage was occurring from the 6-inch foundation drain located in the side walls of the basin 4.1 ft. from bottom of basin. Water movement was observed from around the wing walls. It was assumed to be coming from the foundation drain.

There is no apparent movement at the riser.

The 48-inch R.C. pipe spillway consisted of 7-20 ft. sections. Attachment 7 shows the movement that has taken place in the conduit. Pipe section 5 is completely detached. There is a gap of 1.0 ft. at the upper end and 1.5 ft. at the lower end. There is no apparent lateral movement in the conduit.

To date there has been very little loss of embankment material through the open joints. During construction a sheet metal shield 12 inches wide was placed over each joint prior to backfilling. This is acting as a partial support for the embankment at the open joints. There is some indication that saturated material beside and under the pipe is slowly moving into the conduit through the open joints. Pieces of broken concrete were found under pipe joint 4.

The fill above the joints is mixed gray, brown yellow clay with some sand and small gravel, firm moist, moderately plastic. Soil under joints is a dark gray silty clay, very soft, wet and plastic. This material was hand probed to a depth of 6.0 feet with a ruler. No sand or gravel was encountered during probling. It resembles stiff grease. This soil appears similar to sample Nos. 301.A.3 and 302.A.7, both at 10- to 12-foot depths, tested at the SCS Soil Mechanics Laboratory. Both samples had natural moisture contents greater than their liquid limits and more than 90 percent less than the No. 200 sieve. The liquidity index for 301.A.3 is 1.42 and 302.A.7 is 1.73.

The Soil Mechanics Laboratory report (3/12/71) places "B" location of principal spillway at approximately centerline station 17+40. The construction drawings show "A" location at that station and "B" at location of constructed principal spillway. (See Geology Section of Attachment 12.)

Hand shelby tube samples have been taken under the conduit at the open joints. These have been sealed and are being preserved for future use if needed.

EMERGENCY MEASURES

To prevent the storage of water, should the conduit fail to function, the following measures have been initiated:

1. The bypass channel has been excavated through the emergency spillway. It has 1.5:1 side slopes and a 30-foot bottom. It will control the flow to elevation 991.0 which is approximately the elevation of the flood plain.

2. A 30-inch corrugated steel pipe is to be inserted in the 48-inch conduit. This is to pass low flow and to prevent flow of water over the open joints.

A copy of the plan (Attachment 11) for the above measures is included. The cost for these measures is estimated to be $15,000.

