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A Design of Los Esteros Flood Control Dam

Constantine G. Hadjidakis

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DATE

A DESIGN OF LOS ESTEROS

FLOOD CONTROL DAM

By

Constantine G. Hadjidakis

A Thesis

Submitted in Partial Fulfillment of the

Requirements for the Degree of Master

of Science in Civil Engineering

The University of New Mexico

1957



A DEGREE OF DOCTOR OF PHILOSOPHY
GRANTED TO THE

BY

COMMISSIONER OF THE UNIVERSITY OF CALIFORNIA



A THESIS

Submitted in Partial Fulfillment of the

Requirements for the Degree of

of Science in Civil Engineering

BOND

U.S.A.

FOR THE

The University of California

This thesis, directed and approved by the candidate's committee, has been accepted by the Graduate Committee of the University of New Mexico in partial fulfillment of the requirements for the degree of

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The author wishes to express his appreciation to Messrs. Eugene Hayes, Charles Johnson, and John Smith of the Alameda District, U. S. Coast and Geodetic Survey, for their valuable assistance and advice.

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CHAPTER I

INTRODUCTION

This thesis covers a preliminary design of the Los Esteros Dam based on limited subsurface investigations and laboratory testing. This earth-fill dam will be 222 feet high and will have a top width of 20 feet.

The zoning of the embankment was based on the availability of pervious and impervious materials. Zoning is defined as the distribution of the borrow materials according to their physical characteristics and properties. The best zoning of earth-fill dams is the one in which a central impervious core is flanked by shells of materials considerably more pervious. The shells enclose and protect the core; the shell affords stability against sudden drawdown and the downstream shell acts as a drain that controls the line of seepage. In this design it was decided that the above arrangement of materials would be the most economical.

The height of the embankment was based on the

INTRODUCTION

GENERAL STATEMENT

This report covers a preliminary study of the
Los Batanes Dam based on limited hydrographic
data and topographic maps. This study will
will be 225 feet wide and will have a crest width of
20 feet.
The design of the dam is based on the
availability of materials and the
Soiling is defined as the distribution of the
materials according to their physical
and properties. The dam is not to be
the one in which a certain amount of
by shells of materials considered as
shells and the process of
stability against sliding and
shell acts as a strain that
In this design it was decided that the
of materials would be the same as
The design of the dam is based on the

capacity and elevation of the spillway. One frequent cause of the failure of earth dams is the use of spillways of insufficient capacity. After the greatest flood to be expected has been determined, the spillway should be designed to take such a flood with a fair factor of safety. Freeboard is the vertical distance from the reservoir surface to the top of the dam at the time that the spillway is discharging the greatest flood to be expected. The U. S. Corps of Engineers requires a minimum of 5 feet of freeboard.

The development of soil mechanics has profoundly affected the design of earth dams, levees, and embankments. In addition, the advancement of earth moving equipment such as sheep's-foot rollers, pneumatic tired rollers and other specialized equipment has made possible the construction of high earth embankments at economical costs. These large earth structures have introduced many new problems which can be solved with the advanced theories of soil mechanics.

The application of the fundamental principles of mechanics to the common materials of construction is far from new. Analysis of steel members to determine stresses and laboratory testing to determine strengths have long been common practice. Rational design methods that are

capacity and elevation of the reservoirs and the
basins of the tributaries of the river. The
appliance of the various types of machinery and
flood to be expected has been determined. The
should be designed to take care of the
factor of safety. The design of the
from the reservoir and the basins of the river is the
time that the water will take to reach the
to be expected. The U.S. Corps of Engineers
a minimum of 1 foot of water.

The design of the various types of machinery
affected the design of the various types of
basins. In addition, the design of the
equipment such as water wheels, turbines, and
rollers and other machinery which are used in
the construction of the various types of
coasts. These include the design of the
many new problems which are involved with the
theories of soil mechanics.

The application of the theories of
mechanics to the various types of construction is the
first new. Analysis of the various types of
and laboratory testing to determine the behavior of
been common practice. The various types of

widely used are based on the principle of choosing each member so that its strength shall be greater than its stress by reasonable margin of safety. Except for simple cases, such studies are statically indeterminate; that is, their solution requires the use of moduli of elasticity and other elastic properties. Such studies of relationships between stress, strain and strength constitute an important part of applied mechanics.¹

Soil mechanics is the science that deals with the physical and mechanical properties of soils. Every structure small or large rests on soil. It is therefore important that every civil engineer has a basic understanding of these fundamental properties.

Man's first contact with soil as an engineering material is lost in antiquity, along with the origin of mankind itself; but it is certain that he encountered problems in soil engineering very early. We know that excellent paved highways existed in Egypt several thousand years before the Christian era and were used by the pyramid builders for transportation of the construction materials for those huge structures. Remnants of various types of underground conduits which served

¹Donald W. Taylor, Fundamentals of Soil Mechanics, (New York: John Wiley and Sons, 1948), p. 3.

rigidly used are based on the principle of conservation of energy
member so that the system will be in equilibrium
stress by reasoning in terms of energy. In fact, the energy
cases, each system is associated with a certain amount of energy
their solution requires the use of a certain amount of energy
and other elastic properties. The amount of energy stored in
ships between stress, strain and energy is of importance in
important part of physical mechanics.
Self mechanism is one system. The energy stored in the
physical and mechanical properties of a system. The energy
structure will be large in some cases. The energy stored in
important that every civil engineer should have a certain amount of
standing of these fundamental principles.
Man's first attempt at a machine was the lever. The lever is a
material is less in a lever. The lever is a simple machine.
marked itself. But in the design of a machine, the engineer must
problems in the design of a machine. The engineer must have a
excellent understanding of the principles of mechanics.
thousand years before the lever was used. The lever is a simple
the pyramid, which is a simple machine. The pyramid is a simple
structure which is a simple machine. The pyramid is a simple
of various types of structures. The pyramid is a simple machine.

Donald W. Davis, Professor of Mechanical Engineering
(New York, New York, 1911)

BOND

ancient people as drains, tunnels, aqueducts, and many other kinds of structures which involved the application of principles of mechanics to the soil, have been unearthed at the sites of early civilizations. It is certain from these evidences that engineers in ancient times encountered and solved in some manner many problems in soil engineering, even though we have no record of the methods by which they obtained their results.²

In recent years due to the ability and persistence of Dr. Karl Terzaghi, soil mechanics earned recognition as a distinct branch of civil engineering. His greatest contribution has been the consolidation theory and its practical applications.

²Merlin Grant Spangler, Soil Engineering, (Scranton: International Textbook Co., 1951), p. 2.

ancient people as well as modern people
other kinds of things, and in the same way
of principles of psychology, and in the same way
unrelated to the study of psychology, and in the same way
certain from the study of psychology, and in the same way
times encountered and solved in the same way
in self-education, and in the same way
methods by which they can be applied
In recent years, due to the study of psychology
of Dr. Karl Jaspers, and in the same way
as a distinct branch of psychology, and in the same way
contribution has been made to the study of psychology
practical application.

CHAPTER II

DESCRIPTION OF THE STRUCTURE

The Los Esteros damsite is located on the Pecos River in Guadalupe County, New Mexico, approximately seven miles north of Santa Rosa, New Mexico.

The embankment will consist of a compacted earth and pervious section 3,000 feet in length; its maximum height at the river channel section will be 222 feet. The top of the dam will be at elevation 4837 feet (mean sea level), which will provide 5 feet of freeboard. It is anticipated that the dam would be subjected to flood pools of short duration. The project layout is shown on Plate No. 7 and typical embankment section on Plate No. 8, Appendix B.

The embankment will have a top width of 20 feet which will be surfaced with gravel to serve as a maintenance road. The embankment section will be composed of a central core of impervious rolled fill, flanked upstream and downstream by relatively pervious sections. Embankment slopes will be 1 vertical to 2.5 horizontal to

APPENDIX

WISCONSIN

The Wisconsin Division is located at 1000 Wisconsin

River in Oshkosh County, Wisconsin, near Oshkosh.

Seven miles north of Oshkosh, Wisconsin.

The embankment will consist of a concrete

and pervious wall 10 to 12 feet high and 10 feet

height at the river channel, located at the river

The top of the dam will be at an elevation of 100 feet

sea level, which will provide a freeboard of 10 feet

is anticipated that the dam will be a concrete dam

pool of about 500,000 cubic feet of water.

Place No. 7 and typical cross section of the dam.

Appendix B

The embankment will have a top width of 10 feet

which will be surfaced with gravel or crushed stone

face road. The embankment section will be as shown on

a central core of gravel and crushed stone, flanked by

and down-slopes by relatively heavy vegetation.

ment slopes will be 1:1 vertical to 1:1 horizontal.

elevation 4760 feet, and 1 vertical to 3.0 horizontal to elevation 4615 feet upstream and downstream.

A 6 foot riprap blanket will protect the upstream slopes from wave action and floating debris. The downstream side will be covered with a 3 foot rock blanket for protection against weathering. A cut-off trench will be provided over the full length of the dam to prevent seepage action under the embankment. Where the cut-off contacts rock, the rock will be grouted to a 40 foot depth with grout holes spaced 10 feet on centers. Both abutments will be gunited to minimize seepage along the contact of the embankment and abutment.

A rock-fill trench at the downstream toe will prevent saturation. Rock toes for dams have been constructed of stones ranging in size from gravel up to one or two man stones and in some cases derrick stone. The principal functions of such toes are to furnish drainage and protect the lower part of the downstream slope from all tail-water erosion. The size of rock toes are based upon requirements for seepage relief, and the height to which slope protection must be carried. Filters between rock toes and soils in both embankments and foundations are required. Omission of this important item may lead to the failure of the rock toe to function

elevation 1750 feet, and 1750 feet, and 1750 feet.

elevation 1750 feet, and 1750 feet, and 1750 feet.

A 4 foot wide ditch, 4 foot wide, 4 foot wide.

slopes from wave action, 4 foot wide, 4 foot wide.

downstream side will be covered, 4 foot wide, 4 foot wide.

blanket for protection against wave action, 4 foot wide, 4 foot wide.

trench will be provided over the full length of the bank, 4 foot wide, 4 foot wide.

to prevent leakage of water from the ditch, 4 foot wide, 4 foot wide.

the out-of-the-bank side, the road will be covered, 4 foot wide, 4 foot wide.

a 40 foot deep ditch, 4 foot wide, 4 foot wide.

center. Both segments will be covered, 4 foot wide, 4 foot wide.

escape along the length of the ditch, 4 foot wide, 4 foot wide.

A rock fill trench, 4 foot wide, 4 foot wide.

prevent infiltration, 4 foot wide, 4 foot wide.

constructed of concrete, 4 foot wide, 4 foot wide.

one or two segments, 4 foot wide, 4 foot wide.

The principal drainage, 4 foot wide, 4 foot wide.

drainage and prevent the flow of water, 4 foot wide, 4 foot wide.

slope from all sides, 4 foot wide, 4 foot wide.

are based upon the assumption that the water, 4 foot wide, 4 foot wide.

height to which the water, 4 foot wide, 4 foot wide.

between the water, 4 foot wide, 4 foot wide.

foundations are, 4 foot wide, 4 foot wide.

item may be, 4 foot wide, 4 foot wide.

as a relief outlet for seepage. So long as the pervious material of the foundation contains the necessary range of sizes it will make its own filter if thick enough.

When placed next to stone the smaller particles of the pervious material will perhaps run right through the voids of the stone in the trench, but the larger gravel will stick and a filter will thus be gradually built up.

Because the alluvial material of the Los Esteros foundation met all the requirements of a filter, it was decided that the necessity of a separate filter was not justified.

as a relief on the 10th of August, 1944, to the 1st of September, 1944.
Material of the 10th of August, 1944, to the 1st of September, 1944,
of which it was a part, was placed in the 1st of September, 1944.
When placed in the 1st of September, 1944, the material was placed in the
previous material with the 1st of September, 1944, to the 1st of September, 1944.
of the 1st of September, 1944, to the 1st of September, 1944, to the 1st of September, 1944.
stick and a stick, which was placed in the 1st of September, 1944, to the 1st of September, 1944.
Because the stick was placed in the 1st of September, 1944, to the 1st of September, 1944,
action and all the material of the 1st of September, 1944, to the 1st of September, 1944,
decided that the material of the 1st of September, 1944, to the 1st of September, 1944,
justified.

CHAPTER III

GEOLOGY

The Pecos River at the damsite forms a narrow channel with fairly steep slopes on the sides. The canyon is approximately 225 feet deep and varies in width from about 150 feet at the bottom to about 3,000 feet at the top.

The bottom of the Los Esteros canyon consists of sand and gravel varying from a few feet to about 30 feet in thickness. Under this alluvium and on the abutments is the Santa Rosa sandstone.

The Santa Rosa sandstone has been divided into the following members:

- (1) Upper sandstone member.
- (2) Shale member.
- (3) Middle sandstone member.
- (4) Lower sandstone member.

The upper sandstone member is a brown to a gray, dense, fine grained, platy to massive sandstone that weathers into rounded surface, with ribbed cup holes.

CHARTER 111

Geology

The Pecora River at the Kanabite section is a

channel with fairly steep slopes on the sides. The

canyon is approximately 100 feet wide at the top and

widens from about 150 feet at the bottom to about 200

feet at the top.

The bottom of the Pecora River canyon consists of

sand and gravel varying from a few feet to about 10

feet in thickness. Under this is a layer of gravel

abundant in the Santa Rosa formation.

The Santa Rosa formation has been divided into the

following members:

(1) Upper sandstone member

(2) Shale member

(3) Middle sandstone member

(4) Lower sandstone member

The upper sandstone member is a brown to gray

dense, fine grained, glass to medium sandstone

weathering into rounded pebbles. It is typical of the

The shale member is red to gray that frequently is arenaceous in its basal portion. The thickness of the shale member is extremely variable. It has a maximum thickness of 50 feet but is absent locally. The middle sandstone member is a gray to brown, medium to coarse grained, platy to massive-bedded sandstone that weathers into irregular blocks. The lower sandstone member is friable, purplish red "salt and pepper" textured, fine grained, platy to thin-bedded, micaceous silty sandstone. It weathers into rounded and exfoliated boulder like ledges, and occasionally contains bone fragments. Chocolate red shale occurs in the upper part of this member west and north of Esterito Dome. It has a maximum thickness of 112 feet. The Santa Rosa sandstone varies in thickness from about 1,000 to 1,500 feet over most of its extent in the Pecos Basin.³

At low stages the Pecos River loses its entire flow between Delia and Colonias. A determination of the losses between Anto Chico and Colonias shows that a flow of more than 50 cubic feet per second at Anton Chico is necessary to pass any water beyond the leakage zone to

³Joseph M. Gorman and Raymond C. Robeck, Geology and Asphalt Deposits of North-Central Guadalupe County, New Mexico, Oil and Gas Investigations Map.

Colonias. The river begins to flow again about 5 miles below Colonias and at Santa Rosa it has a base flow of about 20 cubic feet per second. In the vicinity of Santa Rosa and between Santa Rosa and Puerto de Luna, numerous springs discharge into the Pecos River causing a marked increase in the flow of the river. Many of the springs in the vicinity of Santa Rosa on both sides of the river issue from sink holes.⁴ No tests have been made to determine the loss of flow, or to locate the troublesome spots that will cause the above condition at the damsite. However, tests will be made by the Albuquerque District, U. S. Corps of Engineers, before the actual design of the dam.

The presence of sink holes and perennial springs indicate that loss of water will occur above and below the dam. Stripping of the overburden material and a grout curtain extending 40 feet into rock below the cut-off would eliminate most of the leakage and prevent any piping action at the damsite.

The foundation of the damsite as a whole is very good and should present no serious problems. Excavation

⁴The Pecos River Joint Investigation Report, National Resources Planning Board, (Washington: U. S. Government Printing Office, 1942), p. 39.

to sound rock will be from a few feet to as much as 30 feet with a conservative average of about 20 feet.

to some extent, it is the same as the first, but it is
less with a considerable increase in the amount of the

CHAPTER IV

CONSTRUCTION MATERIALS

A subsurface investigation was performed by the Albuquerque District, U. S. Corps of Engineers, to determine the amount and type of borrow materials, that can be used in the embankment. It consisted of auger holes and core borings through the soil overburden in locations as shown on Plate No. 7, Appendix B.

The results of the above field investigations indicate that sufficient impervious and pervious material would be available for the earth embankment within a two miles radius. Suitability of this material was verified by numerous laboratory tests performed on composite samples obtained from the impervious borrow areas "A", "B", and gravel pit.

All samples obtained from the field explorations were classified in accordance with the Corps of Engineers method. Where necessary, routine tests consisting of grain-size analyses and Atterberg limits were performed to establish the classification. Numerous tests of the

above types were performed on selected samples of foundation and borrow materials as an aid in evaluating the materials with respect to permeability and shear strength.

Specific gravities of selected samples were also determined. One shear test was performed on material obtained from the impervious borrow area "B". The impervious borrow material was tested to determine its compaction characteristics by the modified AASHO compaction test. Summaries of laboratory test results are shown on Tables 1, 2, 3, 4, and Plates 11-17, Appendix C.

Large quantities of pit-run sand and gravel are located in the gravel bars investigated. Additional sources of sand and gravel would be available in selected locations off U. S. Highway 66 below Santa Rosa and Tucumcari, New Mexico.

Field investigations also indicate that ample quantities of rock for riprap would be available from the spillway, outlet-tunnel and shaft, and cut-off excavations.

above types were not found on sections exposed in
 foundation and bottom materials in the river channel.
 the materials with respect to composition and
 strength.

Specific gravities of selected samples were
 determined. One sample was selected for analysis
 obtained from the investigation of the river
 impervious bottom material was shown to be
 composition characterized by the following values
 composition test. Comparison of laboratory test results
 are shown on Table A, B, C, and D (pages 11-13).

Appendix C.

Large quantities of gravel and sand are
 located in the gravel bars, particularly in the
 sources of sand and gravel would be available in selected
 locations off U. S. Highway 80 (see map) and
 Technical, New Mexico.

Field investigation also indicates that
 quantities of rock and gravel could be available from the
 highway, gravel pits, and sand, and gravel
 sections.

TABLE 1
SOILS INVESTIGATION
LOS ESTEROS DAM SITE
RESULTS OF CLASSIFICATION TESTS^a
IMPERVIOUS BORROW AREA "A"

| Hole No. | Lab No. | Depth (Ft.) | Gravel (%) | Sand (%) | Fines (%) | L.L. (%) | P.I. (%) | Classification (Corps of Engineers) |
|----------|---------|-------------|------------|----------|-----------|----------|----------|-------------------------------------|
| 13 | 25243 | 0.0--2.5 | 9 | 31 | 60 | 22 | 6 | Sandy Silt (ML) |
| | 25244 | 2.5--6.0 | 10 | 20 | 70 | 24 | 10 | Sandy Clay (CL) |
| | 25245 | 6.0--7.8 | 5 | 10 | 85 | 28 | 13 | Clay (CL) |
| 14 | 25246 | 0.0--2.5 | 7 | 18 | 75 | 29 | 12 | Sandy clay (CL) |
| | 25247 | 2.5--3.5 | 8 | 19 | 73 | 24 | 10 | Sandy Clay (CL) |
| | 25248 | 3.5--6.5 | 3 | 20 | 77 | 21 | 6 | Sandy Silt (ML) |
| | 25249 | 6.5--8.0 | 3 | 7 | 90 | 24 | 10 | Clay (CL) |
| | 25250 | 8.0--9.0 | 2 | 10 | 88 | 25 | 11 | Clay (CL) |
| | 25251 | 9.0--11.5 | 2 | 14 | 84 | 25 | 12 | Clay (CL) |
| | 25252 | 11.5--13.5 | 2 | 3 | 95 | 38 | 20 | Clay (CL) |
| 15 | 25253 | 13.5--16.7 | 6 | 14 | 80 | 30 | 14 | Clay (CL) |
| | 25254 | 0.0--0.5 | 8 | 30 | 62 | 22 | 5 | Sandy Silt (ML) |
| | 25255 | 0.5--2.6 | 2 | 9 | 89 | 31 | 11 | Clay (CL) |
| 16 | 25256 | 0.0--1.0 | 9 | 36 | 55 | 22 | 7 | Sandy Silt (ML) |
| | 25257 | 1.0--4.7 | 4 | 32 | 64 | 24 | 10 | Sandy Clay (CL) |

^aObtained from the U. S. Corps of Engineers, Albuquerque District.

TABLE 1 - Continued

| Hole No. | Lab No. | Depth (Ft.) | Gravel (%) | Sand (%) | Fines (%) | L.L. (%) | P.I. (%) | Classification (Corps of Engineers) |
|----------|---------|-------------|------------|----------|-----------|----------|----------|-------------------------------------|
| 20 | 25258 | 0.0--0.3 | 6 | 47 | 47 | 18 | 1 | Sandy Silt (ML) |
| | 25259 | 0.3--2.5 | 6 | 43 | 51 | 21 | 3 | Sandy Silt (ML) |
| | 25260 | 2.5--3.5 | 8 | 44 | 48 | 19 | 3 | Sandy Silt (ML) |
| 21 | 25261 | 0.0--1.0 | 6 | 32 | 62 | 21 | 5 | Sandy Silt (ML) |
| | 25262 | 1.0--2.2 | 8 | 28 | 64 | 28 | 12 | Sandy Clay (CL) |
| | 25263 | 2.2--3.5 | 6 | 17 | 77 | 26 | 12 | Sandy Clay (CL) |
| | 25264 | 3.5--8.3 | 1 | 17 | 82 | 25 | 12 | Clay (CL) |

TABLE I - Continued

| No. | Ref. | Depth (m) | Level | Base | Spills | 3-1 m.p. | 1-1 m.p. | 1-1 m.p. | Notes or Comments |
|-----|------|-----------|-------|------|--------|-------------|-------------|-------------|-------------------|
| 31 | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |
| | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |
| | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |
| | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |
| 32 | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |
| | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |
| | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |
| | 1956 | 0.1-0.0 | 0 | 0 | 0 | 0 | 0 | 0 | Small spill |

TABLE 2
SOILS INVESTIGATION
LOS ESTEROS DAM SITE
RESULTS OF CLASSIFICATION TESTS^a
IMPERVIOUS BORROW AREA "B"

| Hole No. | Lab No. | Depth (Ft.) | Gravel (%) | Sand (%) | Fines (%) | L.L. (%) | P.I. (%) | Classification (Corps of Engineers) |
|----------|---------|-------------|------------|----------|-----------|----------|----------|-------------------------------------|
| 29 | 25296 | 0.0--2.2 | 17 | 35 | 48 | 24 | 7 | Gravelly Sandy Clay (CL) |
| | 25297 | 2.2--3.5 | 6 | 34 | 60 | 25 | 9 | Sandy Clay (CL) |
| | 25298 | 3.5--4.7 | 8 | 29 | 63 | 27 | 8 | Sandy Clay (CL) |
| 30 | 25299 | 0.0--2.5 | 4 | 24 | 72 | 29 | 13 | Sandy Clay (CL) |
| | 25300 | 2.5--4.7 | 3 | 22 | 75 | 29 | 13 | Sandy Clay (CL) |
| | 25301 | 4.7--6.5 | 4 | 27 | 69 | 27 | 7 | Sandy Clay (CL) |
| | 25302 | 6.5--7.7 | 3 | 22 | 75 | 28 | 12 | Sandy Clay (CL) |
| | 25303 | 7.7--8.5 | 1 | 20 | 79 | 28 | 11 | Sandy Clay (CL) |
| | 25304 | 8.5--10.5 | 1 | 20 | 79 | 28 | 11 | Sandy Clay (CL) |
| | 25305 | 10.5--13.5 | 0 | 21 | 79 | 29 | 9 | Sandy Clay (CL) |
| | 25306 | 13.5--18.0 | 0 | 21 | 79 | 28 | 11 | Sandy Clay (CL) |
| | 25307 | 18.0--20.5 | 0 | 22 | 78 | 32 | 16 | Sandy Clay (CL) |
| | 25308 | 20.5--23.5 | 1 | 13 | 86 | 34 | 16 | Clay (CL) |

^aObtained from the U. S. Corps of Engineers, Albuquerque District.