Respectfully submitted

Kyle L. Moran

Kyle L. Moran, Assistant State Conservation Engineer

9/12/73

Date

Leonard L. Myers

Leonard L. Myers, Geologist

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Date

Clinton W. Liezert

Clinton W. Liezert, Civil Engineer

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Robert F. Fonner

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Joseph M. Zurlo

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Date

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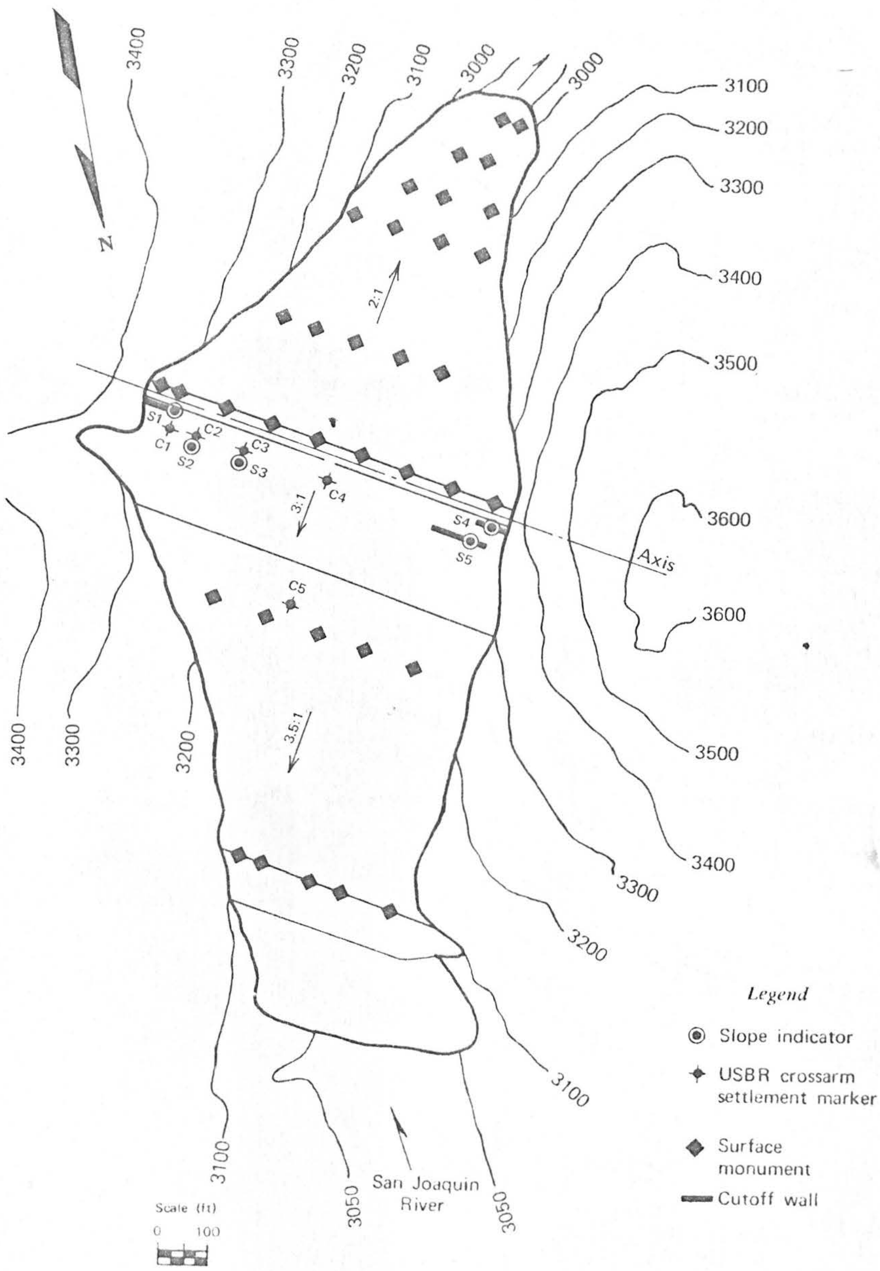

Fig. 2. Plan of Mammoth Pool Dam showing location of instrumentation.

[14]Arthur Casagrande, <u>Embankment Dam Engineering</u> (New York, N Y.,1973), p. 370

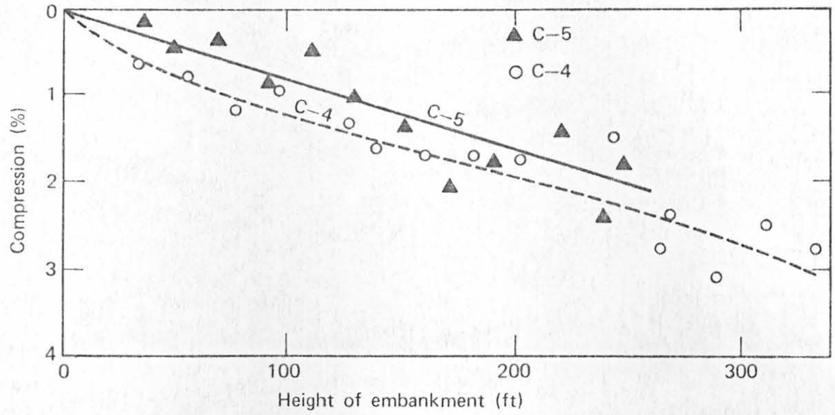

Fig. 5. Compression of embankment, Mammoth Pool Dam. (After Bechtel, 1960.)

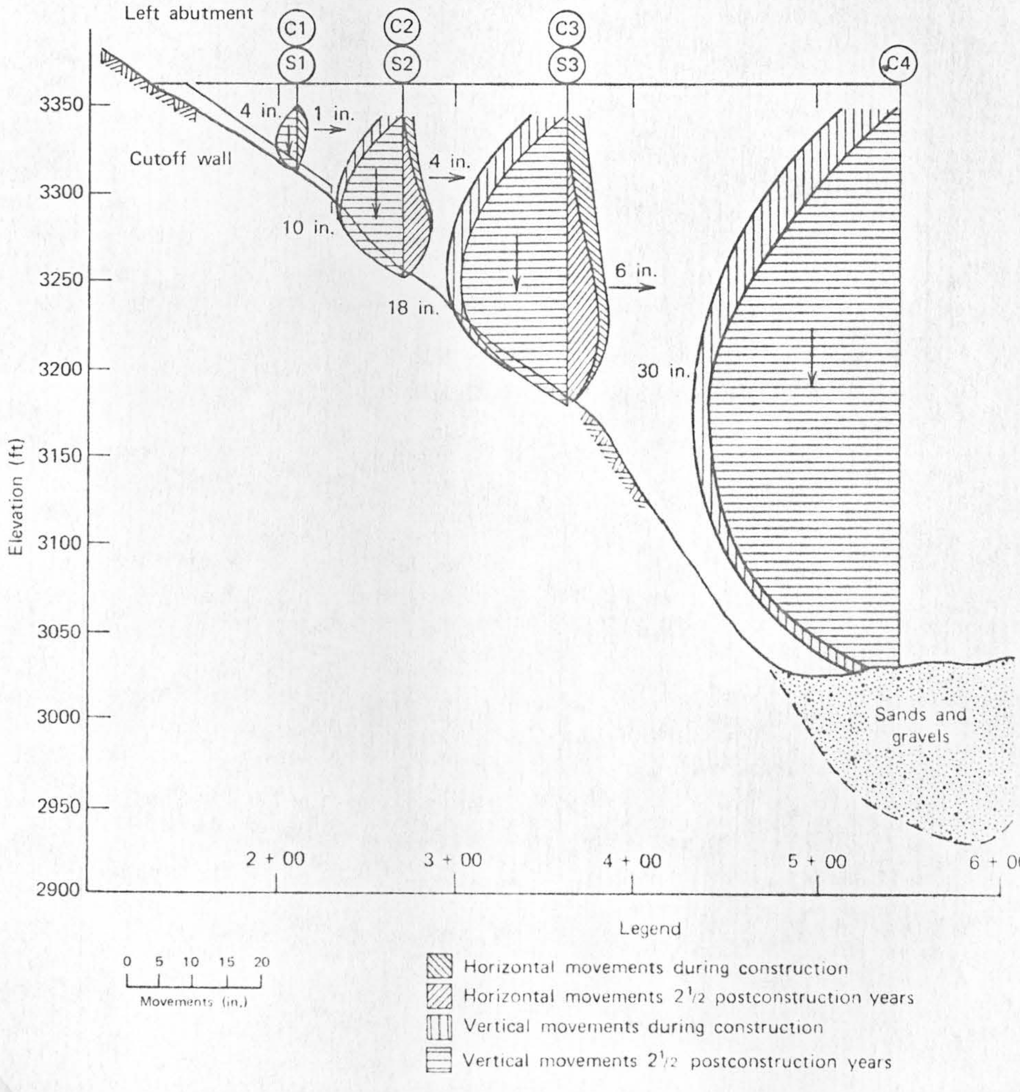

Fig. 6. Horizontal and vertical movements, Mammoth Pool Dam.