TABLE 2 - Continued

| Hole No. | Lab No. | Depth (Ft.) | Gravel (%) | Sand (%) | Fines (%) | L.L. (%) | P.I. (%) | Classification (Corps of Engineers) |
|----------|---------|-------------|------------|----------|-----------|----------|----------|-------------------------------------|
| 31 | 25309 | 0.0--3.0 | 4 | 19 | 77 | 33 | 14 | Sandy Clay (CL) |
| | 25310 | 3.0--5.0 | 3 | 20 | 77 | 36 | 18 | Sandy Clay (CL) |
| | 25311 | 5.0--7.0 | 1 | 21 | 78 | 30 | 11 | Sandy Clay (CL) |
| | 25312 | 7.0--8.7 | 2 | 21 | 77 | 32 | 18 | Sandy Clay (CL) |
| | 25313 | 8.7-14.5 | 3 | 43 | 54 | 22 | 9 | Sandy Clay (CL) |
| | 25314 | 14.5-15.5 | 2 | 30 | 68 | 26 | 12 | Sandy Clay (CL) |
| | 25315 | 15.5-16.4 | 2 | 32 | 66 | 25 | 9 | Sandy Clay (CL) |
| 32 | 25316 | 0.0--3.0 | 0 | 12 | 88 | 43 | 22 | Clay (CL) |
| | 25317 | 3.0--6.8 | 1 | 11 | 88 | 38 | 17 | Clay (CL) |
| | 25318 | 6.8-11.0 | 3 | 16 | 81 | 29 | 14 | Clay (CL) |
| | | | | | | | | |
| 33 | 25319 | 0.0--1.0 | 7 | 37 | 56 | 22 | 5 | Sandy Silt (ML) |
| | 25320 | 1.0--3.5 | 1 | 17 | 82 | 45 | 16 | Clay (CL) |
| | 25321 | 3.5--6.5 | 1 | 7 | 92 | 43 | 18 | Clay (CL) |
| | 25322 | 6.5--8.0 | 2 | 11 | 87 | 39 | 17 | Clay (CL) |
| | 25323 | 8.0-10.5 | 5 | 7 | 88 | 31 | 14 | Clay (CL) |
| | 25324 | 10.5-12.5 | 4 | 16 | 80 | 38 | 18 | Clay (CL) |
| | 25325 | 12.5-14.4 | 5 | 16 | 79 | 36 | 17 | Sandy Clay (CL) |
| | 25326 | 14.4-15.0 | 7 | 13 | 80 | 28 | 11 | Sandy Clay (CL) |
| | 25327 | 15.0-17.0 | 6 | 17 | 77 | 30 | 14 | Sandy Clay (CL) |
| | 25328 | 17.0-17.9 | 7 | 23 | 70 | 27 | 10 | Sandy Clay (CL) |
| | 25329 | 17.9-18.8 | 4 | 17 | 79 | 26 | 12 | Sandy Clay (CL) |
| | | | | | | | | |

TABLE 2 - Continued

| Hole No. | Lab No. | Depth (Ft.) | Gravel (%) | Sand (%) | Fines (%) | L.L. (%) | P.I. (%) | Classification (Corps of Engineers) |
|----------|---------|-------------|------------|----------|-----------|----------|----------|-------------------------------------|
| 35 | 25330 | 0.0--1.0 | 4 | 35 | 61 | 25 | 7 | Sandy Clay (CL) |
| | 25331 | 1.0--3.0 | 7 | 29 | 64 | 27 | 8 | Sandy Clay (CL) |
| | 25332 | 3.0--6.5 | 6 | 32 | 62 | 24 | 10 | Sandy Clay (CL) |
| | 25333 | 6.5--10.2 | 2 | 25 | 73 | 26 | 12 | Sandy Clay (CL) |
| 36 | 25334 | 0.0--0.8 | 2 | 28 | 70 | 29 | 8 | Sandy Clay (CL) |
| | 25335 | 0.8--4.0 | 4 | 36 | 60 | 27 | 10 | Sandy Clay (CL) |
| | 25336 | 4.0--8.2 | 1 | 38 | 61 | 26 | 10 | Sandy Clay (CL) |
| | 25337 | 8.2--10.5 | 2 | 29 | 69 | 28 | 13 | Sandy Clay (CL) |
| | 25338 | 10.5--11.8 | 1 | 29 | 70 | 28 | 13 | Sandy Clay (CL) |
| 37 | 25339 | 0.0--0.6 | 12 | 44 | 44 | 17 | 2 | Sandy Silt (ML) |
| | 25340 | 0.6--1.8 | 7 | 35 | 58 | 32 | 11 | Sandy Clay (CL) |
| 38 | 25341 | 0.0--0.8 | 4 | 39 | 57 | 23 | 8 | Sandy Silt (ML) |
| | 25342 | 0.8--3.6 | 6 | 30 | 64 | 26 | 9 | Sandy Clay (CL) |
| | 25343 | 3.6--4.5 | 11 | 31 | 58 | 25 | 10 | Sandy Clay (CL) |
| | 25344 | 4.5--11.5 | 10 | 22 | 68 | 31 | 16 | Sandy Clay (CL) |
| | 25345 | 11.5--13.8 | 3 | 24 | 73 | 35 | 19 | Sandy Clay (CL) |

TABLE 3
SOILS INVESTIGATION
LOS ESTEROS DAM SITE
RESULTS OF CLASSIFICATION TESTS^a
GRAVEL BAR AREA

| Hole No. | Lab No. | Depth (Ft.) | Gravel (%) | Sand (%) | Fines (%) | L.L. (%) | P.I. (%) | Classification (Corps of Engineers) |
|----------|---------|-------------|------------|----------|-----------|----------|----------|-------------------------------------|
| Test Pit | 25346 | 0.0--0.5 | 49 | 47 | 4 | 19 | NP | Sandy Gravel (GW) |
| | 25347 | 0.5--6.5 | 73 | 23 | 4 | 17 | NP | Sandy Gravel (GW) |

| Test Hole | Depth to Shale (Ft.) |
|-----------|----------------------|
| 1 | 3.0 |
| 2 | 3.0 |
| 3 | 2.5 |
| 4 | 4.5 |

^aObtained from the U. S. Corps of Engineers, Albuquerque District.

MESSAGE OF CERTIFICATION AND
 FOR REMITTANCE OF BILL
 TO THE INVESTIGATION
 OF THE

| DATE | TIME | PLACE | REMARKS | REMARKS | REMARKS | REMARKS | REMARKS | REMARKS | REMARKS |
|------|------|-------|---------|---------|---------|---------|---------|---------|---------|
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) |
| (11) | (12) | (13) | (14) | (15) | (16) | (17) | (18) | (19) | (20) |

| | | | | | | | | | |
|------|------|------|------|------|------|------|------|------|------|
| (21) | (22) | (23) | (24) | (25) | (26) | (27) | (28) | (29) | (30) |
| (31) | (32) | (33) | (34) | (35) | (36) | (37) | (38) | (39) | (40) |
| (41) | (42) | (43) | (44) | (45) | (46) | (47) | (48) | (49) | (50) |

TABLE 4
SUMMARY OF PERMEABILITY TESTS OF
EMBANKMENT MATERIAL^a
LOS ESTEROS DAM SITE

| Material Type | No. of Samples | Average Void Ratio | Average Permeability Ft/Min. | Design Value |
|---------------|----------------|--------------------|------------------------------|---|
| Sandy Clay | 3 | 0.552 | 0.044 | 1 X 10 ⁻⁶ Ft/Min. 1000 X 10 ⁻⁴ Ft/Min. |
| Clay | 2 | 0.455 | 0.0006 | |
| Sandy Gravel | 1 | | 0.4240* | |
| Sandy Gravel | 1 | | 0.04030* | |

*Estimated from Grain Size Distribution.

^aObtained from the U. S. Corps of Engineers, Albuquerque District.

CHAPTER V

GENERAL DESIGN REQUIREMENTS OF AN EMBANKMENT

The two major requirements of a good embankment design are (1) safety, and (2) economy. Before the design of a dam is started, economic studies are made to determine the most economical combination of the height of the dam and the capacity of the spillway. The topographic features and location of the site are also major factors to be considered.

It is of primary importance to the designer to have a thorough knowledge of subsurface conditions. In the preliminary examination of a project, the number of borings should be limited until the best site is selected. A good site offers a combination of topographic and geological features suitable for both embankment and spillway. During the preliminary investigation for site selection, the designer should consider the following requirements:

- (1) Type of construction materials.
- (2) Location of source.

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GENERAL INSTRUCTIONS FOR THE FIELD

1. PURPOSE

The purpose of this manual is to provide instructions for the field.

design and (2) layout, and (3) execution.

design of a plan is based on a knowledge of the terrain.

to determine the most convenient and shortest route.

height of the land and the position of the buildings.

topographic features and location of the buildings.

major features to be identified.

It is of primary importance to the designer to have

a thorough knowledge of the terrain and its features.

preliminary examination of a plan is necessary to determine

whether the plan is feasible and whether the design is correct.

lected. A good plan should be a combination of the two.

and geological features and the location of the buildings.

spillage. During the preliminary examination the designer

selection the designer should consider the following points:

requirements:

(1) Type of construction material.

(2) Location of the building.

(3) Distance of hauls.

(4) Formation of the foundation.

After the site is selected, the number of exploratory holes can be increased until a thorough knowledge of the underground conditions is obtained.

In the first phase of subsurface investigations, the drilling program is aimed primarily at identification and classification of the soil and bedrock encountered, the location of the water table, moisture content of soils under consideration either as foundations or potential sources of fill materials, and similar basic information. At the later phases of subsurface investigation, undisturbed samples are obtained for thorough testing in the laboratory.

There are many methods of embankment design. Some are more accurate than others. The author will not attempt to discuss these methods. However, Justin, Hinds and Creager enumerate the following criteria necessary for the safe design of an earth dam. "An earth dam should be so designed that:

(1) There is no danger of overtopping.

(2) The seepage line is well within the downstream face. (See Plate 8, Appendix B.)

(3) Distance of land.

(4) Formation of the land.

After the site is selected, the first step is to determine the depth and direction of the water table. The water table is the level to which the water in the ground will rise. It is the level of the water in the ground. The water table is the level of the water in the ground. The water table is the level of the water in the ground.

In the first phase of a site investigation, the drilling program is aimed primarily at determining the location and classification of the soil and rock. The location of the water table is determined by the location of the water table. The location of the water table is determined by the location of the water table. The location of the water table is determined by the location of the water table.

There are many methods of determining the location of the water table. The most common method is to use a piezometer. A piezometer is a device that measures the pressure of the water in the ground. The pressure of the water in the ground is measured by a piezometer. The pressure of the water in the ground is measured by a piezometer. The pressure of the water in the ground is measured by a piezometer.

- (1) There is no danger of waterlogging.
- (2) The seepage rate is low and the water is clean.
- (3) The seepage rate is low and the water is clean.

- (3) The upstream face slope is safe against sudden drawdown.
- (4) The upstream and downstream slope is flat enough that, with the materials utilized in the embankment, they will be stable and show a satisfactory factor of safety by recognized methods of analysis.
- (5) The upstream and downstream slopes of the earth dam are flat enough that the shear stress induced in the foundation is enough less than the shear strength of the material in the foundation to insure a suitable factor of safety.
- (6) There is no opportunity for the free passage of water from the upstream to the downstream face.
- (7) Water which passes through and under the dam when it reaches the discharge surface has a pressure and velocity so small that it is incapable of moving the material of which the dam or its foundation is composed.
- (8) The upstream face is properly protected against wave action and the downstream face is protected against the action of

(1) The object of the present study is to determine the effect of the various factors on the rate of the reaction.

(2) The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction.

(3) The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction.

(4) The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction.

(5) The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction.

(6) The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction. The object of the present study is to determine the effect of the various factors on the rate of the reaction.

the rain.⁵"

Overtopping is one of the most frequent causes of dam failure. This is due to insufficient freeboard or inadequate spillway capacity. A spillway should be designed for the maximum flood possible and with a safety factor. The water should never be allowed to go over the top of an embankment since this will cause its failure.

The slopes of earth dams should be chosen to provide stability with an adequate safety factor. Usually the steepest practicable slopes are the most economical; however, in certain cases greater over-all economy may be affected from the standpoint of both construction and maintenance costs by employing slopes flatter than the minimum required by stability criteria.⁶ One of the methods used to determine the stability of embankment slopes is the slip circle method. It is simpler and faster than other methods and the results are accurate enough for everyday practice.

Seepage control is one of the major problems in

⁵Justin Hinds and Creager, Engineering for Dams, (New York: John Wiley and Sons Co., 1947), III, 662.

⁶U. S. Corps of Engineers, Earth Embankments, Engineering Manual, Civil Works Construction, CXXIII, (1954), 13.

the rain. Overtopping is one of the most frequent causes of dam failure. This is due to insufficient height or inadequate spillway capacity. A spillway should be designed for the maximum flood possible and with a safety factor. The water should never be allowed to rise over the top of an embankment since this will cause failure.

The slopes of earth dams should be designed to provide stability with an adequate safety factor. Usually the steepest practicable slopes are the most economical; however, in certain cases greater safety economy may be obtained from the use of a flatter slope. Construction and maintenance costs are also affected. Flatter than the minimum required by stability criteria. One of the methods used to obtain an adequate factor of safety is the use of a slope. This is the most common method and is the simplest and easiest to construct. The results are accurate enough for practical purposes. Seepage control is one of the major problems of

²Arthur W. Dyer and Lester W. Dyer, *Water Engineering* (New York: John Wiley and Sons, Inc., 1954), p. 111.

³U. S. Corps of Engineers, *Water Engineering* (Engineering Manual, Civil Works Administration, 1954) (1954), 13.

earth dam design. The purpose of using different materials in an embankment section is primarily to control seepage through or under the dam. One of the most common arrangements consists of a central impervious core, flanked upstream and downstream by relatively pervious sections. The shells enclose and protect the core; the upstream shell affords stability against sudden drawdown and the downstream shell acts as a drain that controls the line of seepage. It is used when there is a considerable amount of pervious material, and it is well adapted to an impervious foundation or a pervious foundation with a positive earth-fill, cut-off trench to impervious strata.

Slopes of earth dams should be protected against wind and wave erosion, ice formation and floating debris. The requirements are different for upstream and downstream slopes, with the upstream slope generally requiring a more extensive treatment. Dumped riprap is the most common type of upstream slope protection. The rock should be well graded from spalls to the maximum size required. The thickness of riprap for the main embankment of earth dams should be based on the wave heights computed for determining freeboard requirements before the allowances for setup and runup are made.

earth dam design. The purpose of using filter
materials in an embankment section is to prevent
seepage through or under the dam. On a low dam
arrangement consists of a central filter core
flanked upstream and downstream by filter
sections. The filter material is placed in the
upstream shell either directly against the
and the downstream shell and a filter core
the line of seepage. In a low dam there is a
sufficient amount of filter material and it is
added to an existing foundation or a
foundation with a filter section. In a high dam
impermeable filter.
Type of filter. A filter is a material
wind and wave action. The filter is a
The requirements are different for different
slopes, with the upstream slope generally requiring
more extensive treatment. The filter is a
common type of filter used in the design of
should be well graded and the filter material
required. The thickness of filter for the
bankment of earth dam should be based on the
height computed for the upstream face and
before the filter is placed in the dam.

The minimum thickness of riprap for flood control reservoirs is 12 inches. For reservoirs having conservation pools, the riprap need extend from the crest to a point $1\frac{1}{2}$ times the wave height, measured vertically below the conservation pool level.⁷

The downstream slope of an embankment should be protected against runoff or wind erosion. Grass or any kind of vegetative cover is generally the most desirable type of downstream slope protection. In arid or semiarid regions where this is not possible, earth embankments should be protected with coarse gravel or rock blankets having a minimum thickness of 6 inches depending on the gradation of the blanket material.

Filter blankets are used between the riprap and the embankment to provide drainage and protect the embankment material from the piping action of waves. A filter blanket should satisfy two major criteria:

- (1) Its 15 per cent size should be smaller than 5 times the 85 per cent size of the embankment material.
- (2) Its 15 per cent size should be larger than 5 times the 15 per cent size of the embankment

⁷Ibid., 14

material.

Filters many times are eliminated if the riprap thickness is increased. Whether to use a filter or increase the riprap thickness depends on the judgment of the designer.

SECRET

Please note that the information in this report

concerns the activities of the organization in the

increase the number of members and the

of the organization.

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U.S. DEPARTMENT OF JUSTICE
FEDERAL BUREAU OF INVESTIGATION
WASHINGTON, D.C.

CHAPTER VI

METHOD OF DESIGN

Stability Analysis.---Both upstream and downstream slopes of the maximum river section were investigated for stability. The stability of an earth dam is that property which enables it to stay in position. An embankment is stable if the resultant of all the forces acting upon it does not result in movement. If the forces resisting movement are in exact balance with those forces tending to produce movement, the dam would be barely stable and the factor of safety would be unity. This would be a dangerous condition because the slightest increase in the acting forces would result in failure. However, if the safety factor is 1.50 or more the embankment will be safe against overturning. This is an empirical value which is considered adequate in structures of earth or rock.

The differential slice method of analysis was considered the simplest, fastest and most accurate. In the above method of design, it is assumed that failure

CHAPTER VI

STABILITY OF WEIR

Stability Analysis - Both upstream and downstream

slopes of the weir. The stability of the weir is then for stability. The stability of the weir is then property which enables it to stay in position. The weir is stable if the resultant of all the forces acting upon it does not result in a overturning moment. Forces resisting movement are in exact balance with those forces tending to produce movement. The weir would be barely stable and the factor of safety would be unity. This would be a dangerous condition because of the increase in the lifting force would result in the weir. However, if the factor of safety is 1.15 or more the weir will be safe against overturning. The factor of safety is the ratio of the sum of the resisting forces to the sum of the driving forces. The differential of the forces is then considered the lifting force and the driving force. The above method of analysis is the same as the method of analysis of a dam.

in an embankment will have a sliding surface nearly circular in cross-section. Plates 9 and 10, Appendix B, illustrate the application of this method. A number of slices, represented by vertical lines, were drawn through the rotating segment. Any vertical line from the slope to the dam to the dangerous circle represents the weight W of a strip infinitely small in width. The components N and T of one of these vertical lines represent the resolution of the weight W into forces normal and tangential to the dangerous circle. The N and T components were then plotted above appropriate base lines on vertical extensions of the slices, and the points were connected by smooth curves. The area under each curve was measured by a planimeter. These areas, multiplied by the unit weight of the material, gave the total N and T forces acting on the particular circle. The summation of the N forces multiplied by the tangent of the angle of friction gave the total resisting frictional force along the arc. Any cohesive value of the material along the length of the arc of the dangerous circle must be added to the friction value determined from the N forces to obtain the total force resisting shear along the arc. However, in this design the cohesion of the impervious, and pervious material was

in an embankment... circular in cross-section... illustrate the... slides, represented... through the... the slope to the... the weight of a... components W and T... represent the... normal and tangential... and T components... base lines of vertical... points were connected... each curve was... multiplied by the... total W and T forces... The summation of the... of the angle of... frictional force... the material along... circle may be... from the W forces... shear along the... cohesion of the...

considered zero. The summation of the T forces gave the total overturning force tending to cause the material to shear along the arc. The safety factor is the sum of the resisting forces to the sum of the overturning forces, as follows:

$$S. F. = \frac{\sum N \tan \phi + LC}{\sum T} = \frac{\text{Resisting Forces}}{\text{Overturning Forces}}$$

$\sum N$ = Sum of normal forces

ϕ = Angle of internal friction of the
material which the circle cuts

L = Length of arc

C = Unit cohesion

$\sum T$ = Sum of tangential forces

Consideration was given to three conditions which might be critical. These are as follows:

- (1) Construction Condition - This is the condition of the embankment immediately after its completion. At that period the water in the soil, which remained during construction, may develop pore pressures. It is also considered that the embankment is not fully consolidated. These conditions are developed if the soil has a low value of permeability. Excess pore pressures within a slope will cause a decrease in effective normal pressures along the

failure arc. Tangential pressures are not affected except as the pressure of water may affect the unit weight of the soil. In soil mechanics, the usual method of determining the frictional resistance of soil with excess pore pressure is to reduce the total normal pressure by an amount equal to the pore pressure and to use resulting effective normal pressure in computing the shearing resistance. Because it is very difficult to determine the magnitude of pore pressures this condition is usually analyzed by using the shear values obtained from the unconsolidated undrained test, for the determination of the normal forces. However, if the material is free draining the pore pressures developed are insignificant to affect stability of the structure. Because the upstream and downstream shells are composed of pervious material it was decided on the basis of the above discussion that this condition would not be critical.

- (2) Post Construction Condition - This condition applies years after the construction of the

embankment. During this condition, the embankment and foundation are expected to be fully consolidated. Direct shear values and moist unit weights were used in this analysis. A minimum safety factor of 1.50 was required.

- (3) Sudden Drawdown Condition - The upstream slope of an earth-embankment is assumed to be saturated under drawdown conditions. If the speed of drawdown is rapid compared to the rate at which water can escape from the pores of the soil, an unbalanced hydrostatic force results which may cause failure of the slope. This is the condition analyzed for the stability of a slope subjected to sudden drawdown. When determining the stability of an upstream dam slope against drawdown, we must consider if the shell material is or is not free draining. If the material is clean sand and gravel or rock it will drain as fast as the water level goes down. Therefore, the resisting and driving forces within the drawdown range are calculated for the dry or moist weight of the material. Since the upstream shell of this embankment is composed of free draining material the moist

weight of the soil would have been used if the sudden drawdown case was analyzed. On the basis of the above discussion it was decided that no hydrostatic forces would be developed in the embankment, and therefore the sudden drawdown case would not be critical.

Analysis of the stability in the post construction condition was made with the shear strength values and unit weights applicable to each material. Due to the inconclusive shear tests results, the cohesion of the impervious material was eliminated in the stability analysis. Because no shear testing was done on the sand and gravel, the angle of shear after lengthy consideration and investigation, was assumed to be 33 degrees. This was based on the shape and gradation of the material. The adopted design angle of shears for the various materials are as follows:

| Material | Direct Shear | |
|------------|--------------|--------|
| | ϕ° | C. TSF |
| Impervious | 22 | 0 |
| Pervious | 33 | 0 |

The unit weights of the various types of materials, as used in stability analyses are as follows:

weight of the soil, which was found to be

uniform throughout, and was found to be

about 1.25 times that of the water.

It was found that the soil was

in the condition of a loose sand.

However, this would not be true if

Analysis of the soil is in the following

condition was made with the following

unit weights applied to each material.

Inconclusive about each material, the

laboratory material was eliminated in the

analysis. Because no other testing

and gravel, the angle of repose of the

and investigation, was found to be

was based on the same and direction of the

The adopted design and in the

materials are as follows:

| Material | Unit Weight | |
|----------|-------------|----------|
| | lb/cu ft | lb/cu yd |
| Gravel | 125 | 375 |
| Sand | 125 | 375 |

The unit weights of the various types of

used in the design are as follows:

| Material | Unit Weight, p.c.f. | | | |
|------------|---------------------|-------|-----------|-----------|
| | Dry | Moist | Saturated | Submerged |
| Impervious | 101.6 | 116.0 | 126.6 | 64.2 |
| Pervious | 126.2 | 135.0 | 141.7 | 79.3 |
| Rock | 100 | --- | --- | --- |

Numerous trial failure surfaces were analyzed to determine the critical arc for each slope. At first a 1 on 2 slope was tried to elevation 4760 feet, but the critical factor of safety was 1.43, which is below the minimum. Then a 1 on 2.5 slope was tried to the same elevation and the minimum safety factor was 1.51. This was considered satisfactory. A 1 on 3 slope was tried from elevation 4760 feet to elevation 4615 and the total section was analyzed. The minimum safety factor of 1.84 obtained was considered very safe. Since the upstream and downstream slopes are symmetrical, the trial arcs apply to both sides. The above analysis is shown on Plates Nos. 9 and 10, Appendix B.

Flow nets were not utilized in this design, because the sudden drawdown case was not critical. However, the latest trend in embankment design is to eliminate them, since their degree of accuracy is questionable. Whenever the sudden drawdown condition is applicable, submerged unit weights are used for the normal forces and saturated

unit weights for the tangential forces. This is important for the investigation of the worse condition in the embankment. When submerged unit weights are used for computing the normal forces of the embankment, it is assumed that the water causes a reduction in the effective weight of the soil, which in turn causes a reduction in the resisting forces. The weight of the water cannot be considered in the computation of frictional resistance. The above method is the same as if the saturated weight of the soil were reduced by the magnitude of the hydrostatic force. The use of saturated unit weights for the tangential forces increases the magnitude of the overturning force. The reduction of the resisting forces, and increase of the overturning forces yields a more conservative factor of safety.

Slope Protection.--Dumped riprap will be used for upstream slope protection. The riprap should meet the following gradation requirements:

| <u>Screen Size in Inches</u> | <u>Percentages Passing by Weight</u> |
|------------------------------|--------------------------------------|
| 18 - - - - - | 100 |
| 12 - - - - - | 55-100 |
| 10 - - - - - | 38-86 |
| 8 - - - - - | 13-66 |
| 6 - - - - - | 0-46 |
| 4 - - - - - | 0-14 |

The thickness of the riprap blanket was determined

from the maximum wave height expected. This was computed on the basis of 55 m.p.h. wind velocity and 6.34 miles fetch. Fetch is defined as the straight length of the reservoir subject to wind action in statute miles. These values were supplied by the Hydrology Section of the Albuquerque District, U. S. Corps of Engineers. Wave height computations are shown in Appendix A. Using 4.34 foot wave height, a 22 inch minimum riprap thickness is required.⁸ However, a 6 foot thickness will be used in this design in order to prevent any excessive maintenance of the slopes. The downstream slope will be protected by a 3 foot rock blanket. The rock blanket should meet the same gradation requirements as the riprap.

A filter blanket between the riprap and the pervious shells will not be required, since the pervious material meets all the requirements of a filter. A complete study of this is shown on Plate No. 6 and in Computations, Appendix A.

Seepage.--The seepage under the embankment will be controlled by the positive cut-off of impervious material and by a grout curtain extending 40 feet into the rock.

⁸U. S. Corps of Engineers, Slope Protection for Earth Dams, (Vicksburg: Waterways Experiment Station, 1949), p. 37.

Since this is a flood control dam, some loss of water is permissible, as long as it does not affect the safety of the structure. According to the seepage studies shown on Plate No. 8, Appendix B, the discharge of water through the embankment will be 0.001111 cubic feet per minute per foot of length or 1.417 cubic feet per minute for the entire length of dam. These values indicate that the seepage through the embankment is too small to do any damage. To protect the downstream toe of the dam, a trench will be excavated and filled with derrick stone. This trench will carry all the water seepage into the river and prevent saturation and eventual disintegration of the downstream toe.

Consolidation.---The change in volume of a soil mass under load is called consolidation. If the soil is saturated, the decrease in volume under stress is accomplished by water being squeezed from the voids, the rate of which is controlled by the permeability of the soil. If the soil is free-draining, the consolidation process is very rapid. However, in the case of fine-grained soils the pore spaces are very small and the consolidation process is of long duration. Initially the stress is carried partly by the soil skeleton and partly by the pore water and is transferred entirely to

the soil skeleton only after consolidation is complete. The consolidation process is very important, because it affects the settlement of structures. The amount of settlement which will occur on certain types of structures, such as earth dams, and railroad and highway embankments, is important where the final grade of the structure is affected.

The sandstone foundation of the Los Esteros damsite is considered excellent to carry the proposed embankment. This conclusion was based on visual inspection and experience. It was decided by the investigating party that its consolidation would be negligible to affect the stability and safety of the embankment. Since no consolidation tests were performed on the impervious material, a consolidation test from another project with material having the same physical properties was investigated. The results of the investigation show that the impervious core of the embankment will consolidate 2.9 feet in a 50 year period. The 50 year value was selected because it is considered to be the life of the dam. Consolidation studies of the impervious material are shown on Plates Nos. 1-5, and in Computations, Appendix A. It is expected that the pervious material will consolidate during construction and, therefore, will not present any

problem.

Sliding.--The wedge method of analysis was used to investigate the embankment against sliding. It was assumed that the critical plane is located between the foundation and embankment. The analysis consisted of determining the forces acting on the mass of soil represented by triangle ABC in Fig. 4, Page 66, Appendix A. The weight of the material in the wedge was computed, and each load was multiplied by the corresponding angle of internal friction.

Force P_H was evaluated by active pressure considerations. This active pressure tends to push the vertical wall AB to the right causing the wedge to slide along the weak plane. The active pressure was computed as follows:

$$P_H = \frac{\gamma H^2}{2} K_A$$

Where:

γ = Unit weight of the soil

H = Height of embankment

K_A = Coefficient of active pressure

In computing the coefficient of active pressure of the angle δ was considered as having a negative value, since the wall tends to move downward passing the active

wedge of the soil. The angle of inclination β was considered equal to zero, because it was assumed that the plane of the wall was vertical. The angle ω was considered as having a negative value, since the slope of the soil acting on the wall is downward instead of upward. The safety factor against sliding is the ratio of the resisting forces and active pressure. All computations of the above method are shown on Page 66, Appendix A.

Analysis of the wedge method is more complicated if the dam is composed of cohesive soil. When the foundation is composed of a clay layer, trial failure surfaces at various depths must be considered. The consolidation conditions of the clay affect the shearing strengths and determine the location of the critical surface.

Foundation Shear.---An approximate method was used in computing the shearing stress in the foundation. The above method was selected, because the foundation material is composed of sand, gravel and sandstone. Field investigations have shown that the sandstone is well consolidated, and should be able to withstand the embankment pressures. Foundations consisting largely of fine, loose, cohesionless materials or unconsolidated

wedge of the soil. The material is then
contained equal to the weight of the soil
the plate of the soil. The soil is then
considered as having a weight of 1.00
of the soil acting on the plate. The soil
upward. The weight of the soil is then
of the resting force and the weight of the
compressions of the plate. The weight of the
Appendix A.

analysis of the data. The data is then
if the data is corrected for the weight of the
foundation is corrected for the weight of the
surface of the soil. The data is then
consolidation correction of the data. The data
temperature and pressure. The data is then
surface.

Foundation
in computing the bearing capacity of the soil.
above method was selected. The data is then
material is corrected for the weight of the
fluid investigation. The data is then
well consolidated. The data is then
enhancement. The data is then
fine, loose, compacted, and consolidated.

clays and silts may be very defective in shear strength and require thorough investigation. Computations and results of the above method are shown on Page 69 , Appendix A. In this design the total horizontal shear down to the sandstone was computed as follows:

$$s = \frac{h_1^2 - h_2^2}{2} W \tan^2 \left(45^\circ - \frac{\phi_1}{2} \right)$$

where:

s = Total horizontal shear down to rigid boundary

h_1 = Vertical distance from top of dam down to the rigid boundary

h_2 = Vertical distance from base of dam down to rigid boundary

ϕ_1 = Angle of internal friction

W = Effective weight per cubic foot of the material in its actual condition

The average unit shear is determined by the following equation:

$$s_a = \frac{s}{b}$$

where:

s_a = Average unit shear

b = Horizontal distance along base from top shoulder of slope to the toe of the dam

The safety factor against shear in the foundation is the ratio of the average shearing strength at the toe of the dam and maximum section, and the average shearing stress computed from the above method.

The safety factor is the ratio of the average stress in the material to the yield stress of the material. It is the ratio of the average stress in the material to the yield stress of the material. It is the ratio of the average stress in the material to the yield stress of the material.

CHAPTER VII

CONCLUSIONS

The construction of the Los Esteros dam appears to be feasible from a geologic, and engineering viewpoint. More field investigation, and laboratory testing will be needed before a final decision can be reached. However, it is concluded from results of completed explorations that no serious problems of an engineering geology nature will be encountered during construction.

The presence of filled in sink holes and perennial springs present the possibility of serious leakage from the reservoir under a high hydrostatic head of water, which would reopen solution piping in old flow channels. It is believed, however, that a 40 foot grout curtain will prevent the occurrence of the above condition. Since this is a flood control structure some loss of water is not objectionable, as long as it does not affect its stability.

Based on the analytical results presented in Appendix C, it is concluded that the design of the

APPENDIX C

CONTENTS

The construction of the first part of the appendix is
be feasible from a practical and theoretical point of view.
More field investigation and laboratory work is
needed before a final decision can be reached. However,
it is concluded from the results of the preliminary
that no serious problem of stability is involved.
will be encountered during construction.
The presence of the water in the soil is
springing occurs as a result of the water pressure
the reservoir under the foundation. It is
which would result in a serious problem of stability.
It is believed, however, that the water pressure
prevent the occurrence of the serious problem of stability.
this is a first-order approximation and it is
not objectionable, as long as it is used as a
stability.

Based on the analysis of the first part of the
Appendix C is a first-order approximation and it is

embankment section is conservative with respect to stability of slopes and foundations. During the stability investigations it was found that elevation 4760 was the most critical for the 1 on 2.5 slope. At the above elevation the slope was flattened to 1 on 3. The 1 on 3 slope was analyzed and a minimum safety factor of 1.84 was obtained, which was considered quite adequate. The arcs were not allowed to penetrate into the foundation, since the sandstone is very strong, at least relative to the soil. Due to the absence of cohesion the shallow arcs yielded the lowest safety factors. Arcs cutting the impervious material yielded high safety factors.

The results of stability analyses are to be considered tentative, since laboratory shear testing to determine design shear strengths are not yet complete. However, it is considered that selection of design values has been conservative, and based on testing completed to date, it is not anticipated that the conclusions given above regarding the adequacy of the embankment section with respect to stability will be changed.

The consolidation of the foundation will be negligible. The above assumption was based on the sandstone formation. According to the drilling logs the sandstone is classified as very dense, and with

abundant section is conservative with respect to
stability of slopes and foundations. Further investigations
investigations to establish the stability of slopes and
most critical for a road on the river. The river
elevation the slope is estimated to be 1 to 2 feet on
slope was estimated and a slight safety factor of 1.5
was obtained, which was considered to be adequate. The
sides were not allowed to penetrate into the foundation
since the sandstone is very strong and hard and the
the soil. Due to the absence of a foundation and the
yielded the lower water table. The water table
investigations indicated that the water table
The results of the investigation were that the
considered conservative, since the sandstone is very strong
determine the water table and the water table
However, it is considered that the water table is very strong
has been conservative, but it has been estimated that the
date, it is not anticipated that the water table will
above regarding the stability of the foundation and
which respect to stability of the foundation and
The consolidation of the sandstone will be
negligible. The above investigation was made on the
sandstone foundation. According to the findings of the
sandstone is classified as very hard, and is

considerable strength. The impervious core of the embankment will consolidate 2.9 feet in a 50 year period. The 50 year value was selected, because it is considered to be the life of the dam. The initial settlement will be 2.8 feet and will occur during the two year construction period. It is customary to construct earth dams to a somewhat greater height and width than the neat dimensions called for by the plans. In this case the core will be built 2.9 feet higher than the pervious shells. It is believed that its consolidation will not affect the stability of the embankment. The pervious shells of the embankment are expected to consolidate fully during construction.

The safety factor of 2.21 obtained during the investigation of the embankment against sliding, indicates that the structure would be safe. A safety factor of 1.50 is usually required in most designs. This is an empirical value obtained through research and practice. The safety factor of 3.31 against shearing resistance in the foundation is considered adequate. It should be noted that a safety factor of 1.00 indicates that the resisting forces, and driving forces are in balance.

On the basis of the test results, stability, seepage, consolidation studies, safety factor against

considerable strength. The structure was built on the
foundation with concrete. The foundation was built on the
The 50 year return was estimated to be 1.5 times
to be the life of the structure. The structure was built
2.5 feet and with tower. The tower was built on the
period. It is estimated that the structure will last for
some time. The structure was built on the foundation
called for by the design. The structure was built on the
basis 2.5 feet higher than the ground level. It is
believed that the structure will last for
safety of the structure. The structure was built on the
foundation and is expected to last for
construction.

The safety factor of the structure was estimated to be
investigation of the structure. The structure was built on the
that the structure was built on the foundation. The structure
is usually tested by the structure. The structure was built on the
value obtained from the structure. The structure was built on the
factor of 1.5 was obtained from the structure. The structure was built on the
foundation and is expected to last for
that a safety factor of 1.5 was obtained from the structure. The structure was built on the
forces and driving forces. The structure was built on the
On the basis of the structure. The structure was built on the
energy, the structure was built on the foundation.

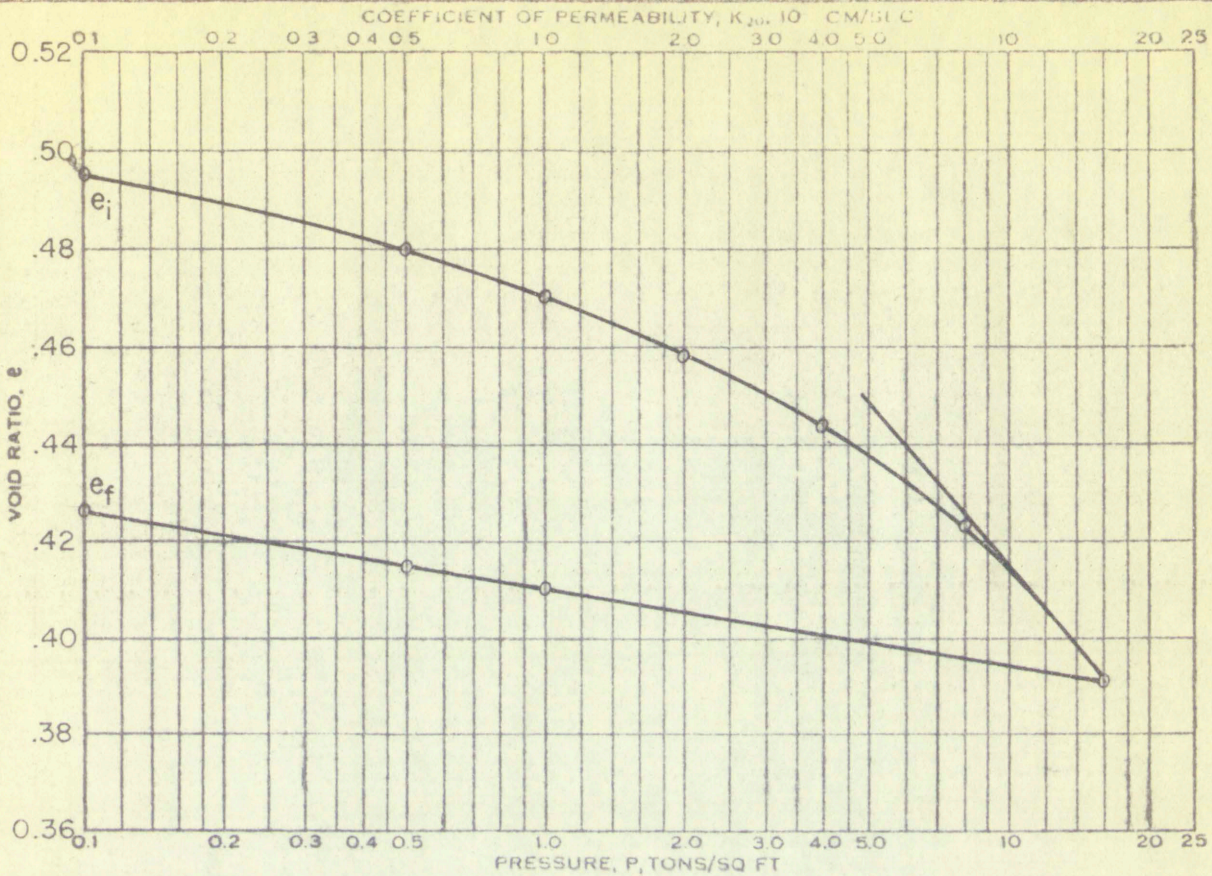
foundation shear, it is concluded that this embankment will be safe as a flood control structure.

to maintain the same level of service as the previous year

will be made as a result of the same

CHARTER
BOARD
OFFICE
SCHOOL
DISTRICT

APPENDIX A
COMPUTATIONS



TEST DATA

Type of Specimen

REMOLED

Overburden Pressure, P_0 Tons/Sq FtPreconsol Pressure, P_c Tons/Sq FtCompression Index, C_c 0.11Permeability at Initial e K_{20} _____ $\times 10^{-10}$ Cm/Sec
Ft/Min

(2H)

Initial Ht. 0.92 in. Diam. 4.0/in.

Initial Saturation, S_0 % 81Final Saturation, S_f % 92

Initial Dry Density Lbs/Cu Ft 114

Initial Water Content, W_0 % 14.7

Remarks:

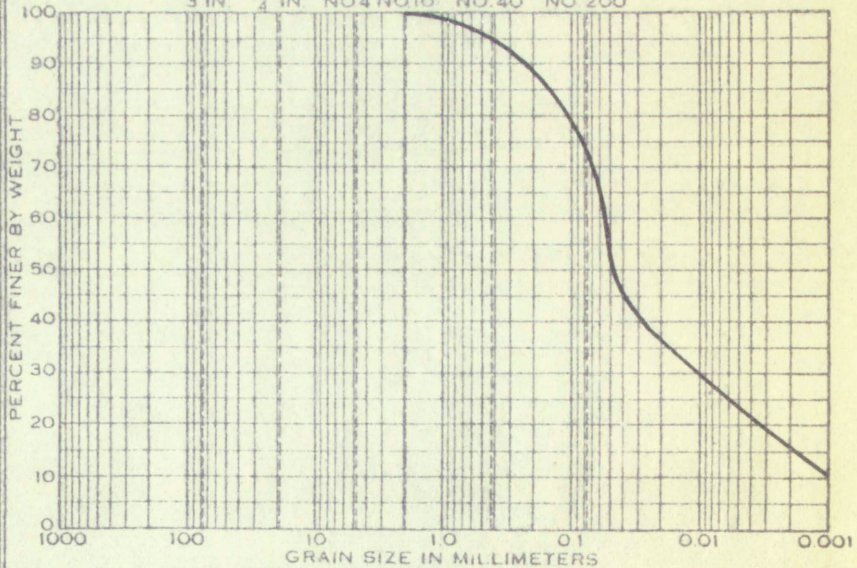
Expansion Pressure=0.10 T/SQ.FT.

Classification Sandy Clay (CL)

LL 31 G 2.72

PL 15 D_{10}

U.S. STANDARD SIEVE SIZE

3 IN. $\frac{3}{4}$ IN. NO.4 NO.10 NO.40 NO.200

| | | | | | | |
|---------|--------|------|--------|--------|------|--------------|
| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
| | Coarse | Fine | Coarse | Medium | Fine | |

Project

LOS ESTEROS DAMSITE

EMBANKMENT MATERIAL

Area

BORROW AREA "B"

Boring No.

COMPOSITE SAMPLE

Sample No.

Elev or Depth

Date 1 JULY 1957

CONSOLIDATION TEST REPORT

PLATE I

Consolidation-Test Data

Initial height of sample = $Z = 0.92"$

Diameter of sample = $4.01"$

Initial void ratio = 0.495

Initial water content = 14.7%

$G = 2.72$

Area of container = $\pi r^2 = 3.1416 \times 2.005^2 = 12.6292$ sq.in.

$V = \text{Volume} = AZ = 11.6189$ cu.in.

$S = \frac{WG}{e} = \frac{0.147 \times 2.72}{0.495} = 0.8077$

$\gamma_t = \frac{G + Se}{1 + e} \gamma_w = \frac{2.72 + (0.8077 \times 0.495)}{1.495} \times 62.4 =$

$= 130.2163$ lb./cu. ft.

$\gamma_t = \gamma_s \frac{1+W}{1+e} = \gamma_s \frac{1+14.7}{1+0.495}$; $\gamma_s = 169.7293$ lb./cu. ft.

$V_s = \text{Volume of solids}$ $V_s = \frac{V}{1+e}$; $e = 0.495$

$V_s = 7.7718$ cu.in. $W_s = V_s s = 7.7718 \times 0.0982 = 0.7632$ lb.

| Tons per Sq. Ft. Pressure | e_i Void Ratio | W_s lb. | $A \gamma_s$ | $\frac{Z = W_s(1+e)}{A \gamma_s}$ in | Ave. 2H. in. | 2H (cm) | H (cm) |
|---------------------------------|---------------------|-----------|--------------|---|-----------------|------------|-----------|
| 0 | 0.495 | 0.7632 | 1.2402 | 0.9200 | | | |
| 1 | 0.471 | | | 0.9052 | | | |
| 2 | 0.458 | | | 0.8972 | 0.9012 | 2.29 | 1.145 |
| 4 | 0.445 | | | 0.8892 | 0.8932 | 2.27 | 1.135 |
| 8 | 0.423 | | | 0.8757 | 0.8824 | 2.24 | 1.12 |
| 16 | 0.3905 | | | 0.8557 | 0.8657 | 2.20 | 1.10 |

Consolidation Test

Initial height of sample = 2.5 in.

Diameter of sample = 1 in.

Initial water content = 25.5%

Initial void ratio = 0.75

$\sigma_v = 2.5$ lb/in²

Area of consolidation = 0.785 in²

$V =$ Volume of soil = 1.96 in³

$S = 25.5\%$ Initial water content

$w = 25.5\%$

$$\gamma_s = \frac{G}{V_s} = \frac{2.65}{1.96} = 1.35 \text{ g/cm}^3$$

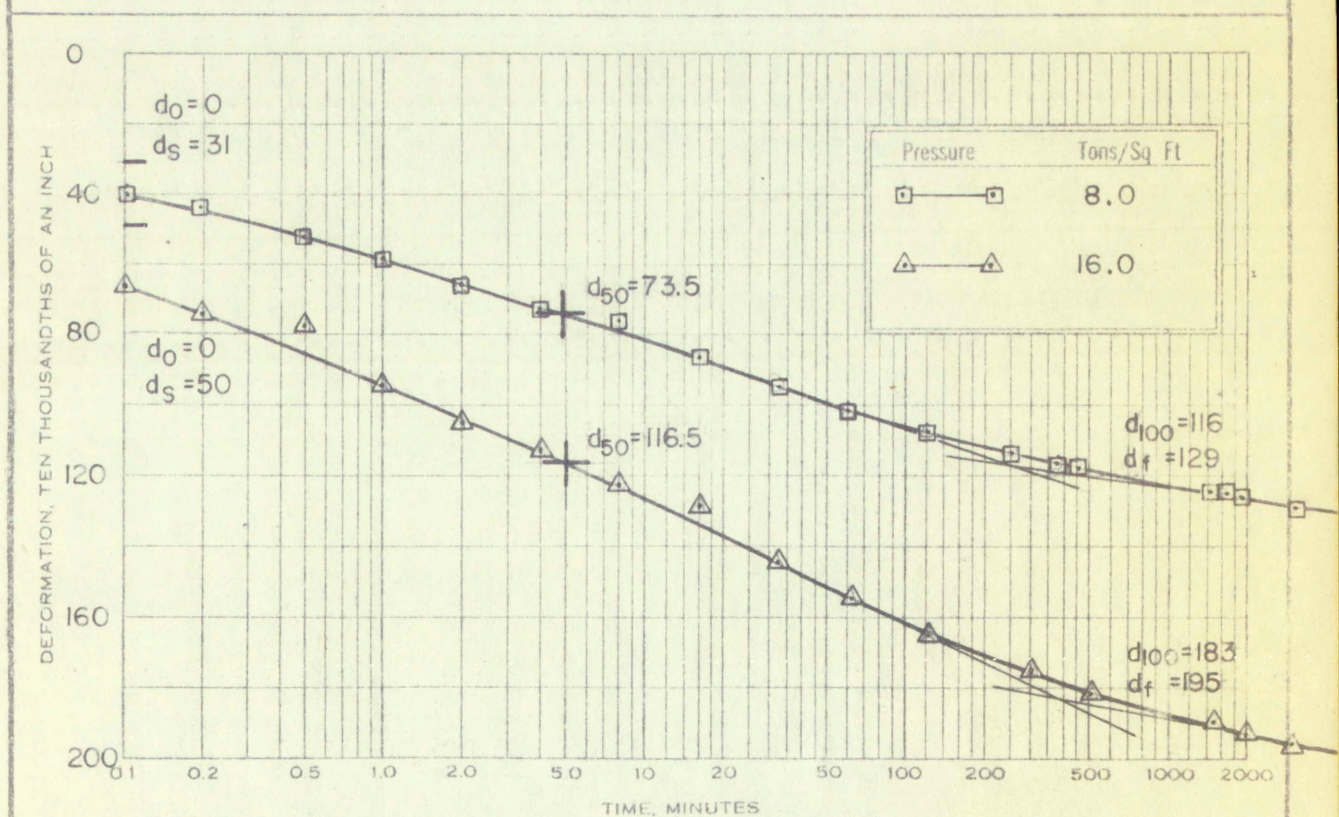
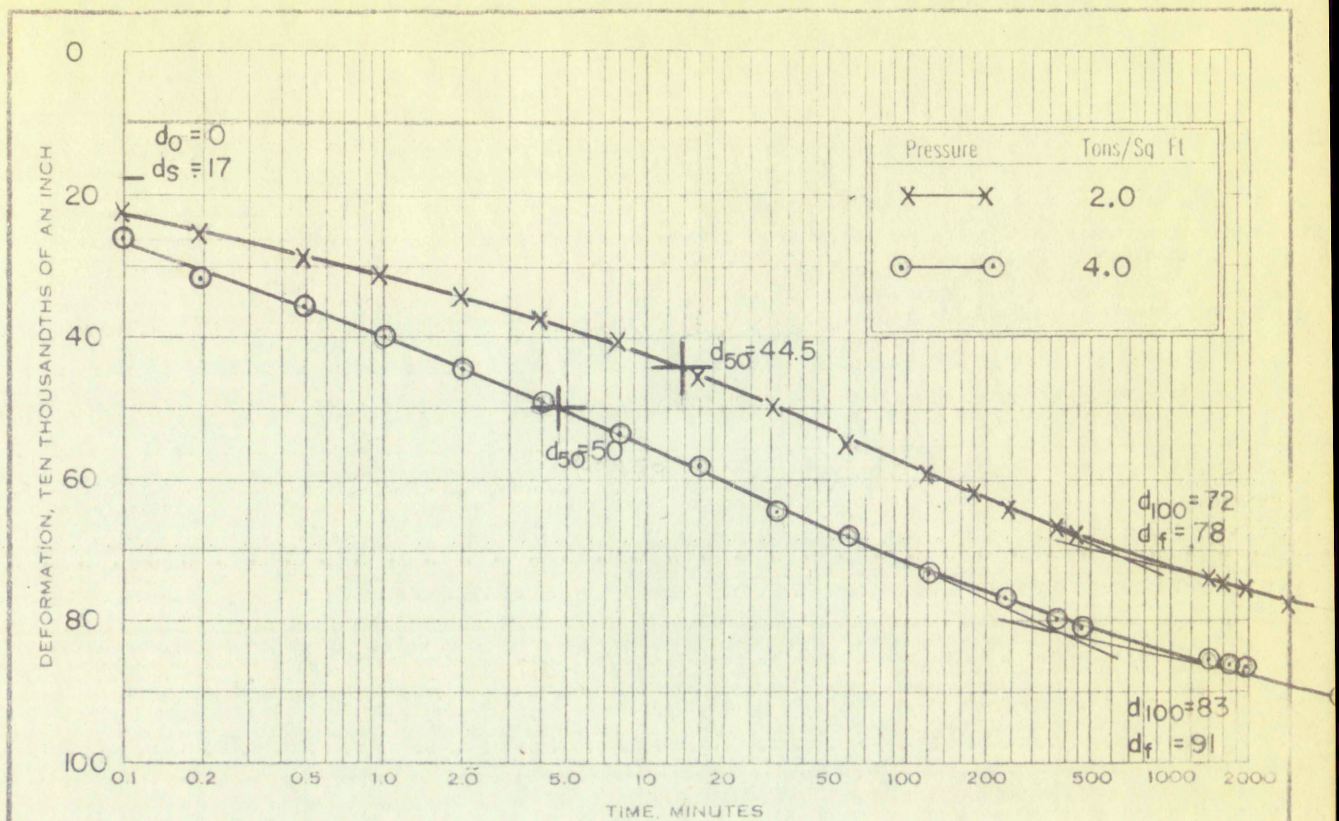
$\gamma_t = 1.35 \text{ g/cm}^3$ Initial total unit weight

$$\gamma_t = \frac{\gamma_s}{1 + e} = \frac{1.35}{1 + 0.75} = 0.77 \text{ g/cm}^3$$

$V_v =$ Volume of voids = 1.47 in³

$V_s = 0.49 \text{ in}^3$ Volume of solids

| Pressure (lb/in ²) | Final height (in) | Final void ratio | Final water content (%) | Final unit weight (g/cm ³) |
|--------------------------------|-------------------|------------------|-------------------------|--|
| 0 | 2.50 | 0.75 | 25.5 | 0.77 |
| 1 | 2.45 | 0.70 | 25.5 | 0.82 |
| 2 | 2.40 | 0.65 | 25.5 | 0.87 |
| 4 | 2.35 | 0.60 | 25.5 | 0.92 |
| 8 | 2.30 | 0.55 | 25.5 | 0.97 |
| 16 | 2.25 | 0.50 | 25.5 | 1.02 |



Project

LOS ESTEROS DAMSITE - EMBANKMENT MATERIAL
BORROW AREA "B"

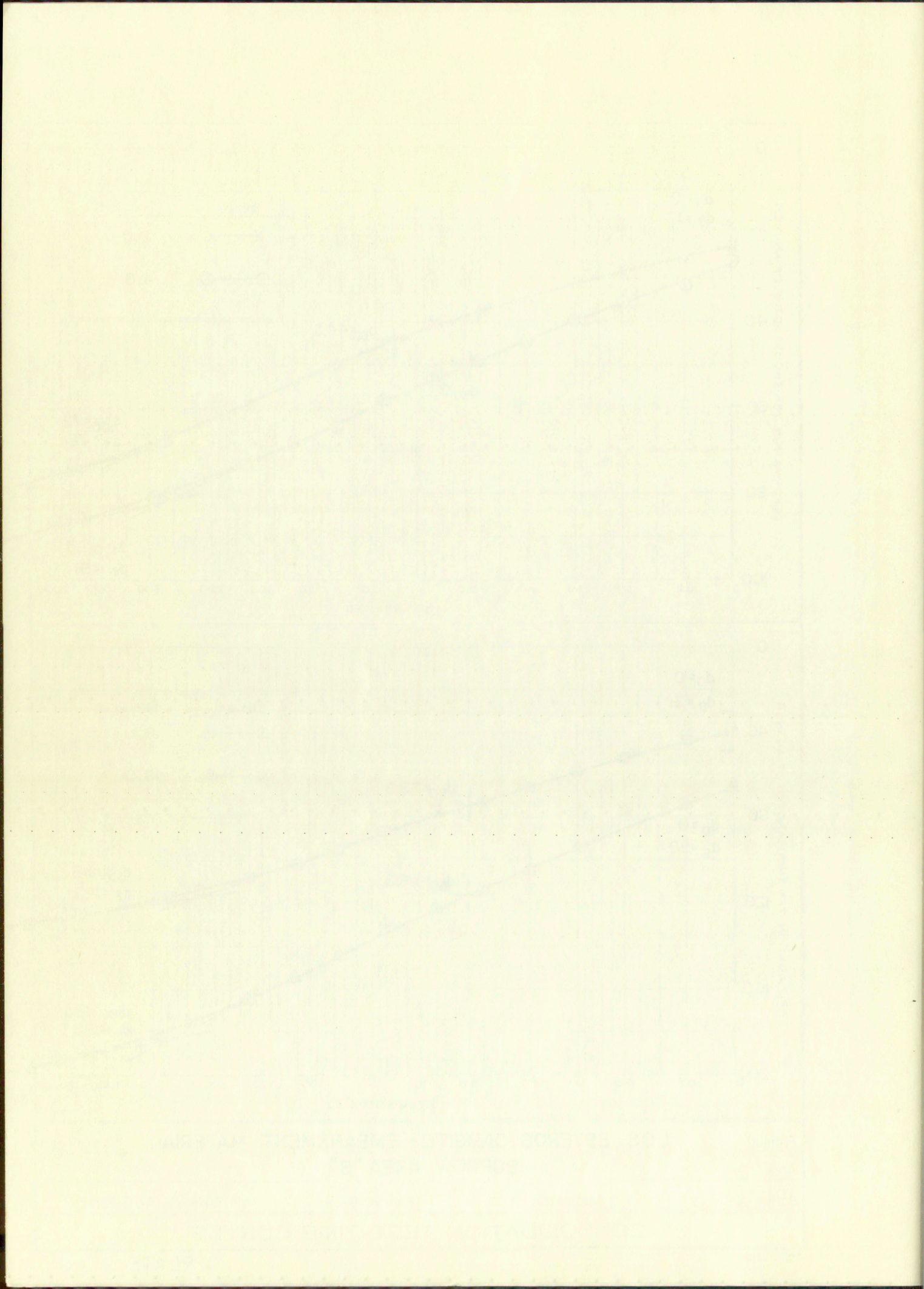
Boring No.