[15] Arthur Casagrande Embankment Dam Engineering (New York, N.Y. 1973), p. 372

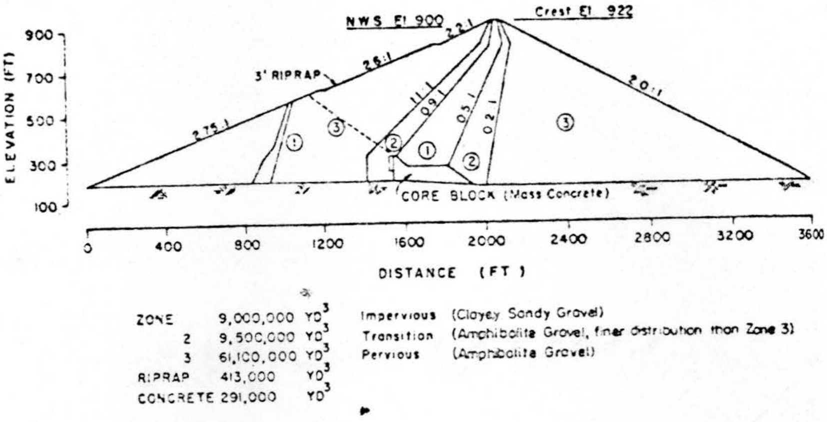

FIG. 1.—OROVILLE DAM MAXIMUM SECTION

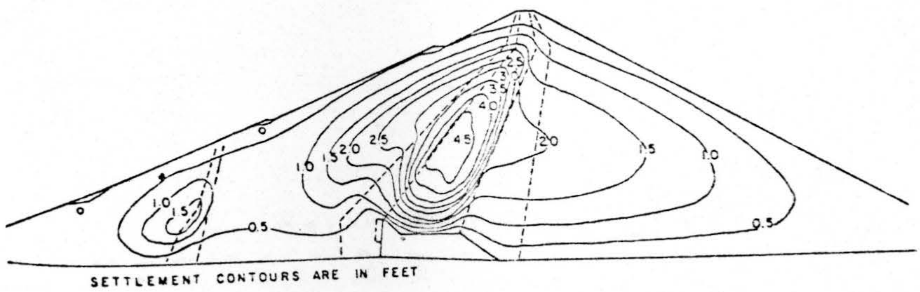

FIG. 5.—CONTOURS OF CALCULATED SETTLEMENT IN OROVILLE DAM

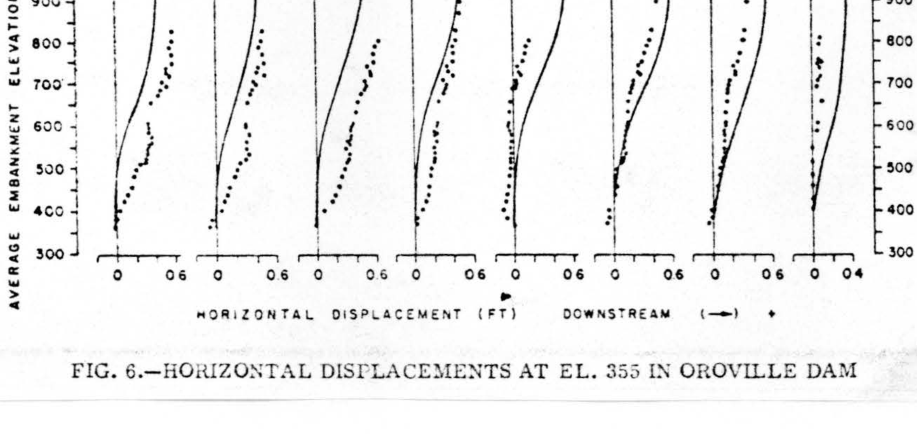

FIG. 6.—HORIZONTAL DISPLACEMENTS AT EL. 355 IN OROVILLE DAM

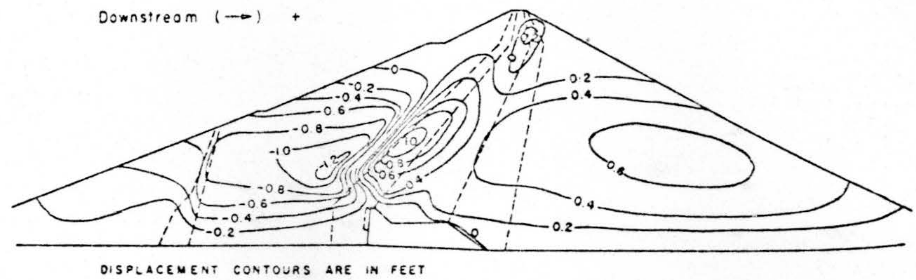