Sample No.

Elev or Depth

Date 2 AUG 1957

CONSOLIDATION TEST-TIME CURVES



Loading

1. 0 to 2.0 tons/sq. ft.

DATA:

$$d_o = 0$$

$$d_s = 17$$

$$d_{100} = 72$$

$$d_f = 78$$

$$i = \frac{d_o - d_s}{d_o - d_f}$$

$$r = \frac{d_s - d_{100}}{d_o - d_f}$$

$$C_v = \frac{0.196H^2}{t_{50}}$$

$$r = \frac{55}{78} \times 100 = 70.5\%$$

$$i = \frac{17}{78} \times 100 = 21.8\%$$

$$C_v = \frac{0.196 \times 1.145^2}{13.5 \times 60} = 3.2 \times 10^{-4} \text{ cm}^2/\text{sec.}$$

Loading

2.0 - 4.0 tons/sq. ft.

DATA:

$$d_o = 0$$

$$d_s = 17$$

$$d_{100} = 83$$

$$d_f = 91$$

$$r = \frac{66}{91} \times 100 = 72.5$$

$$i = \frac{17}{91} \times 100 = 18.7\%$$

$$C_v = \frac{0.196 \times 1.135^2}{4.7 \times 60} = 9.0 \times 10^{-4} \text{ cm}^2/\text{sec.}$$

Inventory

1. 0 to 2.0

DATA:

0.0

0.1

0.2

0.3

0.4

0.5

0.6

0.7

0.8

0.9

1.0

1.1

1.2

1.3

1.4

1.5

1.6

1.7

1.8

1.9

2.0

2.1

2.2

2.3

2.4

2.5

2.6

2.7

Inventory

2.0 - 4.0

DATA:

0.0

0.1

0.2

0.3

0.4

0.5

0.6

0.7

0.8

0.9

1.0

1.1

1.2

1.3

1.4

1.5

1.6

Loading

4.0 - 8.0 tons/sq. ft.

DATA:

$$d_o = 0$$

$$d_s = 31$$

$$d_{100} = 116$$

$$d_f = 129$$

$$r = \frac{85}{129} \times 100 = 65.9\%$$

$$i = \frac{31}{129} \times 100 = 24.0\%$$

$$C_v = \frac{0.196 \times 1.12^2}{60 \times 4.7} = 8.7 \times 10^{-4} \text{ cm}^2/\text{sec.}$$

Loading

8.0 - 16.0 tons/sq. ft.

DATA:

$$d_o = 0$$

$$d_s = 50$$

$$d_{100} = 183$$

$$d_f = 195$$

$$r = \frac{133}{195} \times 100 = 68.2\%$$

$$i = \frac{50}{195} \times 100 = 25.6\%$$

$$C_v = \frac{0.196 \times 1.10^2}{5 \times 60} = 7.9 \times 10^{-4} \text{ cm}^2/\text{sec.}$$

100000

4.0 - 3.0 - 2.0

1.0 - 0.0 - 1.0

2.0 - 1.0 - 0.0

0.0

3.0 - 2.0 - 1.0

4.0 - 3.0 - 2.0

5.0 - 4.0 - 3.0

6.0 - 5.0 - 4.0

7.0 - 6.0 - 5.0

8.0

9.0 - 8.0 - 7.0

10.0

11.0 - 10.0 - 9.0

12.0 - 11.0 - 10.0

13.0 - 12.0 - 11.0

14.0 - 13.0 - 12.0

15.0 - 14.0 - 13.0

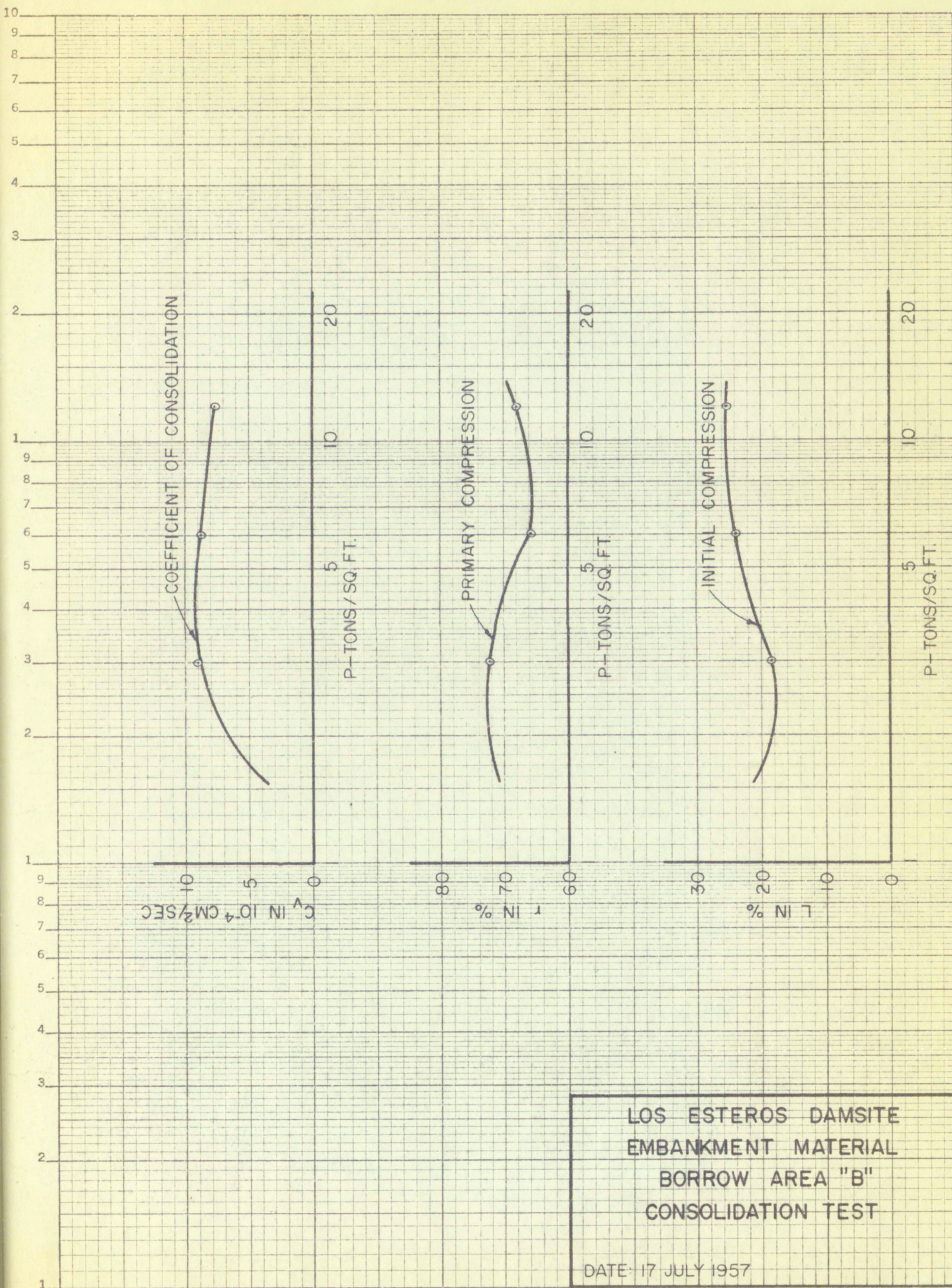
16.0 - 15.0 - 14.0

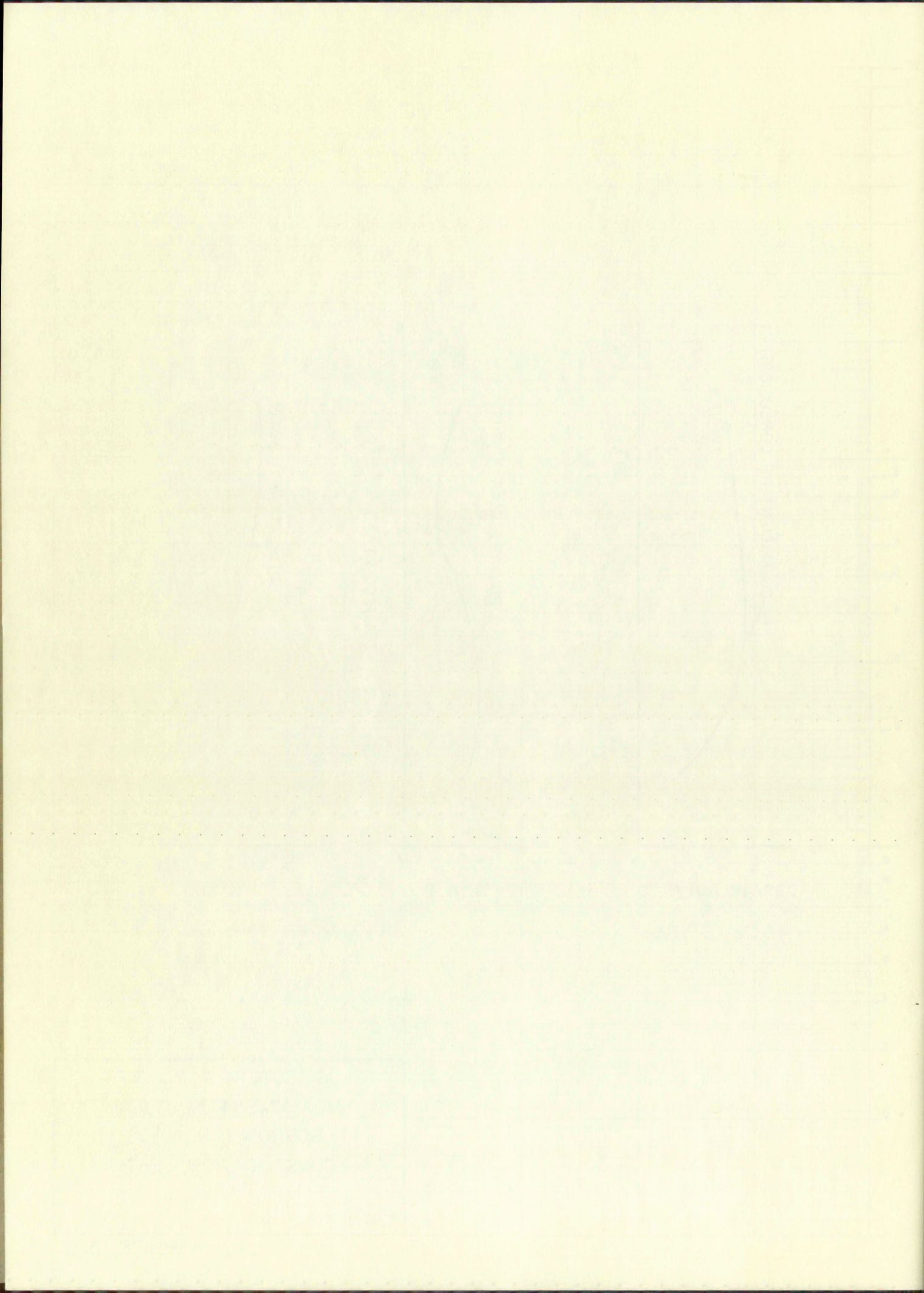
17.0 - 16.0 - 15.0

18.0 - 17.0 - 16.0

19.0 - 18.0 - 17.0

20.0 - 19.0 - 18.0





Settlement Analysis

Assume construction of embankment to last two years and to be continuous.

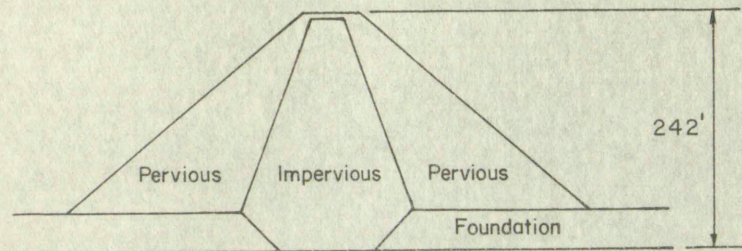


FIG. 1

Average pressure at maximum height of embankment

$$= \frac{242}{2} \times 116 = 14,036 \text{ lb. per sq. ft.}$$

$$e_1 = 0.495$$

$$e_{avg.p} = 0.427$$

$$\Delta H = \frac{e_1 - e_{avg.p}}{1 + e_1} 2 H.$$

$$\Delta H = \frac{0.495 - 0.427}{1 + 0.495} (242)$$

$$\Delta H = \frac{(0.068)(242)}{1.495} = 11.0 \text{ ft.}$$

For 7.0 tons per sq. ft.

$$i = 25\%, r = 66\%, C_v = 8.5 \times 10^{-4} \text{ cm}^2/\text{sec.}$$

$$\text{Initial settlement} = 11.0 \times 0.25 = 2.8 \text{ ft.}$$

$$\text{Primary settlement} = 11.0 \times 0.66 = 7.3 \text{ ft.}$$

$$\text{Secondary settlement} = 11.0 \times 0.09 = 0.9 \text{ ft.}$$

Assumed double drainage

$$t_{\text{years}} = T \frac{H^2}{C_v}$$

Selected Examples

Assume constant rate of return is 10% per year
and so be continuous.



Assume initially an amount of \$1000 at time 0

$$= \frac{1000}{1.1} = 909.09$$

at time 1

$$\Delta A = \frac{1000 - 909.09}{1.1} = 90.91$$

$$\Delta A = \frac{1000 - 909.09}{1.1} = 90.91$$

$$\Delta A = \frac{1000 - 909.09}{1.1} = 90.91$$

For 1000 bond per year

$$1 = 1000 - 909.09 = 90.91$$

$$1000 - 909.09 = 90.91$$

$$1000 - 909.09 = 90.91$$

$$1000 - 909.09 = 90.91$$

$$1000 - 909.09 = 90.91$$

$$1000 - 909.09 = 90.91$$

$$t_{\text{years}} = \frac{(121 \times 30.5)^2 T}{8.5 \times 10^{-4} \times 31.5 \times 10^6} = \frac{13,619,790}{26,775} T$$

$$t_{\text{years}} = 509 T$$

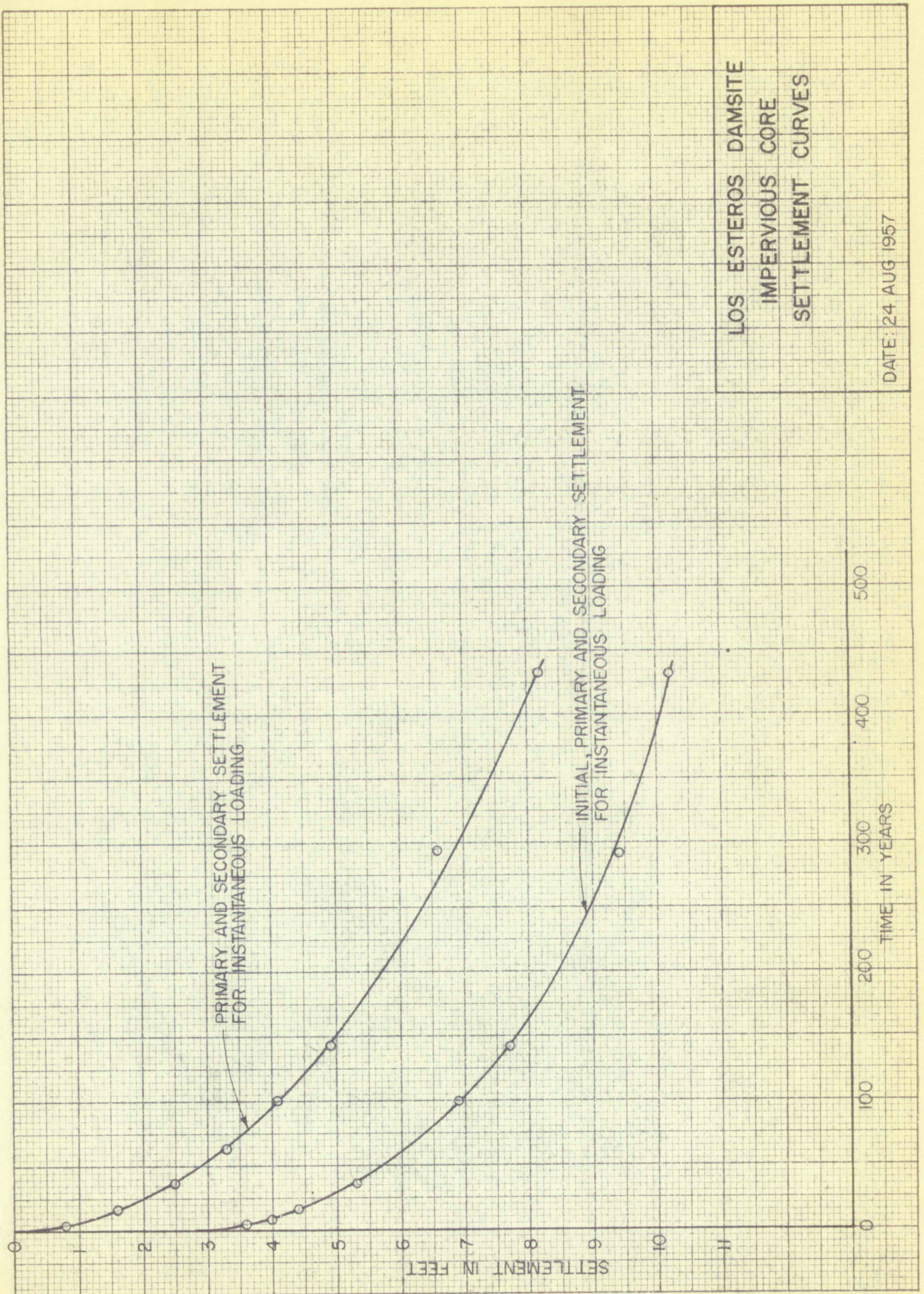
| U % | Initial Sett.Ft. | Primary & Secondary Settlement in ft. | T | t in yrs. | % Total Sett. |
|-----|---------------------|--|---------|--------------|------------------|
| 0 | 2.8 | 0 | 0 | 0 | 25.4 |
| 1 | " | 0.1 | 0.00008 | 0.04 | 26.4 |
| 2 | " | 0.2 | 0.00031 | 0.2 | 27.3 |
| 3 | " | 0.2 | 0.00071 | 0.4 | 27.3 |
| 4 | " | 0.3 | 0.00126 | 0.6 | 28.2 |
| 5 | " | 0.4 | 0.00196 | 1.0 | 29.1 |
| 6 | " | 0.5 | 0.00283 | 1.4 | 30.0 |
| 7 | " | 0.6 | 0.00385 | 2.0 | 30.9 |
| 8 | " | 0.7 | 0.00503 | 2.6 | 31.8 |
| 9 | " | 0.7 | 0.00636 | 3.2 | 31.8 |
| 10 | " | 0.8 | 0.00785 | 4.0 | 32.7 |
| 15 | " | 1.2 | 0.01767 | 9.0 | 36.4 |
| 20 | " | 1.6 | 0.03140 | 16.0 | 40.0 |
| 30 | " | 2.5 | 0.07070 | 36.0 | 48.2 |
| 40 | " | 3.3 | 0.1257 | 64.0 | 55.5 |
| 50 | " | 4.1 | 0.1964 | 100.0 | 62.7 |
| 60 | " | 4.9 | 0.2827 | 143.9 | 70.0 |
| 80 | " | 6.6 | 0.5772 | 293.8 | 85.5 |
| 90 | " | 7.4 | 0.8480 | 431.6 | 92.7 |
| 100 | 2.8 | 8.2 | | | 100.0 |

1941

1941

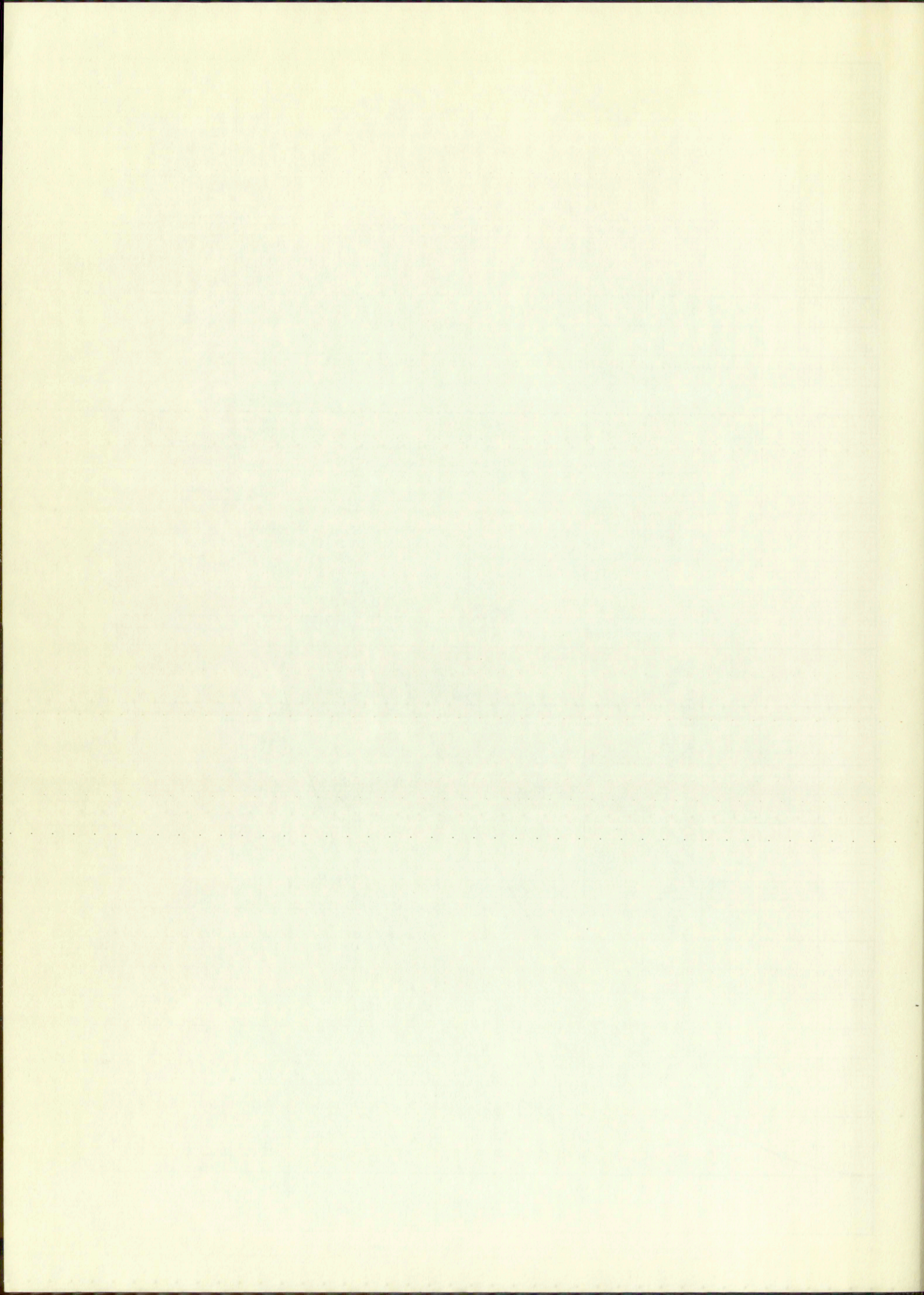
1941

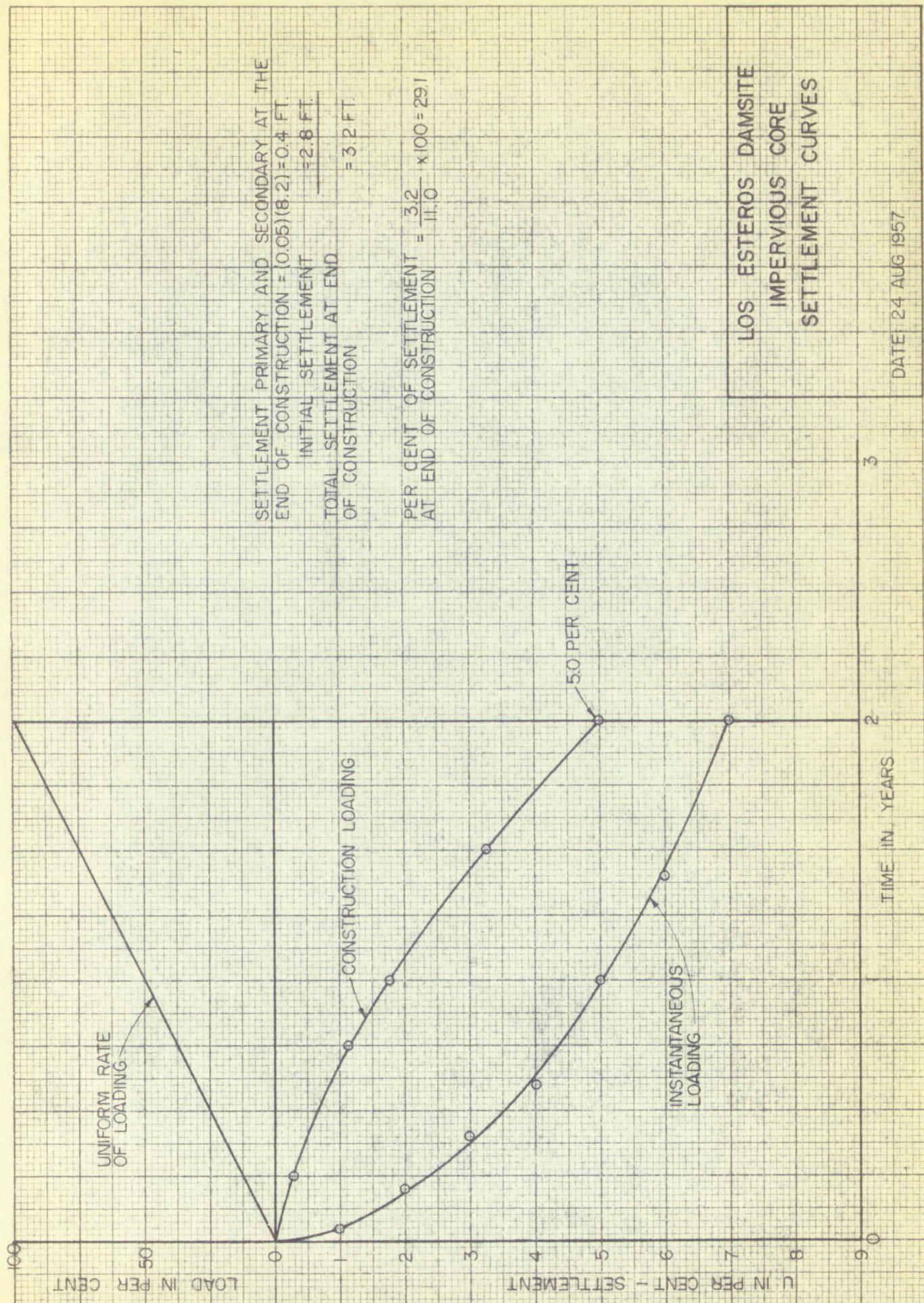
| Year | Initial Settlement | Primary Settlement | Secondary Settlement | Year |
|------|--------------------|--------------------|----------------------|------|
| 0 | 2.8 | 0 | 0 | 0 |
| 1 | " | 0.1 | 0.1 | 1 |
| 2 | " | 0.2 | 0.2 | 2 |
| 3 | " | 0.3 | 0.3 | 3 |
| 4 | " | 0.4 | 0.4 | 4 |
| 5 | " | 0.5 | 0.5 | 5 |
| 6 | " | 0.6 | 0.6 | 6 |
| 7 | " | 0.7 | 0.7 | 7 |
| 8 | " | 0.8 | 0.8 | 8 |
| 9 | " | 0.9 | 0.9 | 9 |
| 10 | " | 1.0 | 1.0 | 10 |
| 11 | " | 1.1 | 1.1 | 11 |
| 12 | " | 1.2 | 1.2 | 12 |
| 13 | " | 1.3 | 1.3 | 13 |
| 14 | " | 1.4 | 1.4 | 14 |
| 15 | " | 1.5 | 1.5 | 15 |
| 16 | " | 1.6 | 1.6 | 16 |
| 17 | " | 1.7 | 1.7 | 17 |
| 18 | " | 1.8 | 1.8 | 18 |
| 19 | " | 1.9 | 1.9 | 19 |
| 20 | " | 2.0 | 2.0 | 20 |
| 21 | " | 2.1 | 2.1 | 21 |
| 22 | " | 2.2 | 2.2 | 22 |
| 23 | " | 2.3 | 2.3 | 23 |
| 24 | " | 2.4 | 2.4 | 24 |
| 25 | " | 2.5 | 2.5 | 25 |
| 26 | " | 2.6 | 2.6 | 26 |
| 27 | " | 2.7 | 2.7 | 27 |
| 28 | " | 2.8 | 2.8 | 28 |
| 29 | " | 2.9 | 2.9 | 29 |
| 30 | 2.8 | 3.0 | 3.0 | 30 |

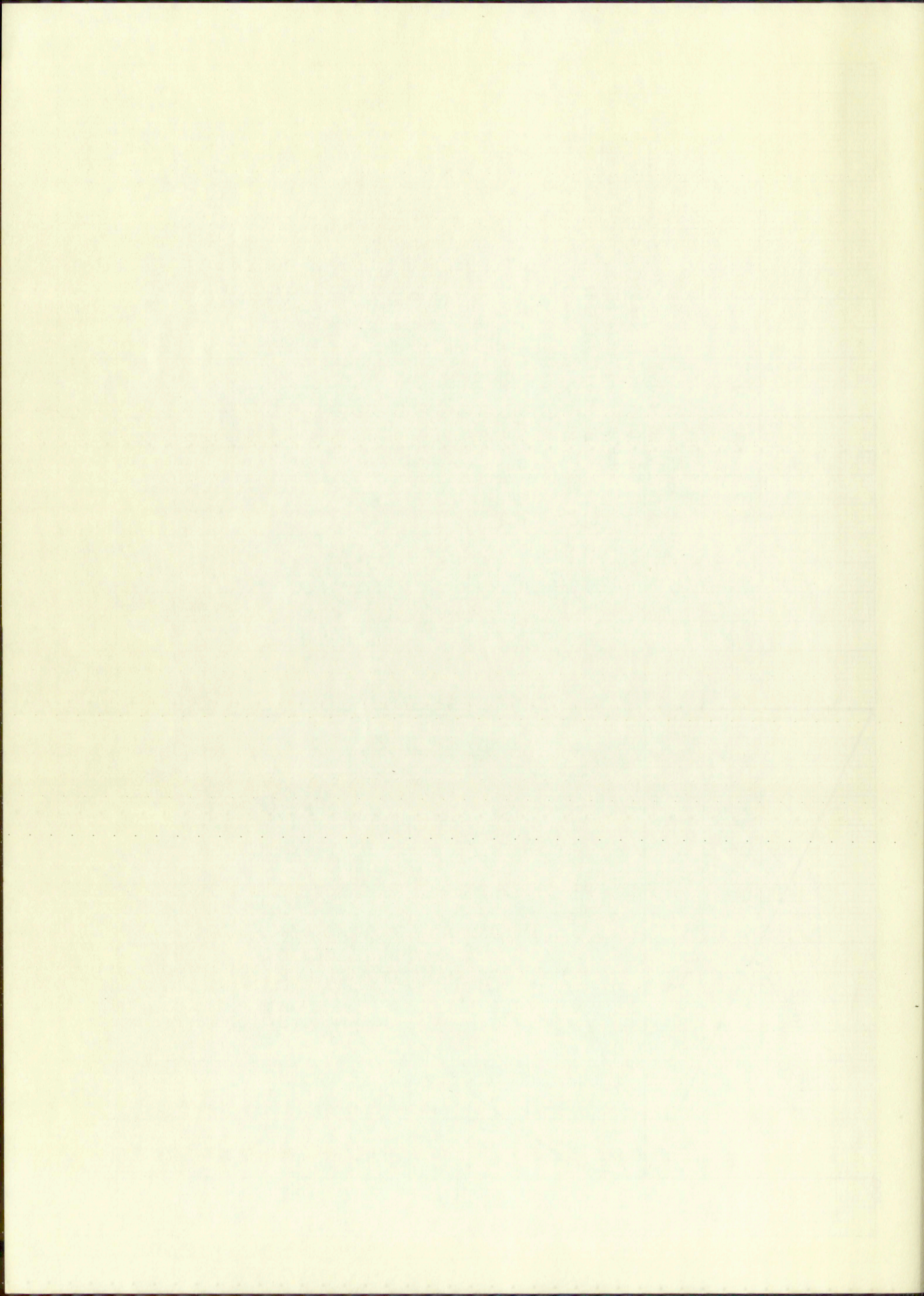


LOS ESTEROS DAMSITE
IMPERVIOUS CORE
SETTLEMENT CURVES

DATE: 24 AUG 1957







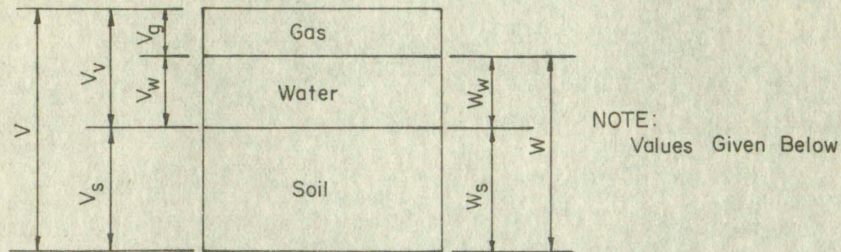
Unit Weight (Impervious)-

FIG. 2

Given:

Optimum unit weight of soil = 116.0 lb./cu. ft.

Optimum moisture content = 14.2 per cent

Specific gravity = 2.72

Computations:

$$\text{Weight of solids} = \frac{116.0}{1.142} = 101.58 \text{ lb./cu. ft.}$$

$$\text{Weight of solids} = 101.58 \times 454 = 46,117 \text{ gms.}$$

$$\text{Weight of water} = 46,117.0 \times 0.142 = 6,549 \text{ gms.}$$

$$\text{Total weight} = 52,666$$

$$V_s = \frac{46,117}{2.72 \times 1} = 16,955 \text{ c.c.}$$

$$V_v = 28,317 - 16,955 = 11,362 \text{ c.c.}$$

$$V_w = \frac{6,549}{1 \times 1} = 6,549 \text{ c.c.}$$

$$V_g = 28,317 - 16,955 - 6,549 = 4,813 \text{ c.c.}$$

$$e = \frac{V_v}{V_s} = \frac{11,362}{16,955} = 0.67$$

$$n = \frac{V_v}{V} = \frac{11,362}{28,317} = 0.40$$

$$s = \frac{V_w}{V_v} = \frac{6,549}{11,362} \times 100 = 57.6 \text{ per cent}$$

Unit Weight (pcf)

| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|-----|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 | 41 | 42 | 43 | 44 | 45 | 46 | 47 | 48 | 49 | 50 | 51 | 52 | 53 | 54 | 55 | 56 | 57 | 58 | 59 | 60 | 61 | 62 | 63 | 64 | 65 | 66 | 67 | 68 | 69 | 70 | 71 | 72 | 73 | 74 | 75 | 76 | 77 | 78 | 79 | 80 | 81 | 82 | 83 | 84 | 85 | 86 | 87 | 88 | 89 | 90 | 91 | 92 | 93 | 94 | 95 | 96 | 97 | 98 | 99 | 100 |
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 | 41 | 42 | 43 | 44 | 45 | 46 | 47 | 48 | 49 | 50 | 51 | 52 | 53 | 54 | 55 | 56 | 57 | 58 | 59 | 60 | 61 | 62 | 63 | 64 | 65 | 66 | 67 | 68 | 69 | 70 | 71 | 72 | 73 | 74 | 75 | 76 | 77 | 78 | 79 | 80 | 81 | 82 | 83 | 84 | 85 | 86 | 87 | 88 | 89 | 90 | 91 | 92 | 93 | 94 | 95 | 96 | 97 | 98 | 99 | 100 |

Given:

Optimum unit weight of sand = 110 pcf

Optimum moisture content = 14.7%

Specific gravity = 2.65

Compaction:

Weight of solids = 110 pcf

Weight of water = 16.17 pcf

Weight of sand = 110 pcf

Total weight = 226.17 pcf

$$V_s = \frac{W_s}{G_s \times \gamma_w} = \frac{110}{2.65 \times 62.4} = 0.674$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{16.17}{62.4} = 0.259$$

$$V_v = \frac{W_v}{\gamma_w} = \frac{16.17}{62.4} = 0.259$$

$$V = V_s + V_w + V_v = 0.674 + 0.259 + 0.259 = 1.192$$

$$\gamma = \frac{W}{V} = \frac{226.17}{1.192} = 189.7$$

$$\gamma_{sat} = \frac{W_s + W_w + W_v}{V} = \frac{110 + 16.17 + 16.17}{1.192} = 189.7$$

$$\gamma_{dry} = \frac{W_s}{V} = \frac{110}{1.192} = 92.28$$

$$\gamma_{min} = \frac{W_s}{V_{min}} = \frac{110}{1.192} = 92.28$$

$$\gamma_{\text{dry}} = \frac{G}{1+e} \quad W = \frac{2.72}{1 + 0.67} (62.4) = \frac{169.7}{1.67} =$$

$$= 101.6 \text{ lb./cu. ft.}$$

Assume soil will be 100 percent saturated.

$$\gamma_{\text{sat.}} = \frac{G + Se}{1 + e} \quad W = (62.4)$$

$$\gamma_{\text{sat.}} = \frac{2.72 + 0.67}{1 + 0.67} (62.4) = \frac{211.5}{1.67} =$$

$$= 126.6 \text{ lb./cu. ft.}$$

$$\gamma_{\text{sub.}} = 126.6 - 62.4 = 64.2 \text{ lb./cu. ft.}$$

$$X_{100} = \frac{1}{1 + 0.01} = 0.9901$$

$$X_{100} = 0.9901$$

$$X_{100} = 0.9901$$

$$X_{100} = \frac{1}{1 + 0.01} = 0.9901$$

$$X_{100} = \frac{1}{1 + 0.01} = 0.9901$$

$$X_{100} = 0.9901$$

$$X_{100} = 0.9901$$

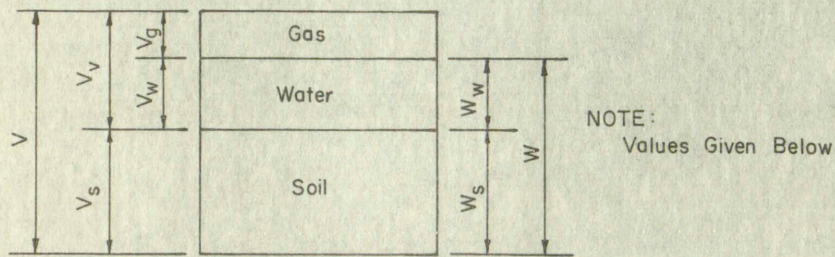
Unit Weight (Pervious)-

FIG. 3

Given:

Optimum unit weight of soil = 135.0 lb./cu. ft.

Optimum moisture content = 7.0 per cent

Specific gravity = 2.69

Computations:

$$\text{Weight of solids} = \frac{135.0}{1.07} = 126.2 \text{ lb./ cu. ft.}$$

$$\text{Weight of solids} = 126.2 \times 454 = 57,295 \text{ gms.}$$

$$\text{Weight of water} = 57,295 \times 0.07 = 4,011 \text{ gms.}$$

$$\text{Total weight} = 61,306 \text{ gms.}$$

$$V_s = \frac{57,295}{2.69 \times 1} = 21,299 \text{ c.c.}$$

$$V_v = 28,317 - 21,299 = 7,018 \text{ c.c.}$$

$$V_w = \frac{4,011}{1 \times 1} = 4,011 \text{ c.c.}$$

$$V_g = 28,317 - 21,299 - 4,011 = 3,007 \text{ c.c.}$$

$$e = \frac{V_v}{V_s} = \frac{7,018}{21,299} = 0.33$$

$$n = \frac{V_v}{V} = \frac{7,018}{28,317} = 0.25$$

Unit Weights (Pervious) -

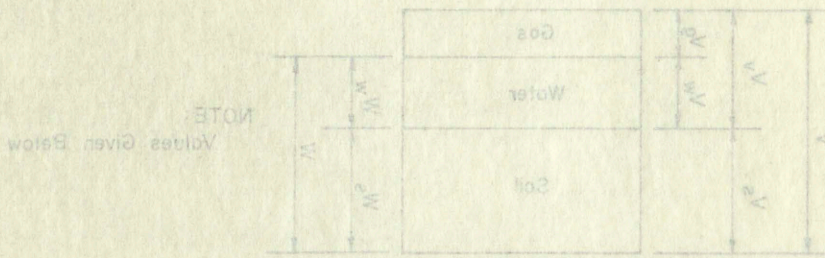


FIG. 3

Given:

Optimum unit weight of soil = 135.0 lb./cu. ft.

Optimum moisture content = 7.0 per cent

Specific gravity = 2.69

Computations:

$$\text{Weight of solids} = \frac{135.0}{1.07} = 126.2 \text{ lb./cu. ft.}$$

$$\text{Weight of solids} = 126.2 \times 4.54 = 572.95 \text{ gms.}$$

$$\text{Weight of water} = 572.95 \times 0.07 = 40.11 \text{ gms.}$$

$$\text{Total weight} = 613.06 \text{ gms.}$$

$$V_s = \frac{572.95}{2.69 \times 1} = 21.299 \text{ c.c.}$$

$$V_v = 28.317 - 21.299 = 7.018 \text{ c.c.}$$

$$V_w = \frac{40.11}{1 \times 1} = 40.11 \text{ c.c.}$$

$$V_g = 28.317 - 21.299 - 40.11 = 3.907 \text{ c.c.}$$

$$e = \frac{V_v}{V_s} = \frac{7.018}{21.299} = 0.33$$

$$n = \frac{V_v}{V} = \frac{7.018}{28.317} = 0.25$$

$$s = \frac{V_W}{V_V} = \frac{4,011}{7,018} \times 100 = 57.2 \text{ per cent}$$

$$\gamma_{\text{dry}} = \frac{G}{1+e} \quad W = \frac{2.69}{1 + 0.33} (62.4) = \frac{167.9}{1.33} =$$

$$= 126.2 \text{ lb./cu. ft.}$$

Assume soil will be 100 percent saturated.

$$\gamma_{\text{sat.}} = \frac{G+Se}{1+e} \quad W = \frac{2.69 + 0.33}{1.33} (62.4) = \frac{188.4}{1.33}$$

$$\gamma_{\text{sat.}} = 141.7 \text{ lb./cu. ft.}$$

$$\gamma_{\text{subm.}} = 141.7 - 62.4 = 79.3 \text{ lb./cu. ft.}$$

$$e = \frac{v}{v'} = \frac{0.11}{0.10} = 1.10 = 110\%$$

$$X_{\text{O}_2} = \frac{0}{1+0} = 0 \quad X_{\text{H}_2} = \frac{0.11}{1+0} = 0.11$$

$$X_{\text{H}_2\text{O}} = 1 - 0.11 = 0.89$$

At 100°C, the vapor pressure of water is 1.013 bar.

$$X_{\text{H}_2\text{O}} = \frac{0.89}{1+0} = 0.89 \quad X_{\text{H}_2} = \frac{0.11}{1+0} = 0.11$$

$$X_{\text{H}_2\text{O}} = 1 - 0.11 = 0.89$$

$$X_{\text{H}_2\text{O}} = 1 - 0.11 = 0.89 \quad X_{\text{H}_2} = 0.11$$

At 100°C, the vapor pressure of water is 1.013 bar.

At 100°C, the vapor pressure of water is 1.013 bar.

At 100°C, the vapor pressure of water is 1.013 bar.

Permeability of Pervious Material-

From gradation curves of plate No. 13

$$D_{20} = 0.80 \text{ to } 0.29 \text{ m.m.}$$

Using Table No. 2 p. 649 of Justin-Hinds and Creager Engineering for Dams, the permeability coefficients of sandy gravel are as follows:

$$K = 2,150 \times 10^{-4} \text{ cm/sec or } 4,240 \times 10^{-4} \text{ ft/min.}$$

$$K = 204 \times 10^{-4} \text{ cm/sec or } 403.0 \times 10^{-4} \text{ ft/min.}$$

Average permeability of pervious material = 1000×10^{-4} ft/min.

Permeability of pervious material-

From gradation curves of plate No. 13

$$D_{50} = 0.83 \text{ to } 0.59 \text{ mm.}$$

Using Table No. 2 p. 648 of "Soils and Foundations"

Engineering for Dams, the permeability coefficients of

sandy gravel are as follows:

$$K = 2.159 \times 10^{-4} \text{ cm/sec or } 4.349 \times 10^{-4} \text{ ft/min.}$$

$$K = 204 \times 10^{-4} \text{ cm/sec or } 403.9 \times 10^{-4} \text{ ft/min.}$$

Average permeability of pervious material = $1000 \times$

$$10^{-4} \text{ ft/min.}$$

Wave Height-

The following data on wind velocity and fetch were obtained from the Hydrology Section, Albuquerque District, U. S. Corps of Engineers.

Maximum 1 Hr. wind velocity = 55 m.p.h.

Maximum fetch = 6.3 miles (5.5 nautical miles)

From Part CXVI Ch. 8, p. 6, Hydraulic Design,

U. S. Corps of Engineers, Civil Works Manual.

$$H = 1.5F^{\frac{1}{2}} + 2.5 - F^{\frac{1}{4}} \text{ (Stevenson's Formula)}$$

(For violent squalls and fetches less than 39 miles)

where:

H = Wave height in feet

F = Fetch in nautical miles

$$1 \text{ statute mile} = \frac{5280}{6080.2} \text{ or } 0.8684 \text{ nautical miles}$$

$$H = 1.5(5.5)^{\frac{1}{2}} + 2.5 - (5.5)^{\frac{1}{4}}$$

$$H = 1.5(2.35) + 2.5 - 1.53 = 3.52 + 2.5 - 1.53 = 4.49 \text{ ft.}$$

Molitor's modification of Stevenson's formula

$$H = 0.196 V^{\frac{1}{2}} F^{\frac{1}{2}} + 2.5 - 1.035 F^{\frac{1}{4}}$$

where V = wind speed in knots

$$V = 55 \text{ m.p.h.} = 47.8 \text{ nautical miles per hour}$$

$$H = 0.196(47.8)^{\frac{1}{2}}(5.5)^{\frac{1}{2}} + 2.5 - 1.035(5.5)^{\frac{1}{4}}$$

$$H = 0.196(6.9)(2.35) + 2.5 - 1.035(1.53)$$

$$H = 3.18 + 2.5 - 1.58 = 4.10 \text{ ft.}$$

Wave Height

The following data were obtained from the following:

obtained from the following:

U. S. Corps of Engineers.

Maximum 1 ft. wave height = 0.1 ft.

Maximum 2 ft. wave height = 0.2 ft.

From 3 ft. to 10 ft. wave height = 0.3 ft.

U. S. Corps of Engineers, Div. 1, 1954.

H = 1.5 ft. + 1.5 ft. (average wave height)

(For violent agitation and rough seas, see 1.5 ft.)

where

H = wave height in feet

V = force in pounds per square foot

I assume that the wave height is 1.5 ft.

H = 1.5 ft. + 1.5 ft. (average wave height)

H = 1.5 ft. + 1.5 ft. + 1.5 ft. (average wave height)

Factor of safety of 1.5 is assumed.

H = 1.5 ft. + 1.5 ft. (average wave height)

Wave V = 1.5 ft. (average wave height)

V = 1.5 ft. (average wave height)

H = 0.1 ft. (average wave height)

H = 0.1 ft. (average wave height)

H = 0.1 ft. (average wave height)

Filter Design-

Part CXIX Ch. 1, p. 17, Seepage Control, U. S.

Corps of Engineers, Civil Works Manual

$$\frac{D_{15} \text{ (of filter)}}{D_{85} \text{ (of base)}} \leq 5$$

$$\frac{D_{15} \text{ (of filter)}}{D_{15} \text{ (of base)}} \geq 5$$

Size D_{85} of base material = 0.14 to 2.9 m.m.

Size D_{15} of base material = 0.001 to 0.008 m.m.

Max size D_{15} of filter = $0.14 \times 5 = 0.70$ m.m.

Max size D_{15} of filter = $0.008 \times 5 = 0.04$ m.m.

$$\frac{D_{15}}{D_{85}} = \frac{0.70}{0.14} = 5.00 = 5$$

$$\frac{D_{15}}{D_{85}} = \frac{0.70}{2.9} = 0.24 < 5$$

$$\frac{D_{15}}{D_{15}} = \frac{0.04}{0.001} = 40 > 5$$

$$\frac{D_{15}}{D_{15}} = \frac{0.04}{0.008} = 5 = 5$$

Filter Design

Part 1. Design of a gravity settling tank

Corps of Engineers, Civil Works Agency

$$\frac{D_{15} \text{ (of filter)}}{D_{85} \text{ (of base)}} > 5$$

$$\frac{D_{15} \text{ (of filter)}}{D_{15} \text{ (of base)}} > 5$$

Size D_{85} of base material = 0.14 to 0.25 mm.

Size D_{15} of base material = 0.001 to 0.038 mm.

Max size D_{15} of filter = 0.15 mm = 0.006 in.