FIG. 7.—CONTOURS OF CALCULATED HORIZONTAL DISPLACEMENT IN OROVILLE DAM

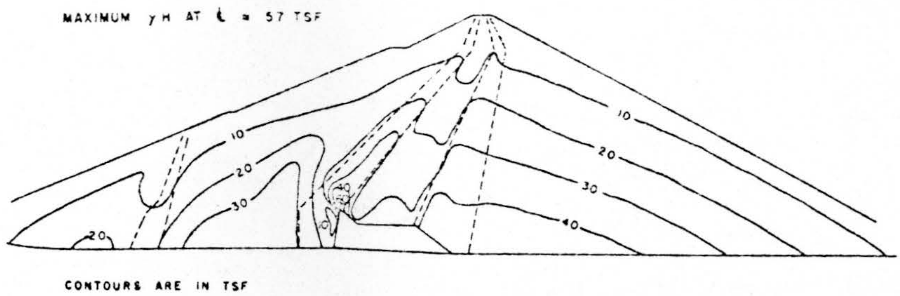

FIG. 8.—CONTOURS OF CALCULATED MAXIMUM PRINCIPAL STRESS IN OROVILLE DAM

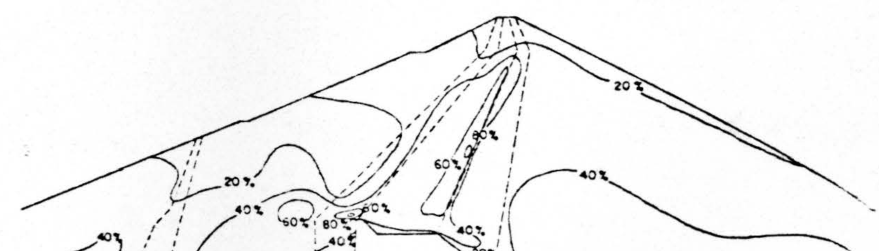

FIG. 9.—CONTOURS OF PERCENTAGE OF STRENGTH MOBILIZED IN OROVILLE DAM

[16] Anonymous,"Movement in Oroville Dam", Am. Soc. C.E. Proc. SM 9 (Sept. 1972) p. 654, 657, 658, 659