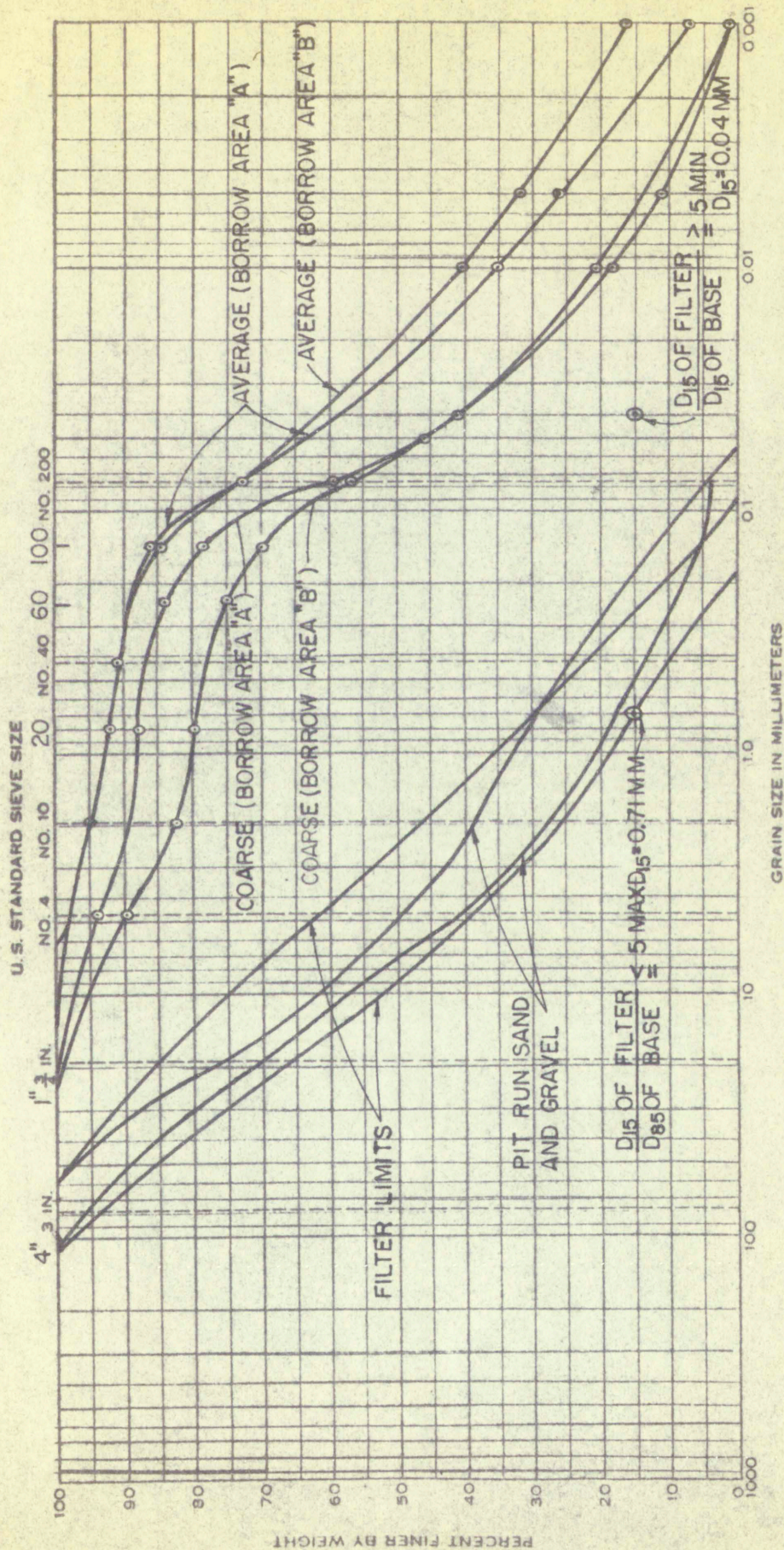
Max size D_{15} of filter = 0.003 mm = 0.00012 in.

$$\frac{D_{15}}{D_{85}} = \frac{0.15}{0.14} = 1.07 > 1$$

$$\frac{D_{15}}{D_{85}} = \frac{0.15}{2.9} = 0.05 < 1$$

$$\frac{D_{15}}{D_{15}} = \frac{0.04}{0.001} = 40 > 5$$

$$\frac{D_{15}}{D_{15}} = \frac{0.04}{0.003} = 13 > 5$$



| COBBLES | | | GRAVEL | | | SAND | | | SILT OR CLAY | | |
|---------|--|--|--------|------|--|--------|--------|------|--------------|--|--|
| | | | Coarse | Fine | | Coarse | Medium | Fine | | | |

Sample No. Elev or Depth

Classification

NatWC LL PL PI

Project LOS ESTEROS DAMSITE

Filter Capabilities of

Area PIT - RUN SAND AND GRAVEL

Boring No. TEST PIT

Date 2 AUG 1957

GRADATION CURVES

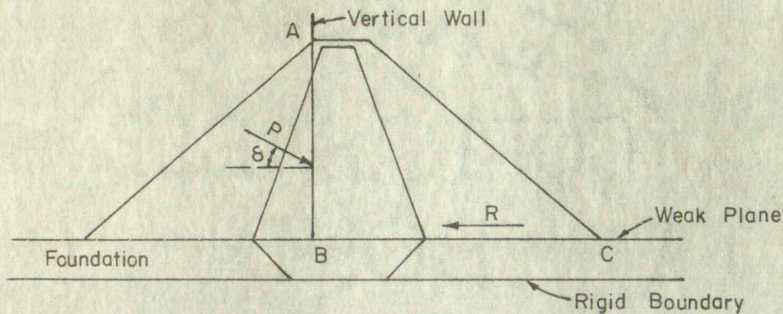
Safety Factor Against Sliding-

FIG. 4

Resisting Forces:

$$\begin{aligned} \text{Impervious material} &= 222 \times 20 \times 116 \times 1 + \frac{212 \times 107}{2} \times 116 \times 1 = 515,000 + 1,315,700 = \\ &= 1,830,700 \text{ lbs.} \end{aligned}$$

$$\begin{aligned} \text{Pervious material} &= \frac{11 + 181}{2} \times 106 \times 135 \times 1 = \\ &= 2,391,400 \text{ lbs.} \end{aligned}$$

$$\begin{aligned} \text{Resisting force acting on } 22^\circ \text{ material} &= 4,222,100 \times \\ \tan 22^\circ &= 1,705,700 \text{ lbs.} \end{aligned}$$

$$\begin{aligned} \text{Resisting force acting on } 33^\circ \text{ material} &= \frac{181 \times 522}{2} \times \\ 135 \times 1 \times \tan 33^\circ &= 6,377,500 \times \\ 0.649 &= 4,138,000 \text{ lbs.} \end{aligned}$$

$$\text{Total resisting forces} = 5,844,700 \text{ lbs.}$$

Driving Forces

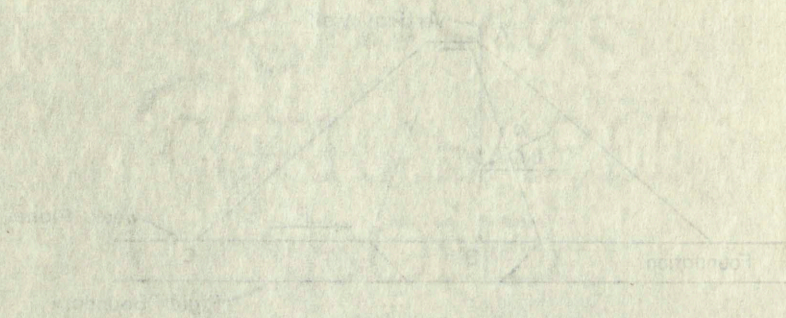
$$P = \frac{\gamma H^2}{2} K_A$$

where:

P = Active pressure

H = Height of embankment

Reaction Forces on a Dam



Reaction Forces

Horizontal reaction = $W \sin \theta$

Vertical reaction = $W \cos \theta$

Resultant reaction = W

Reaction forces on the dam = $W \sin \theta$

Reaction forces on the dam = $W \cos \theta$

Reaction forces on the dam = $W \sin \theta$

Reaction forces on the dam = $W \cos \theta$

Reaction forces on the dam = $W \sin \theta$

Reaction forces on the dam = $W \cos \theta$

Reaction forces on the dam = $W \sin \theta$

Total reaction forces = $W \sin \theta$

Reaction forces

$$P = \frac{W}{2}$$

where:

P = Reaction force

W = Weight of dam

γ = Unit weight of material

K_A = Coefficient of active pressure

From Soil Mechanics, Foundations, and Earth Structures by Gregory P. Tschebotarioff.

$$K_A = \frac{\cos^2 (\phi - B)}{\cos^2 B \cos(\delta + B) \left[1 + \frac{\sin(\phi + \delta) \sin(\phi - \omega)}{\cos(\delta + B) \cos(\omega - B)} \right]^2}$$

where:

ϕ = Angle of internal friction

δ = Angle of wall friction

B = Angle of wall inclination

ω = Angle of surcharge

In this case

$$\phi = 30^\circ$$

$$B = 0$$

$$\omega = -20^\circ$$

$$\delta = -22^\circ$$

then:

$$K_A = \frac{\cos^2 22^\circ}{\cos - 22 \left[1 + \sqrt{\frac{\sin(22-22) \sin(22+20)}{\cos(-22) \cos(-20)}} \right]^2}$$

$$K_A = \frac{0.859}{-0.927} = -0.927$$

$$P_H = \frac{116 \times 22^2}{2} (0.927)$$

$$P_H = 2,858,500 \times 0.927$$

$$P_H = 2,649,800 \text{ lbs.}$$

γ = Unit weight of material

K_A = Coefficient of active pressure

From Soil Mechanics, Terzaghi and Peck

Structures by Gregory S. Ghabrion

$\cos^2(\delta - \beta)$

$$K_A = \frac{\cos^2(\delta - \beta)}{\cos^2 \beta \cos(\delta + \beta) \left[1 + \frac{\sin \delta \sin(\delta + \beta)}{\cos^2 \beta} \right]}$$

where:

δ = Angle of internal friction

β = Angle of wall friction

β = Angle of wall inclination

β = Angle of surcharge

In this case

$$\delta = 30^\circ$$

$$\beta = 0^\circ$$

$$\beta = -30^\circ$$

$$\beta = -30^\circ$$

then:

$$K_A = \frac{\cos^2 30^\circ}{\cos^2 0^\circ \cos(-30^\circ) \left[1 + \frac{\sin 30^\circ \sin(-30^\circ)}{\cos^2 0^\circ} \right]}$$

$$K_A = \frac{0.866}{-0.927} = -0.933$$

$$P_H = \frac{118 \times 2.23}{2} (0.933)$$

$$P_H = 2,858,500 \times 0.933$$

$$P_H = 2,659,800 \text{ lbs}$$

$$\text{S.F.} = \frac{5,844,700}{2,649,800} = 2.21$$

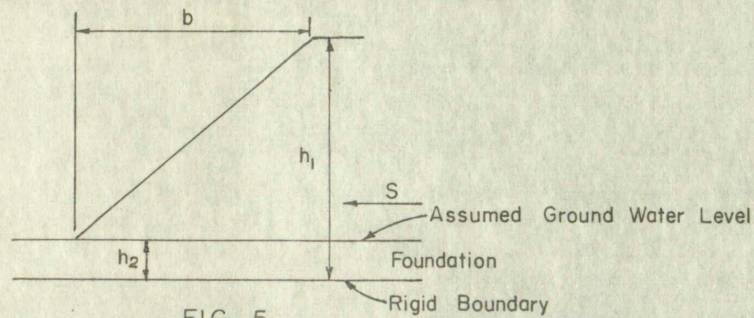
Approximate Shearing Stress in Foundation-

FIG. 5

$$s = \frac{h_1^2 - h_2^2}{2} w \tan^2 \left(45 - \frac{\phi}{2} \right)$$

where:

s = Total horizontal shear down to rigid boundary

h_1 = Vertical distance from top of dam down to rigid boundary

h_2 = Vertical distance from base of dam down to rigid boundary

b = Horizontal shear along base from top shoulder of slope to the toe of the dam

w = Effective weight per cubic foot of the material in its actual condition

ϕ = Angle of internal friction

In this design:

$$h_1 = 242 \text{ ft.}$$

$$h_2 = 20 \text{ ft.}$$

$$b = 628 \text{ ft.}$$

$$\phi = 30^\circ$$

$$= 120 \text{ lb. per cu. ft.}$$

Approximate Bascant Stress in Soil

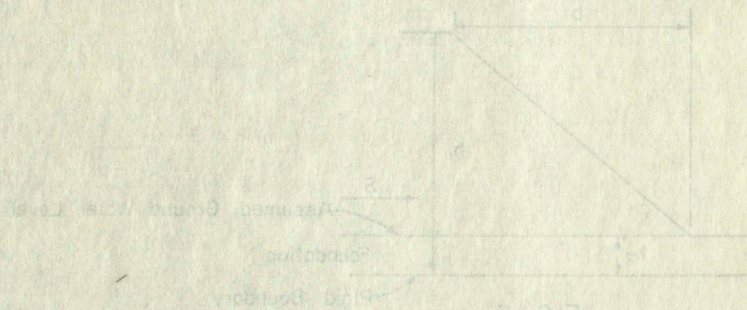


FIG. 3

$$a = \frac{h_1^2 - h_2^2}{2} \tan \phi + \frac{h_1^2 - h_2^2}{2}$$

where:

a = Total horizontal shear from the fixed boundary

h₁ = Vertical distance from the failure surface to the fixed boundary

boundary

h₂ = Vertical distance from the failure surface to the fixed boundary

boundary

b = Horizontal shear along base from the fixed boundary

slope to the toe of the dam

W = Effective weight per cubic foot of the water at

in its actual condition

phi = Angle of internal friction

In this design:

$$h_1 = 24.5 \text{ ft.}$$

$$h_2 = 20 \text{ ft.}$$

$$b = 628 \text{ ft.}$$

$$phi = 30^\circ$$

$$= 120 \text{ lb. per cu. ft.}$$

$$\text{Equivalent liquid unit weight} = \tan^2 \left(45 - \frac{30}{2} \right)$$

$$= 120 \tan^2 30 = 120 \times 0.333 = 40.0 \text{ lb/cu.ft.}$$

Total shear in foundation along b

$$s = \frac{242^2 - 20^2}{20} \times 40.0 = 1,163,300 \text{ lbs.}$$

$$s = 581.7 \text{ tons}$$

$$s_a = \frac{s}{b} = \frac{581.7}{628} = 0.93 \text{ ton per sq. ft.}$$

Approximate Factor of Safety Against Foundation Shear.

Assuming water table at ground surface

$$\text{Unit shear strength below toe} = 20 \times 79.3 \times 0.649 =$$

$$= 1029 \text{ lb. per sq. ft.} = 0.51 \text{ ton per sq. ft.}$$

At point in foundation under upper shoulder of slope =

$$= w_1 = \frac{(20 \times 64.2) + (222 \times 120.0)}{242} = \frac{27,924}{242}$$

$$w_1 = 115.4 \text{ lb. per cu. ft.}$$

$$\text{Shear strength at this point} = 242 \times 115.4 \times 0.404 =$$

$$= 11,280 \text{ lb. per sq. ft.} = 5.64 \text{ tons per sq. ft.}$$

$$\text{Average unit shear} = \frac{0.51 + 5.64}{2} = 3.08 \text{ tons per sq.ft.}$$

$$\text{F.S.} = \frac{3.08}{0.93} = 3.31$$

Equivalent weight of the sample = 1.12 g

Weight of the sample = 1.12 g

Total weight of the sample = 1.12 g

$$x = \frac{1.12 - 0.02}{1.12} = 0.982$$

$$x = 0.982$$

$$x = \frac{0.02}{1.12} = 0.018$$

Approximate weight of the sample = 1.12 g

Approximate weight of the sample = 1.12 g

Unit weight of the sample = 1.12 g

Weight of the sample = 1.12 g

At point in the sample the weight of the sample = 1.12 g

$$W_1 = \frac{(1.12 - 0.02) \times 100}{1.12} = 98.2$$

$$W_1 = 98.2 \text{ per cent}$$

Shear strength of the sample = 1.12 g

Weight of the sample = 1.12 g

$$\text{Average unit shear} = \frac{0.02 + 0.02}{2} = 0.02$$

$$x = \frac{0.02}{1.12} = 0.018$$

APPENDIX B

PLATES

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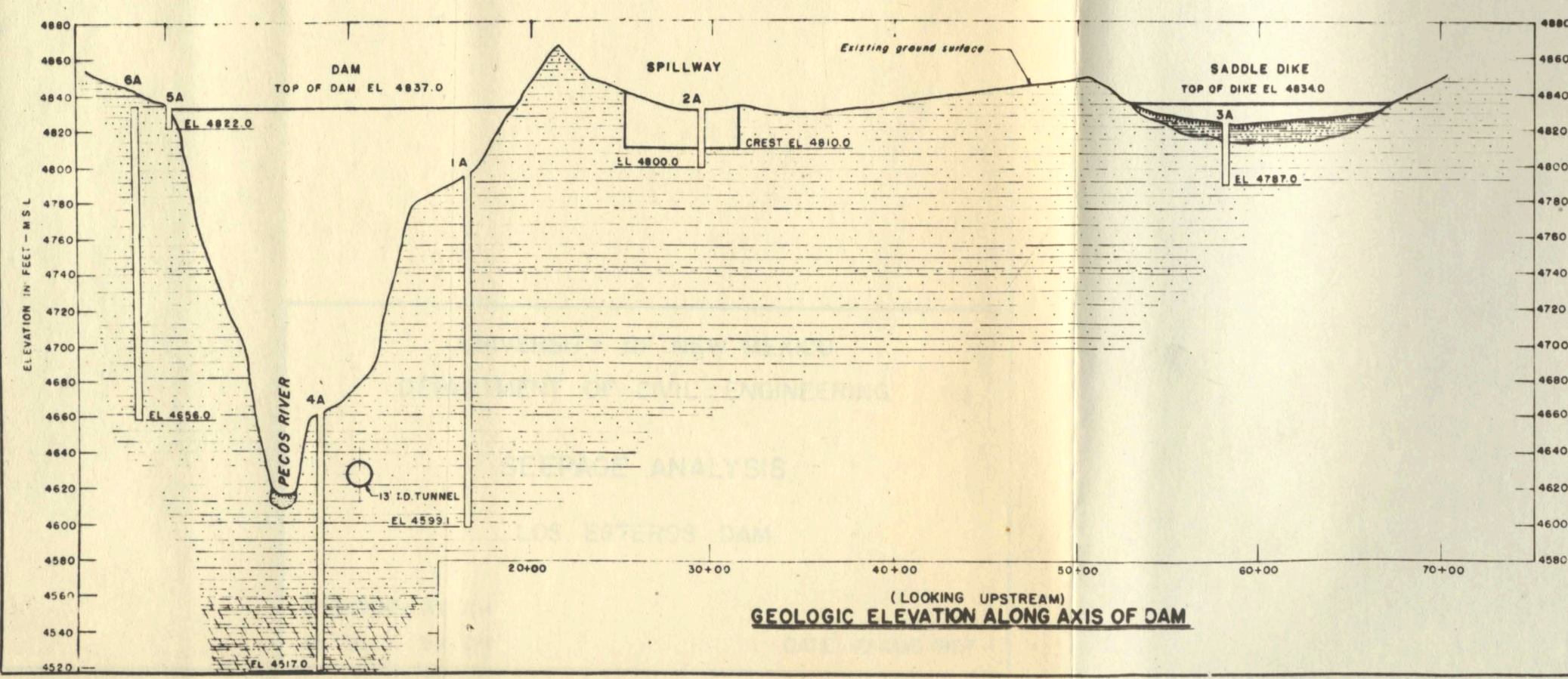
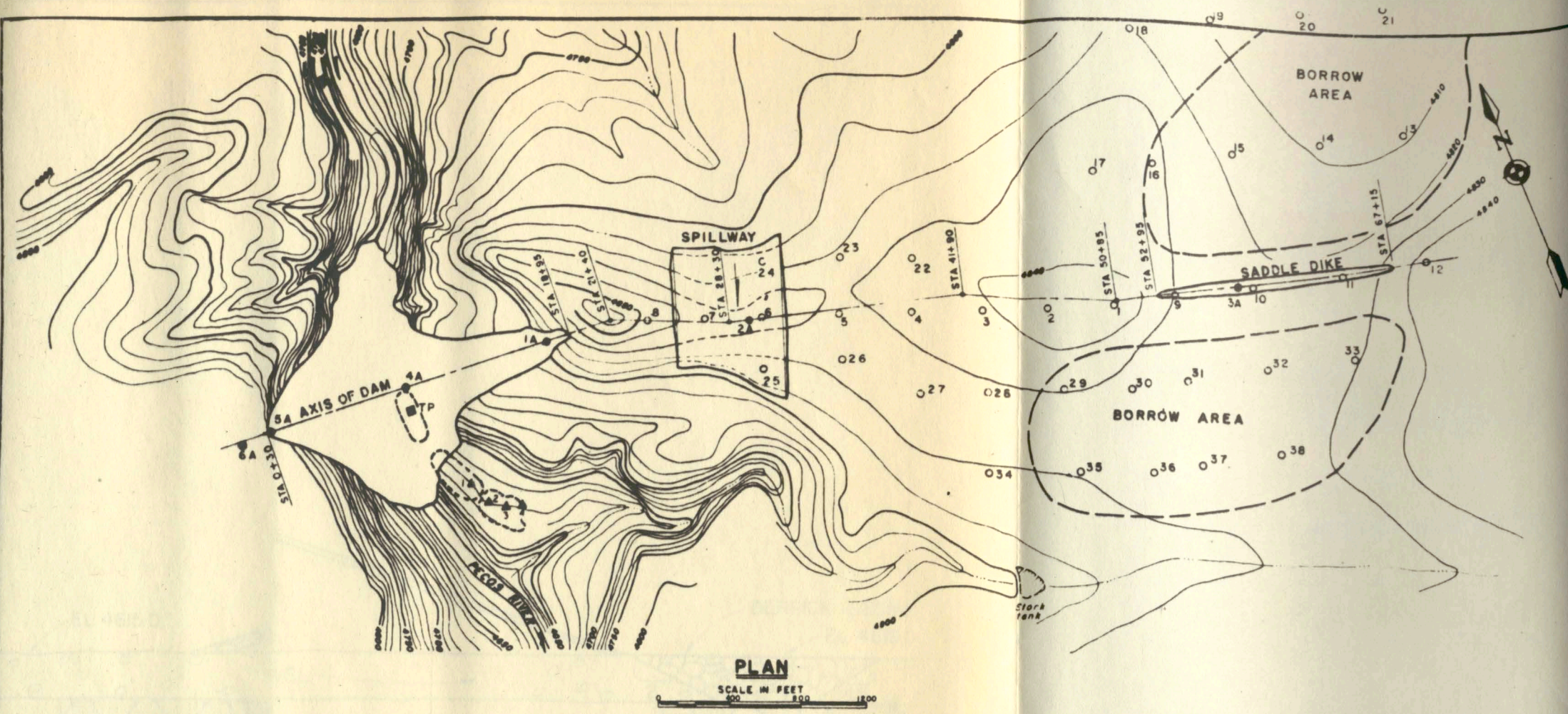
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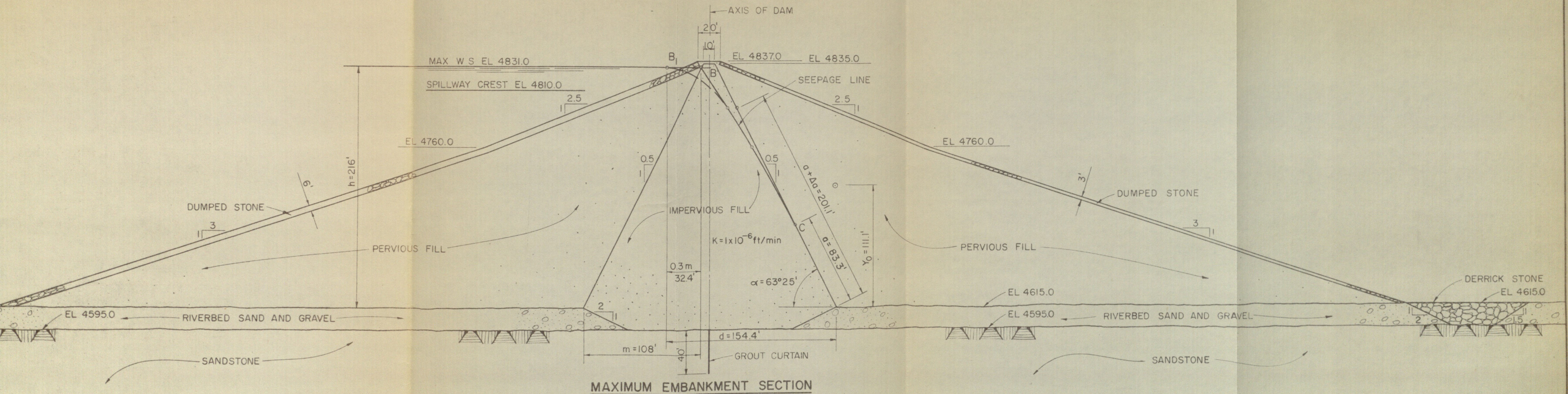
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- LEGEND**
- CORE HOLE
 - AUGER HOLE
 - TEST PIT
 - ▲ TEST HOLE
 - GRAVEL BAR
 - ▨ OVERBURDEN
 - ▨ SANDSTONE INTERBEDDED WITH SHALE
 - ▨ DOLOMITIC LIMESTONE INTERBEDDED WITH SANDSTONE AND SHALE

UNIVERSITY OF NEW MEXICO
 DEPARTMENT OF CIVIL ENGINEERING
 BORROW & SUB-SURFACE INVESTIGATIONS
 LOS ESTEROS DAM

DESIGNED BY: CH
 DETAILED BY: CH
 DATE: 24 AUG 1957
 PLATE 7



SEEPAGE ANALYSIS

$$Y_0 = \sqrt{h^2 + d^2} - d$$

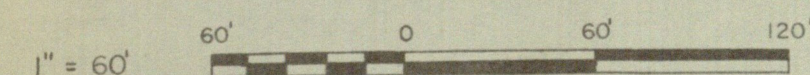
$$Y_0 = \sqrt{216^2 + 154.4^2} - 154.4 = 265.5 - 154.4 = 111.1$$

$$\alpha + \Delta\alpha = \frac{Y_0}{1 - \cos \alpha} = \frac{111.1}{1 - 0.4475} = 201.1'$$

$$\alpha = \frac{3}{4} Y_0 = \frac{3}{4} \times 111.1 = 83.3'$$

$$q = K (\sqrt{d^2 + h^2} - d) = 0.00001 \times 111.1 = 0.001111 \text{ cu ft per min per ft of length}$$

GRAPHIC SCALE



UNIVERSITY OF NEW MEXICO
DEPARTMENT OF CIVIL ENGINEERING

SEEPAGE ANALYSIS

LOS ESTEROS DAM

DESIGNED BY: CH
DETAILED BY: CH

DATE: 10 AUG 1957

PLATE 8

| SOIL CONSTANTS FOR STABILITY ANALYSIS | | | | |
|---------------------------------------|------------------------------|----------------|------------------|-----|
| MATERIAL | φ ANGLE OF INTERNAL FRICTION | C COHESION PSF | UNIT WEIGHTS PCF | |
| | | | OPTIMUM | DRY |
| PERVIOUS | 33° | 0 | 135 | |
| IMPERVIOUS | 22° | 0 | 116 | |
| ROCK | 45° | 0 | | 100 |

| Σ NORMAL FORCES | | | | | |
|-----------------|---------|----------|-------------|-----------|-----------|
| 1 | 2 | 3 | 4 | 5 | 6 |
| MATERIAL | TAN 33° | AREA, SF | WEIGHT, PCF | N, LBS | N x TAN φ |
| PERVIOUS | 0.6494 | 22,100 | 135 | 2,983,500 | 1,937,485 |
| ROCK | 0.6494 | 2,100 | 100 | 210,000 | 136,374 |

TOTAL N-FORCES = 2,073,859 LBS

| Σ TANGENTIAL FORCES | | | |
|---------------------|----------|-------------|-----------|
| MATERIAL | AREA, SF | WEIGHT, PCF | T, LBS |
| PERVIOUS | 7,600 | 135 | 1,026,000 |
| ROCK | 1,000 | 100 | 100,000 |

TOTAL T-FORCES = 1,126,000 LBS

LC = 0

$$FS = \frac{\Sigma N \tan \phi + LC}{\Sigma T} = \frac{2,073,859}{1,126,000} = 1.84$$

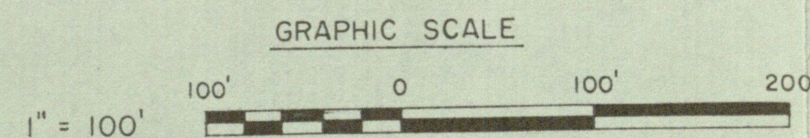
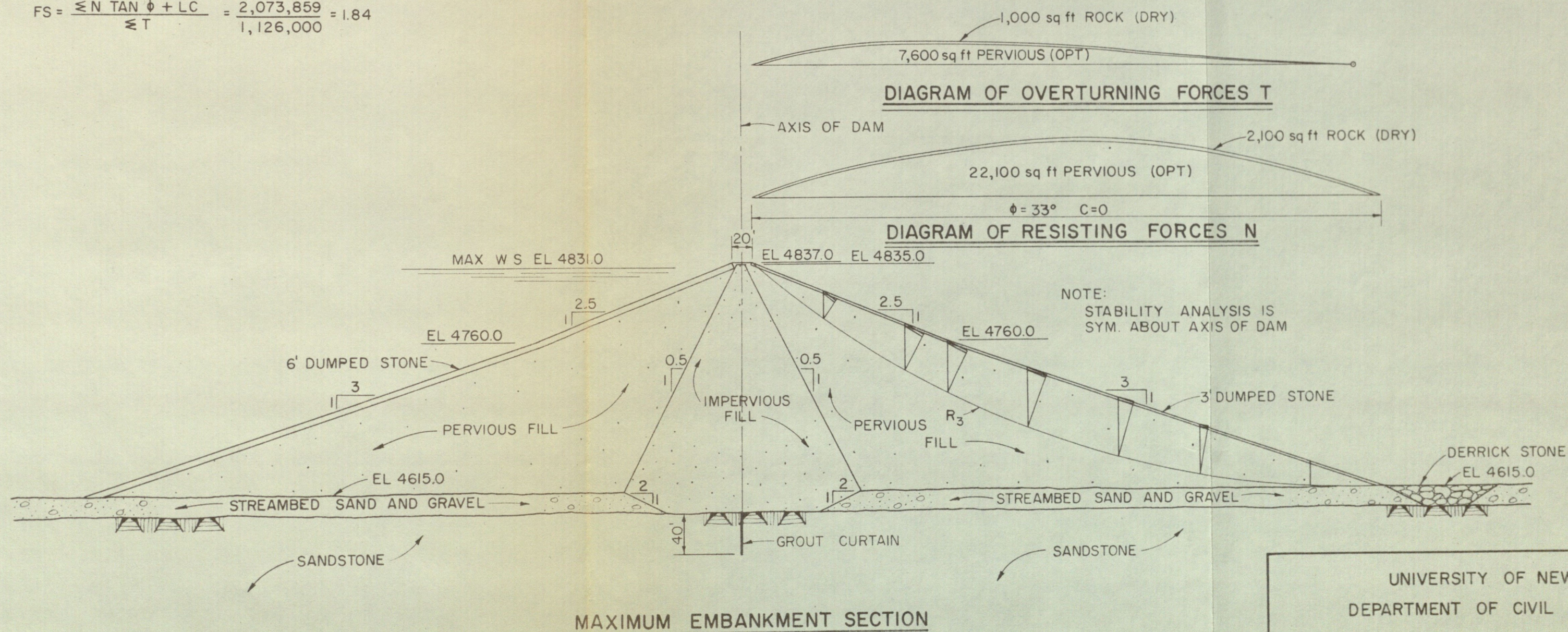
4
○
1.86

6
○
1.87

3
○
1.84

7
○
1.89

2
○
1.93



DESIGNED BY: CH
DETAILED BY: CH

DATE: 10 AUG 1957

PLATE 9

| SOIL CONSTANTS FOR STABILITY ANALYSIS | | | | |
|---------------------------------------|------------------------------|----------------|------------------|-----|
| MATERIAL | φ ANGLE OF INTERNAL FRICTION | C COHESION PSF | UNIT WEIGHTS PCF | |
| | | | OPTIMUM | DRY |
| PERVIOUS | 33° | 0 | 135 | |
| IMPERVIOUS | 22° | 0 | 116 | |
| ROCK | 45° | 0 | | 100 |

| Σ NORMAL FORCES | | | | | |
|-----------------|---------|---------|------------|---------|-----------|
| 1 | 2 | 3 | 4 | 5 | 6 |
| MATERIAL | TAN 33° | AREA SF | WEIGHT PCF | N, LBS | N x TAN φ |
| PERVIOUS | 0.6494 | 1,952 | 135 | 263,520 | 171,130 |
| ROCK | 0.6494 | 544 | 100 | 54,400 | 35,327 |

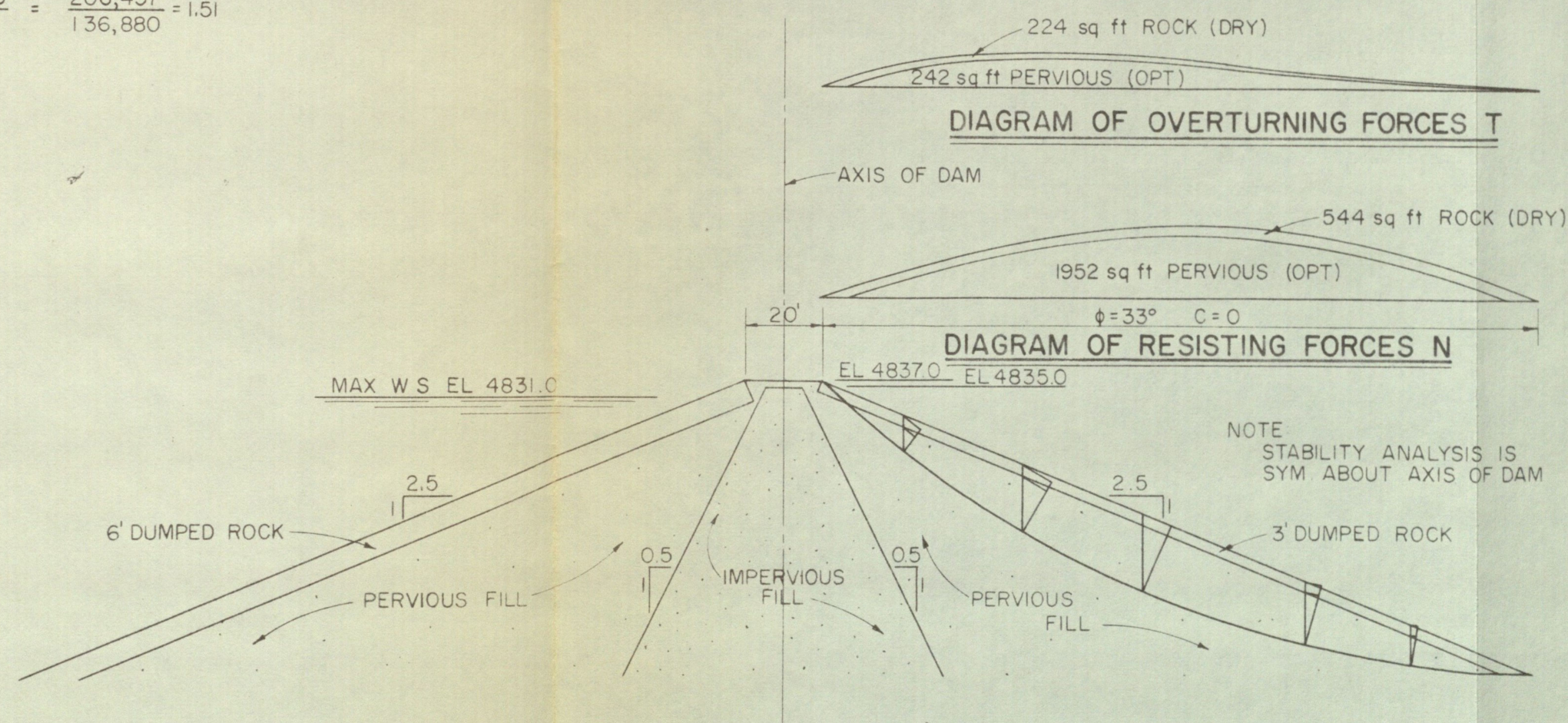
TOTAL N-FORCES = 206,457 LBS

| Σ TANGENTIAL FORCES | | | |
|---------------------|---------|------------|---------|
| MATERIAL | AREA SF | WEIGHT PCF | T, LBS |
| PERVIOUS | 848 | 135 | 114,480 |
| ROCK | 224 | 100 | 22,400 |

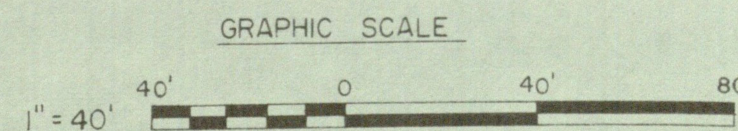
TOTAL T-FORCES = 136,880 LBS

$$\widehat{LC} = 0$$

$$F.S. = \frac{\Sigma N \tan \phi + \widehat{LC}}{\Sigma T} = \frac{206,457}{136,880} = 1.51$$