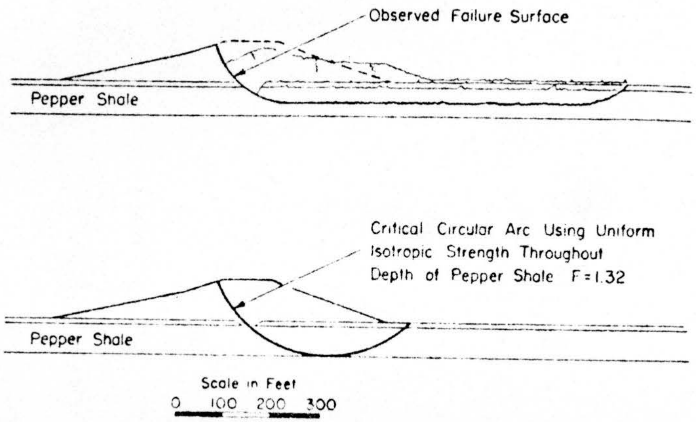

FIG. 5.—FAILURE SURFACES FOR WACO DAM

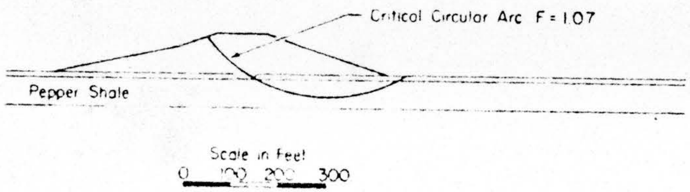

FIG. 6.—ANALYSIS USING UNIFORM ANISOTROPIC STRENGTHS THROUGHOUT DEPTH OF PEPPER SHALE

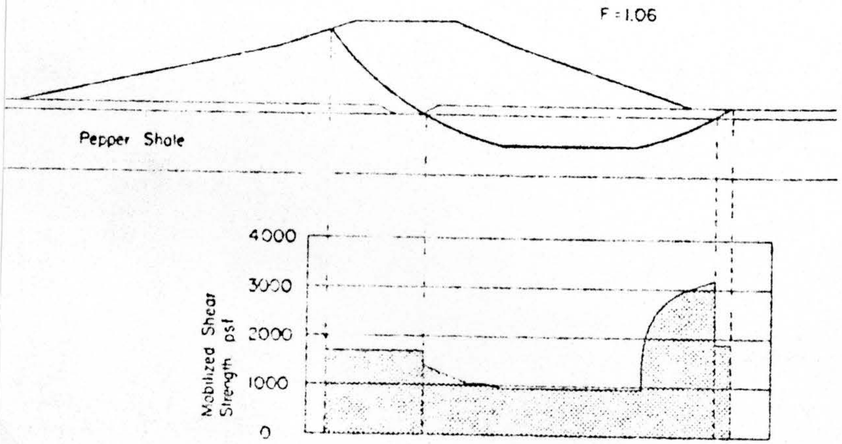

FIG. 7.—MOBILIZED SHEAR STRENGTH ALONG SHEAR SURFACE

uniform throughout the depth of the Pepper shale, the computed factor of safety for the most critical circular arc was 1.07 Thus, considering the anisotropic shear strength of the Pepper shale, the factor of safety was found

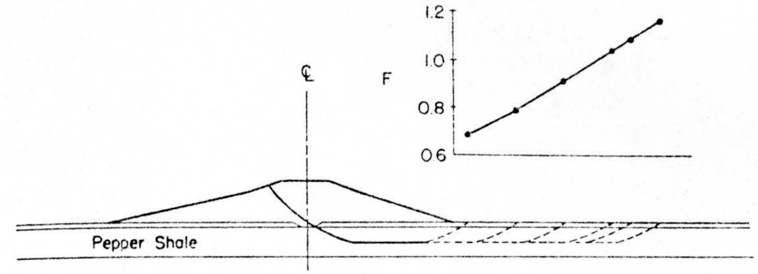

FIG. 8.—INFLUENCE OF EXTENSION OF FAILURE SURFACE DOWNSTREAM TO ASSUMED WEAK ZONE

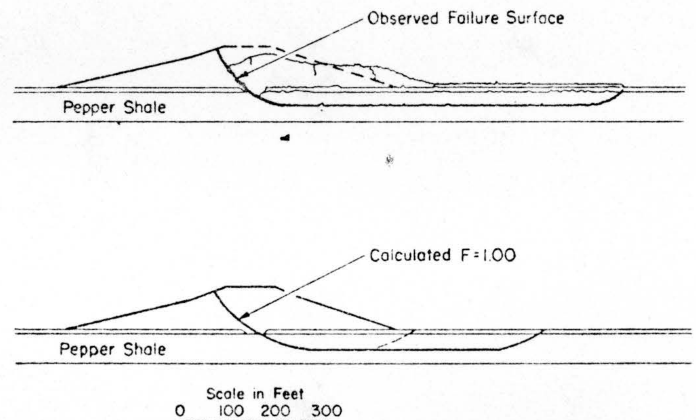

FIG. 9.—ANALYSIS WITH REDUCED SHEAR STRENGTH DOWNSTREAM

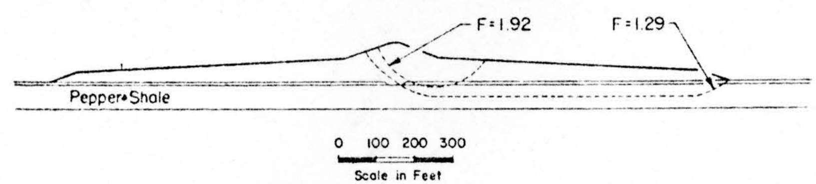

FIG. 10.—ANALYSES OF RECONSTRUCTED SECTION OF WACO DAM

to be considerably lower, and in much better agreement with the occurence of the slide For practical purposes, considering the amount of scatter in the strength data, a calculated value of the factor of safety equal to 1.07 is considered to be consistent with the fact that failure did occur The critical

[17] Anonymous, "Analysis of Waco Dam Slide", Am. Soc. C.E. Proc. SM7 (July 1972) p. 874-875