EMBANKMENT SECTION AT ELEVATION 4760.0



3
0
1.59

4
0
1.82

2
0
1.51

6
0
1.79

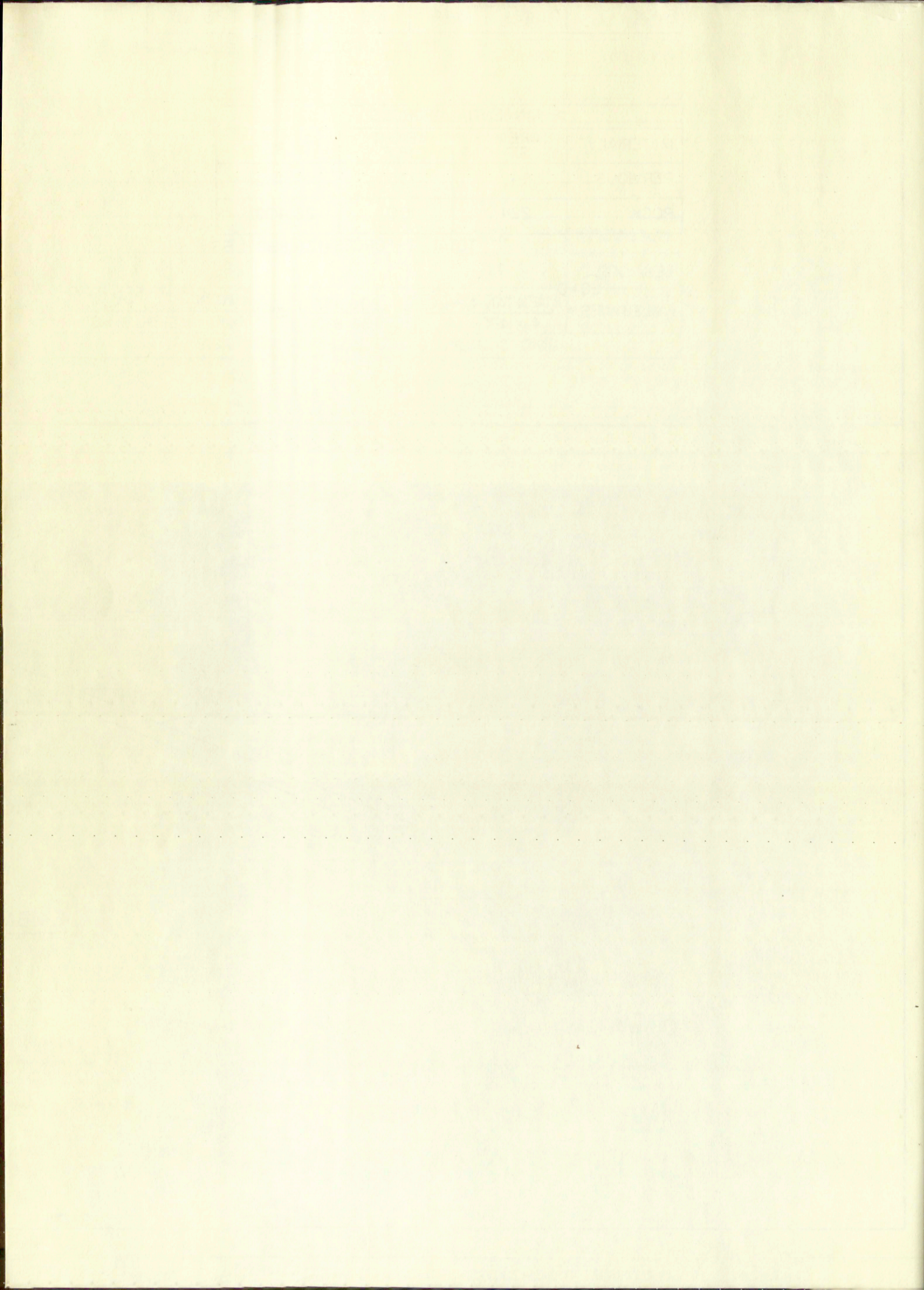
5
0
1.74

UNIVERSITY OF NEW MEXICO
DEPARTMENT OF CIVIL ENGINEERING
STABILITY ANALYSIS
UPSTREAM & DOWNSTREAM SLOPES
LOS ESTEROS DAM

DESIGNED BY: CH
DETAILED BY: CH

DATE: 24 AUG 1957

PLATE 10



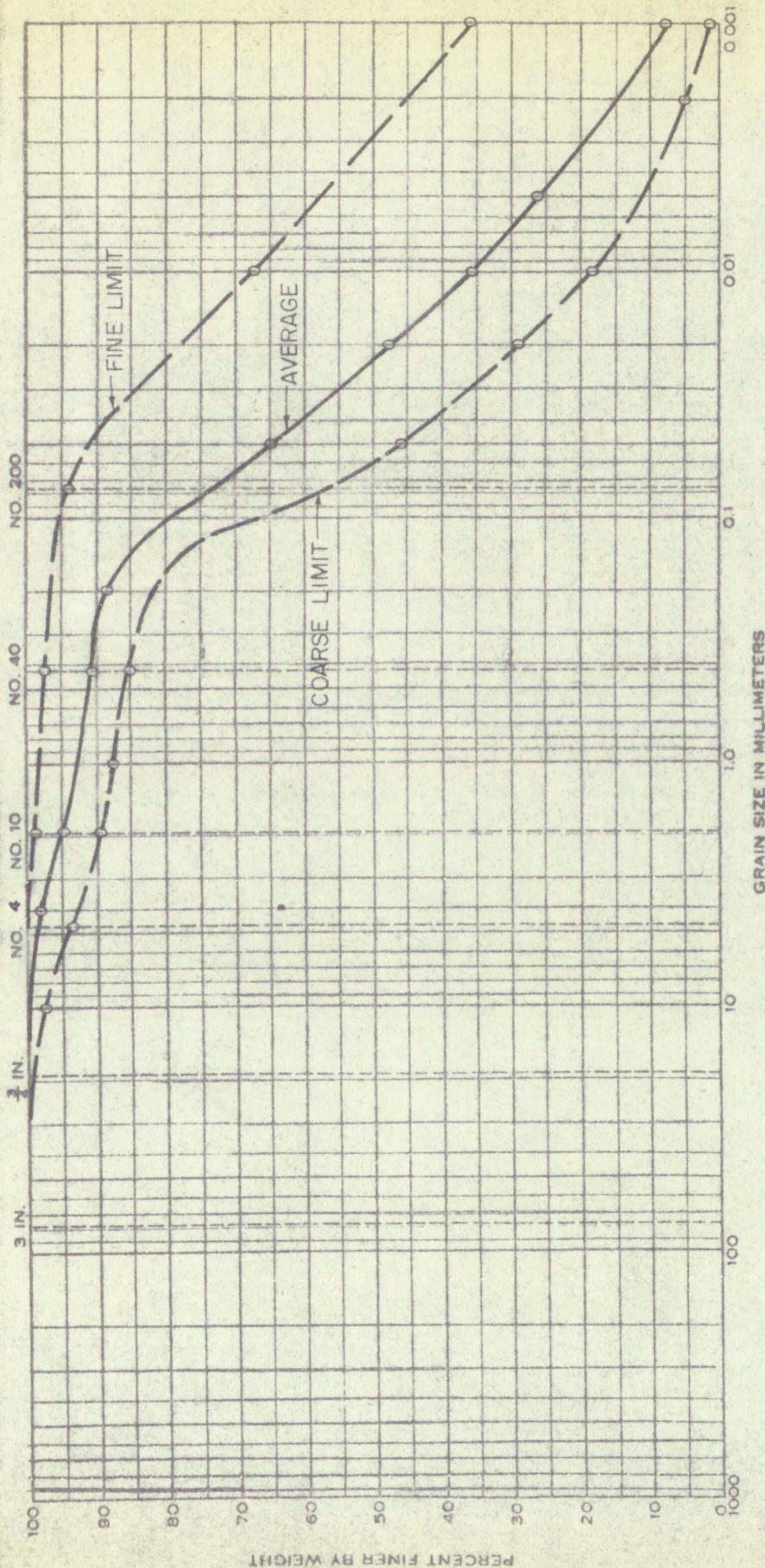
APPENDIX C

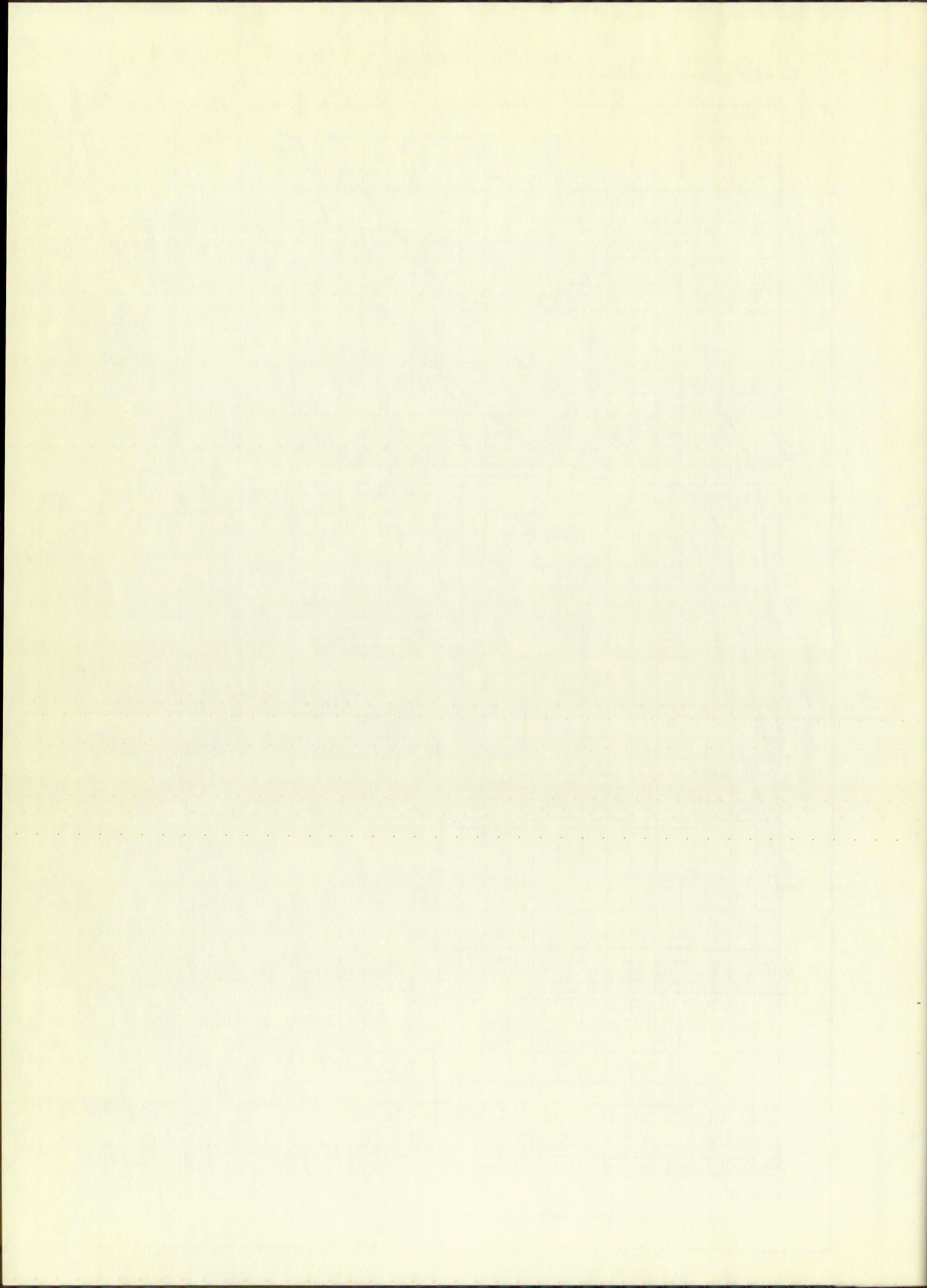
TEST RESULTS

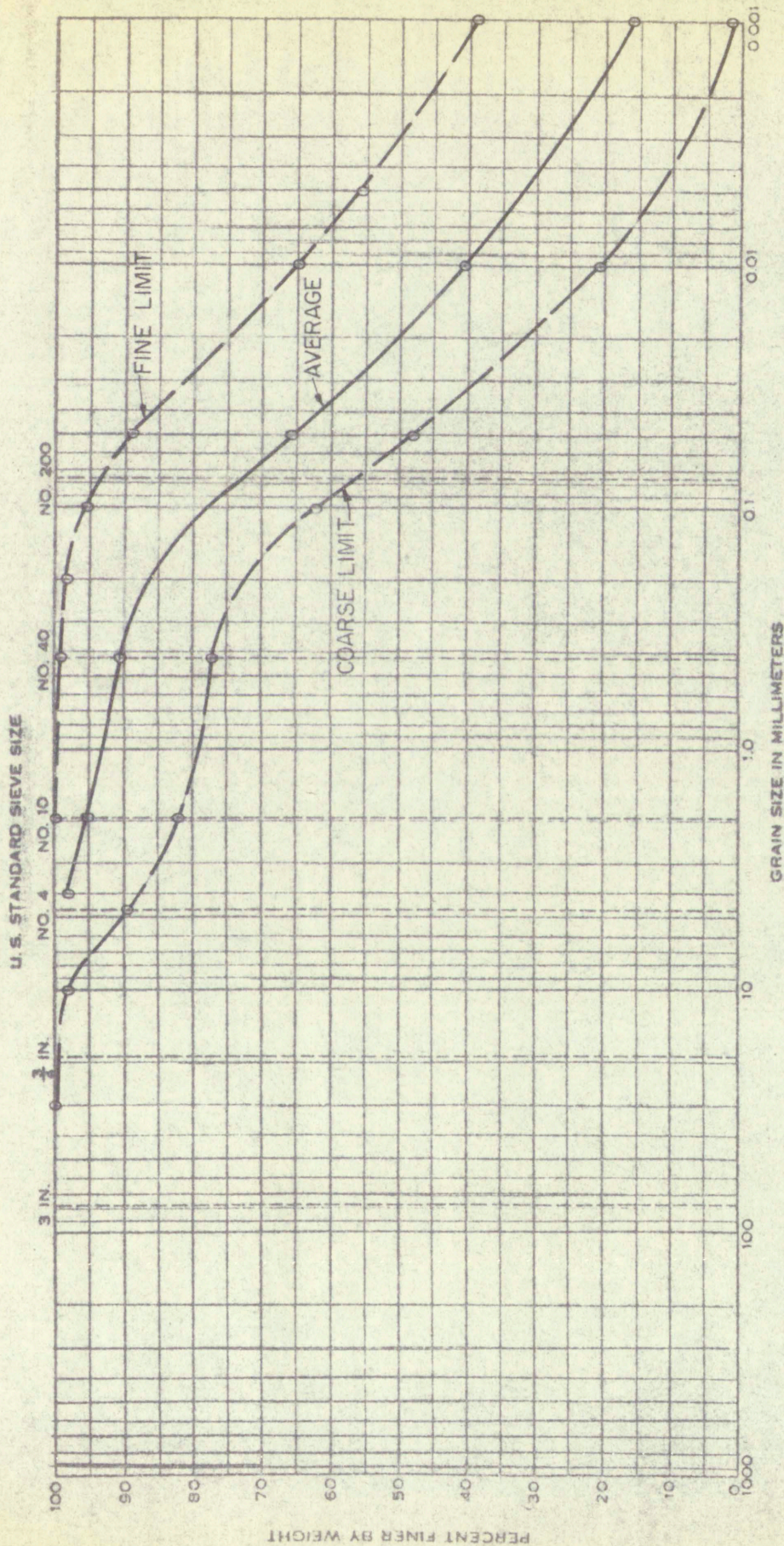
APPENDIX C

TEST RESULTS

U.S. STANDARD SIEVE SIZE

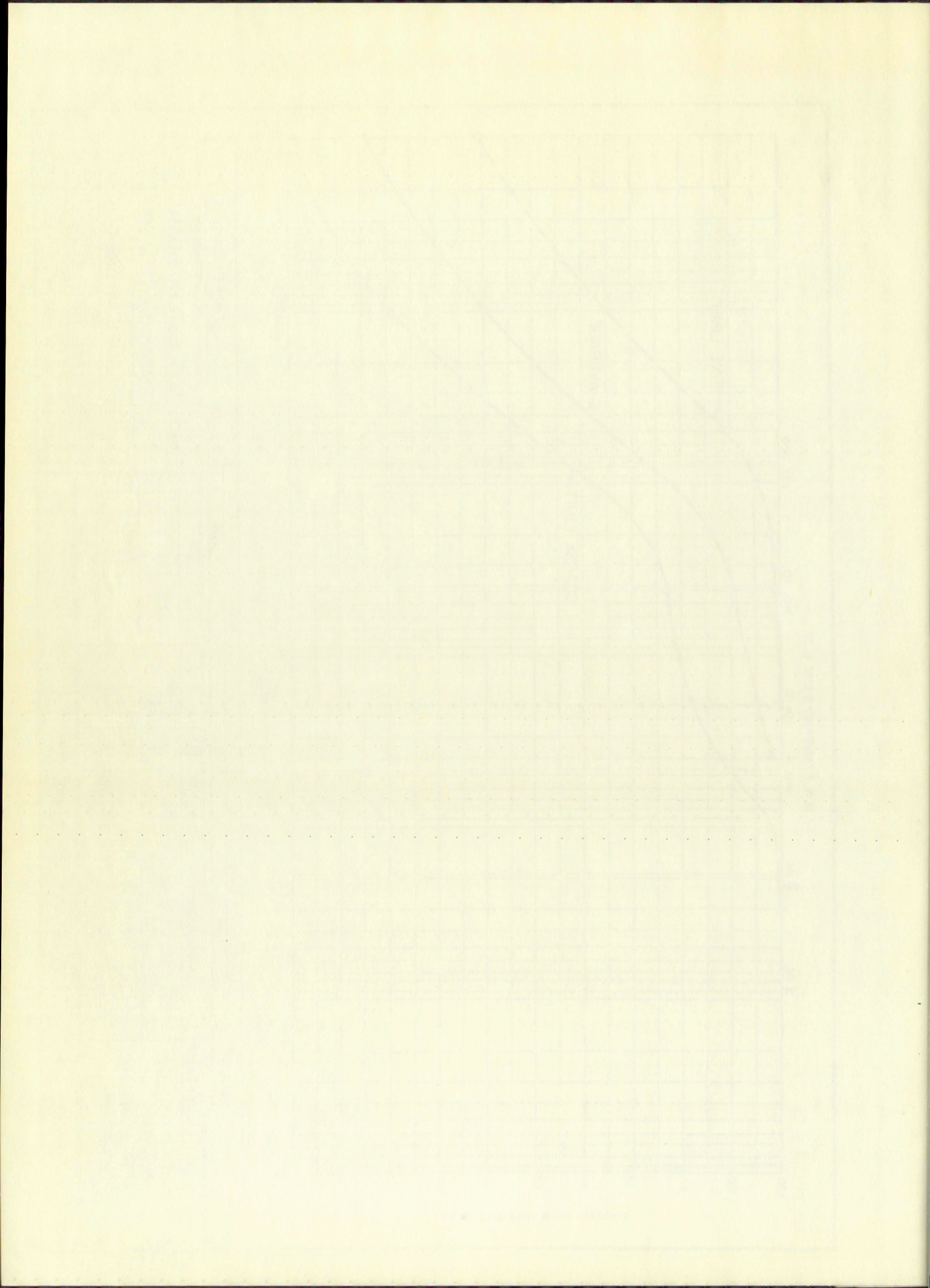


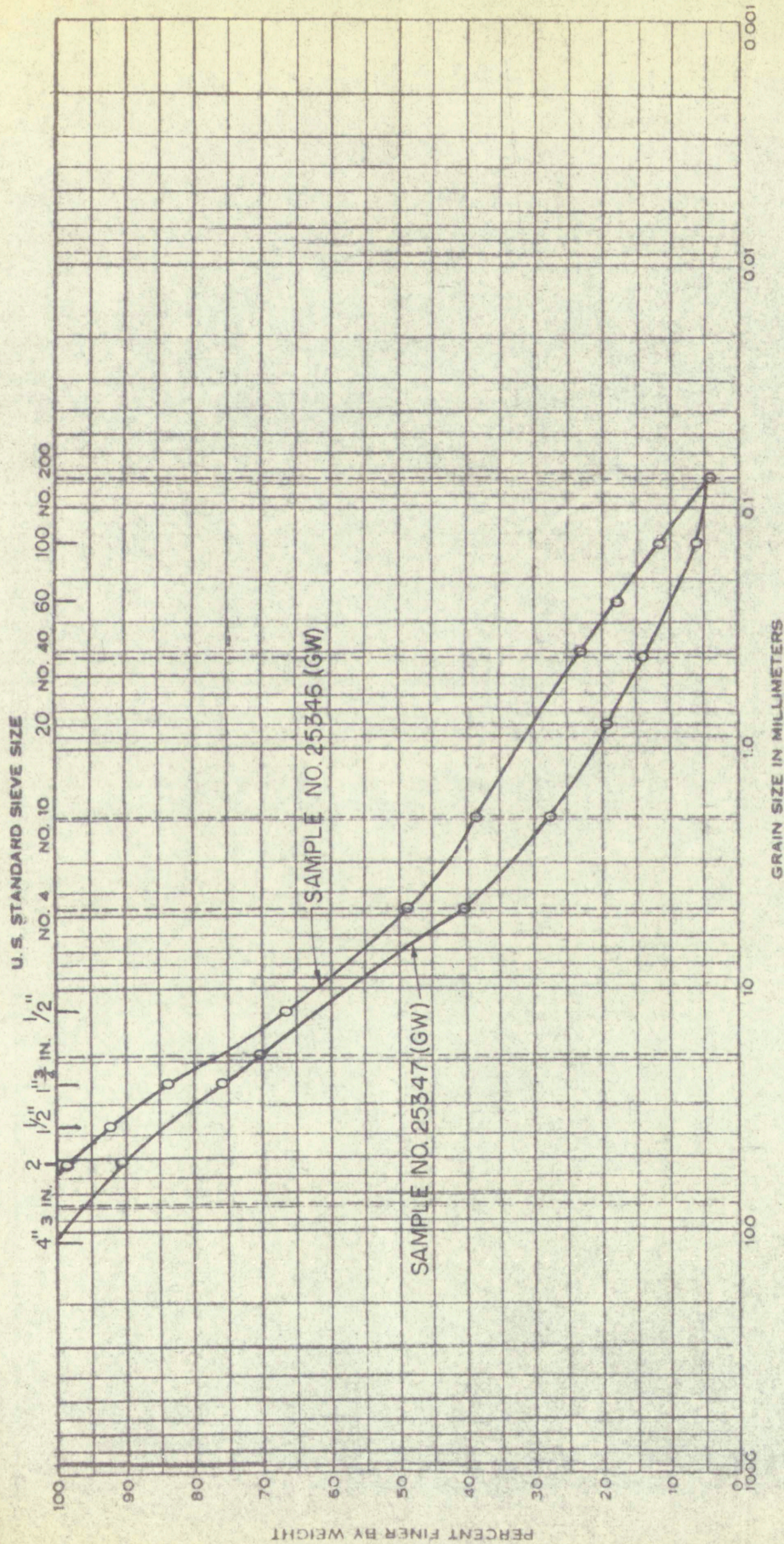


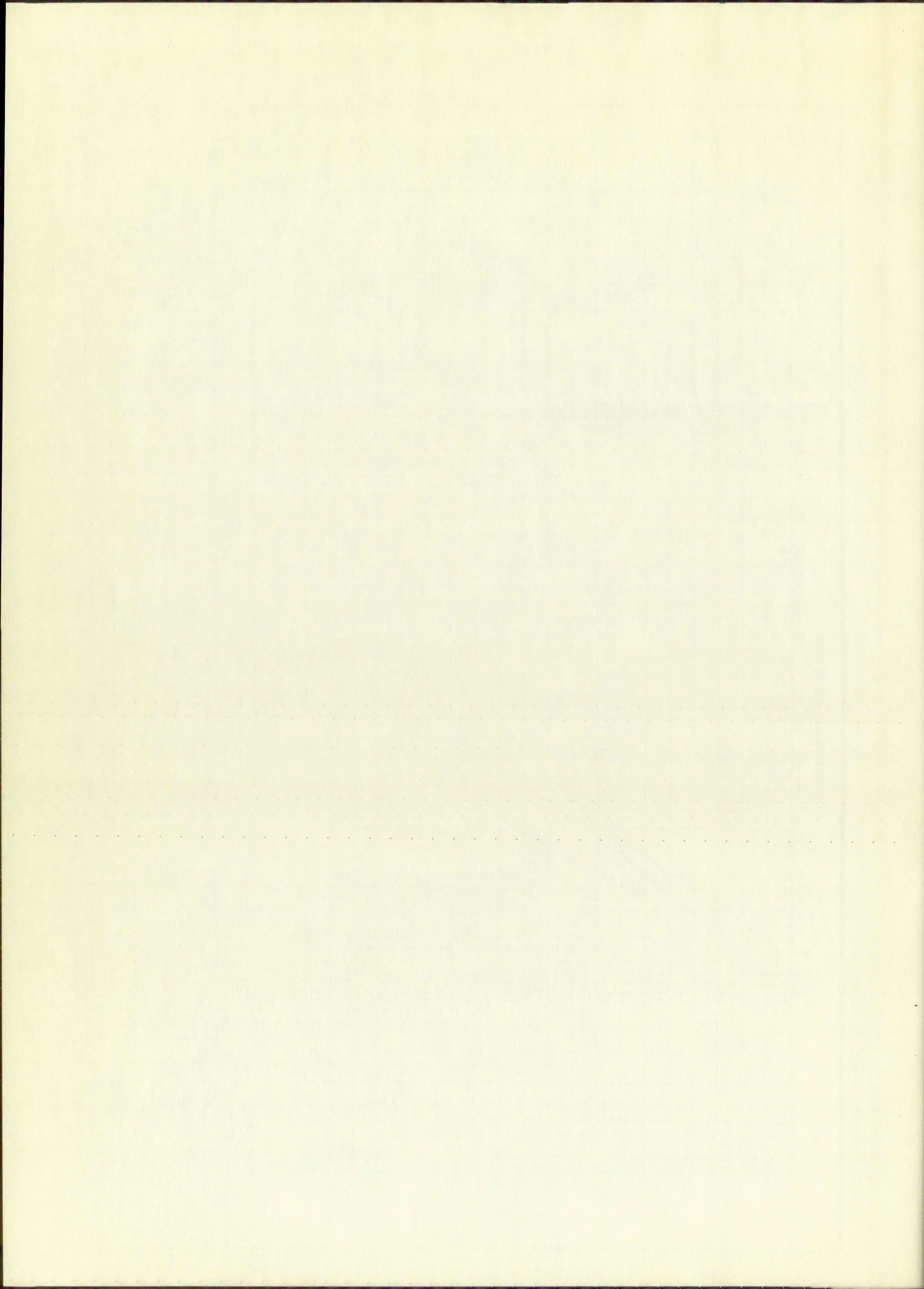


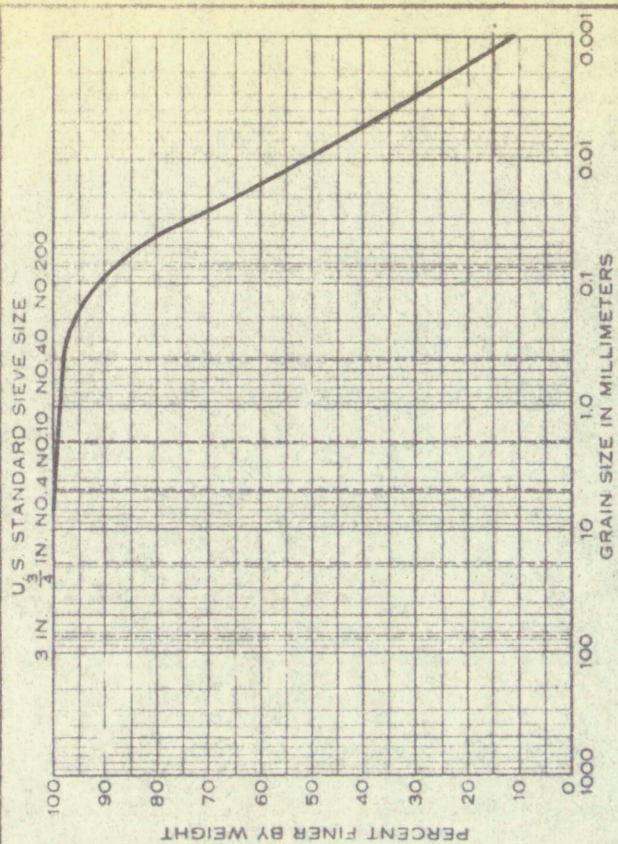
| COBBLES | | GRAVEL | | SAND | | | SILT OR CLAY | |
|---------|--|--------|------|--------|--------|------|--------------|--|
| | | Coarse | Fine | Coarse | Medium | Fine | | |

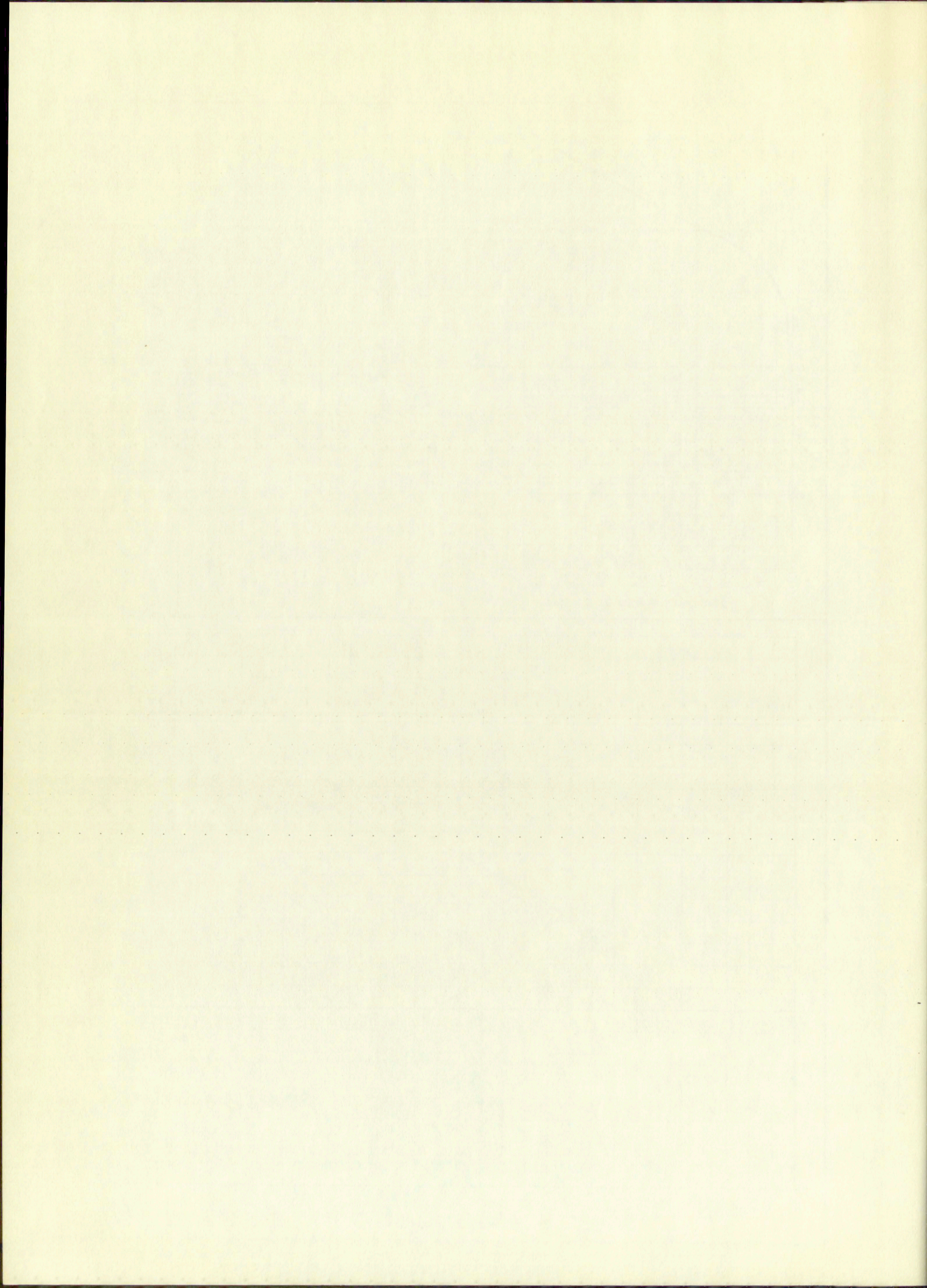
| | | | | | | | | |
|------------------------------|---------------|--------------------------------|-------|----|----|----|---------------------|------------------|
| Sample No. | Elev or Depth | Classification | NatWC | LL | PL | PI | Project | |
| AVERAGE | | CLAYEY SAND (SC) | | 29 | 18 | 11 | LOS ESTEROS DAMSITE | |
| FINE LIMIT | | CLAYEY SAND (SC) | | 45 | 23 | 22 | EMBANKMENT MATERIAL | |
| COARSE LIMIT | | CLAYEY SAND-SILTY SAND (SC-ML) | | 17 | 15 | 2 | BORROW AREA "B" | |
| NOTE: 69 SAMPLES REPRESENTED | | | | | | | Spring No. | COMPOSITE SAMPLE |
| | | | | | | | Date | 20 JULY 1957 |
| GRADATION CURVES | | | | | | | | |



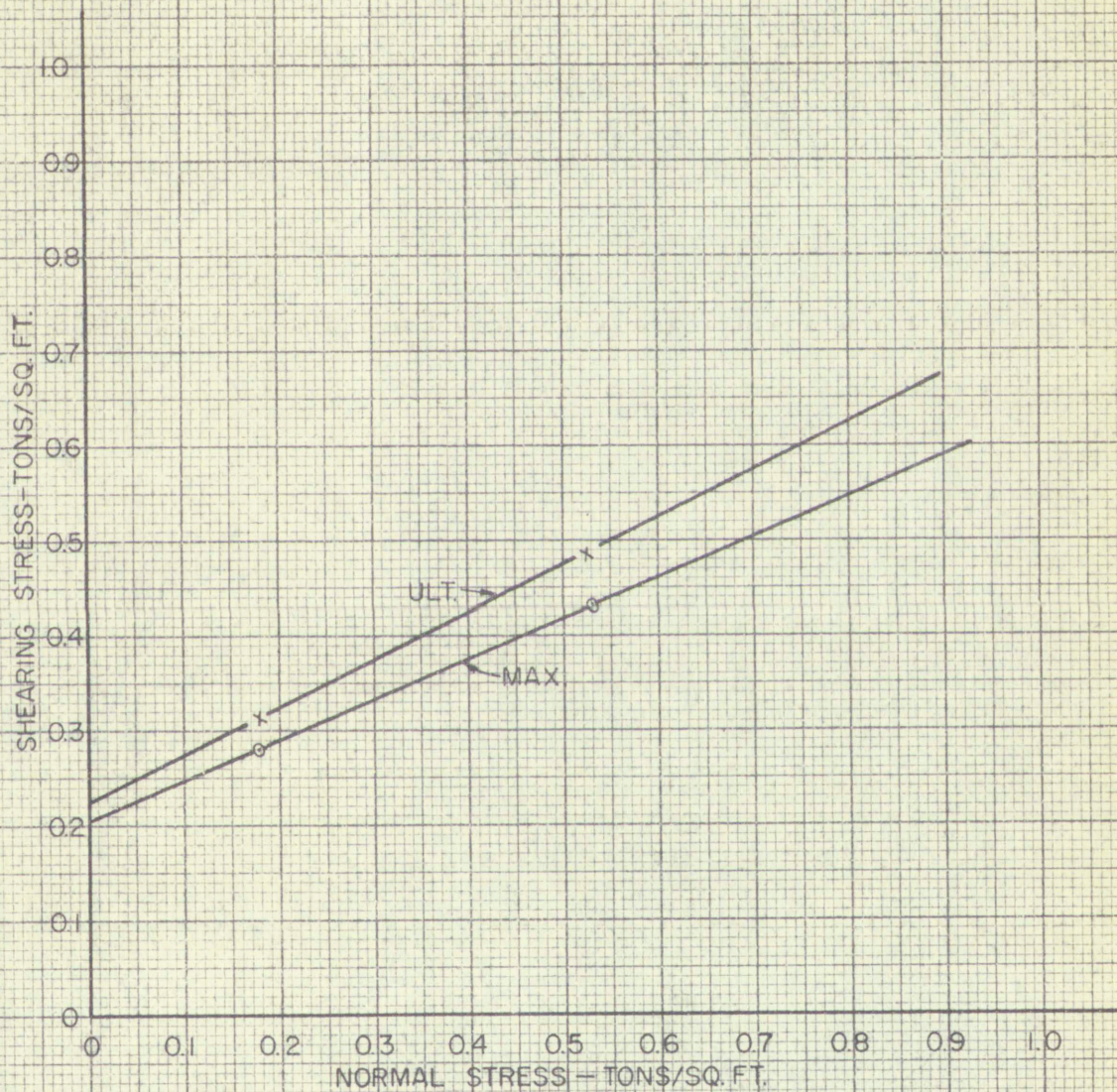






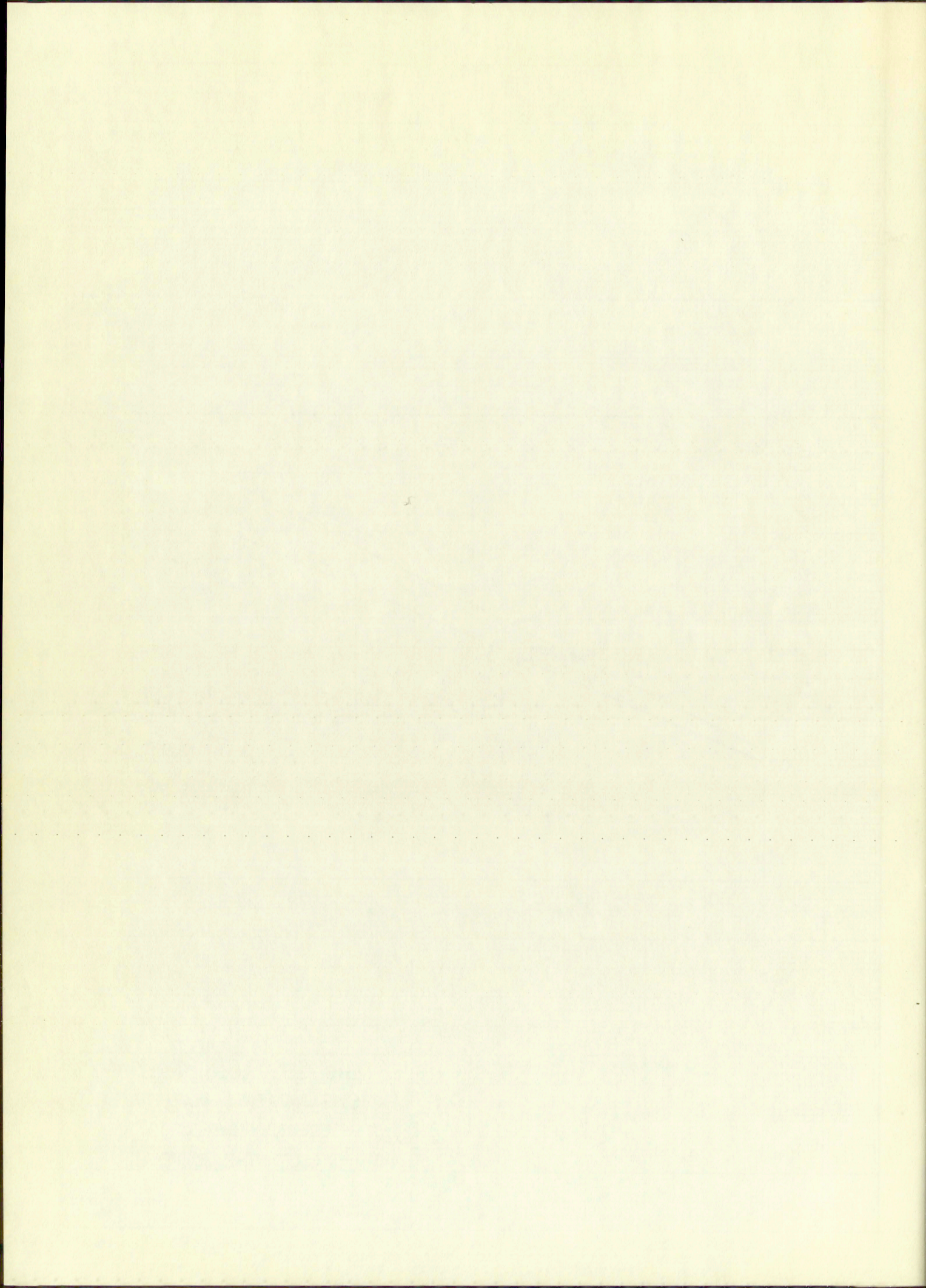


| TEST RESULTS | | |
|--------------|---------|---------|
| | MAX | ULT |
| ϕ | 22° 37' | 26° 31' |
| TAN ϕ | 0.4166 | 0.4991 |
| C-T/SF | 0.21 | 0.23 |

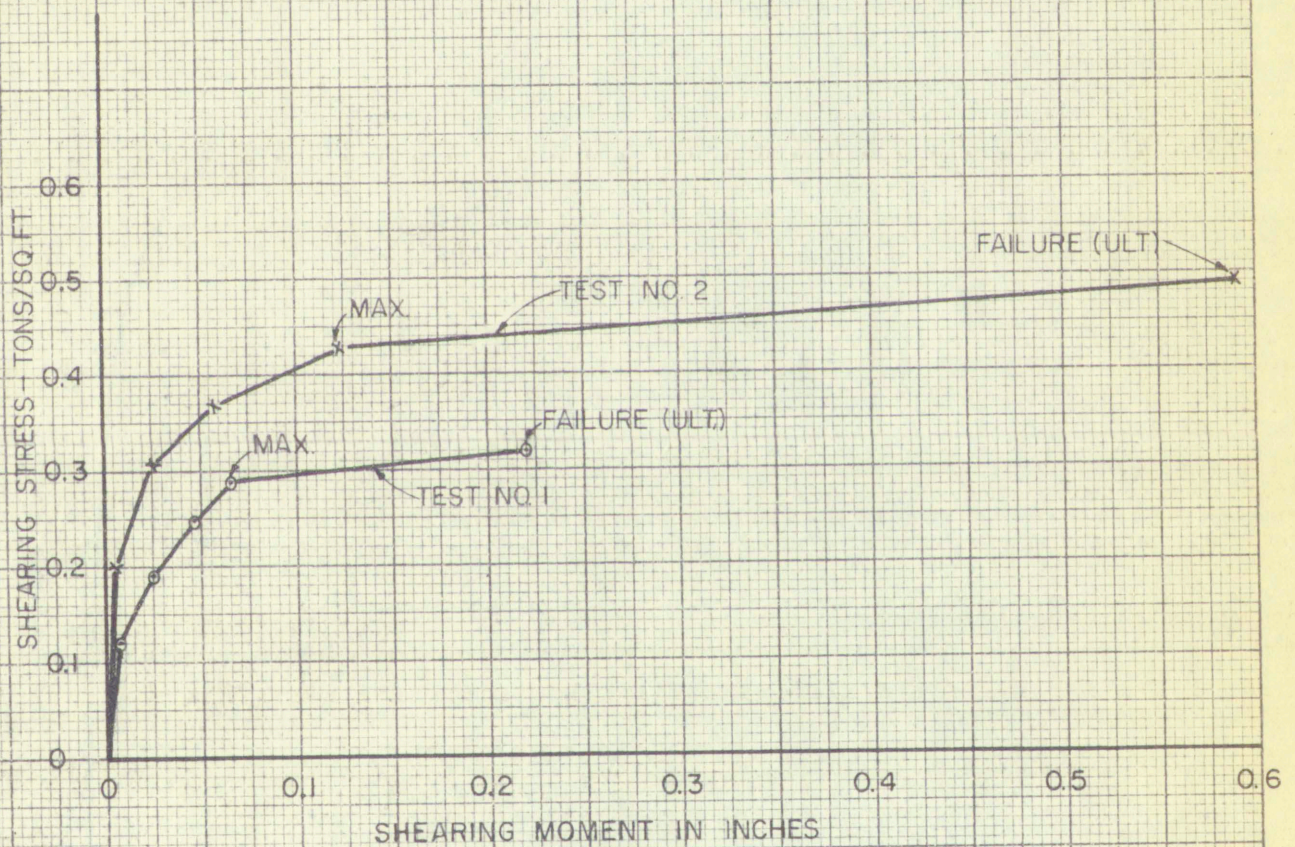


LOS ESTEROS DAMSITE
EMBANKMENT MATERIAL
BORROW AREA "B"
DIRECT SHEAR TEST

DATE: 17 JULY 1957

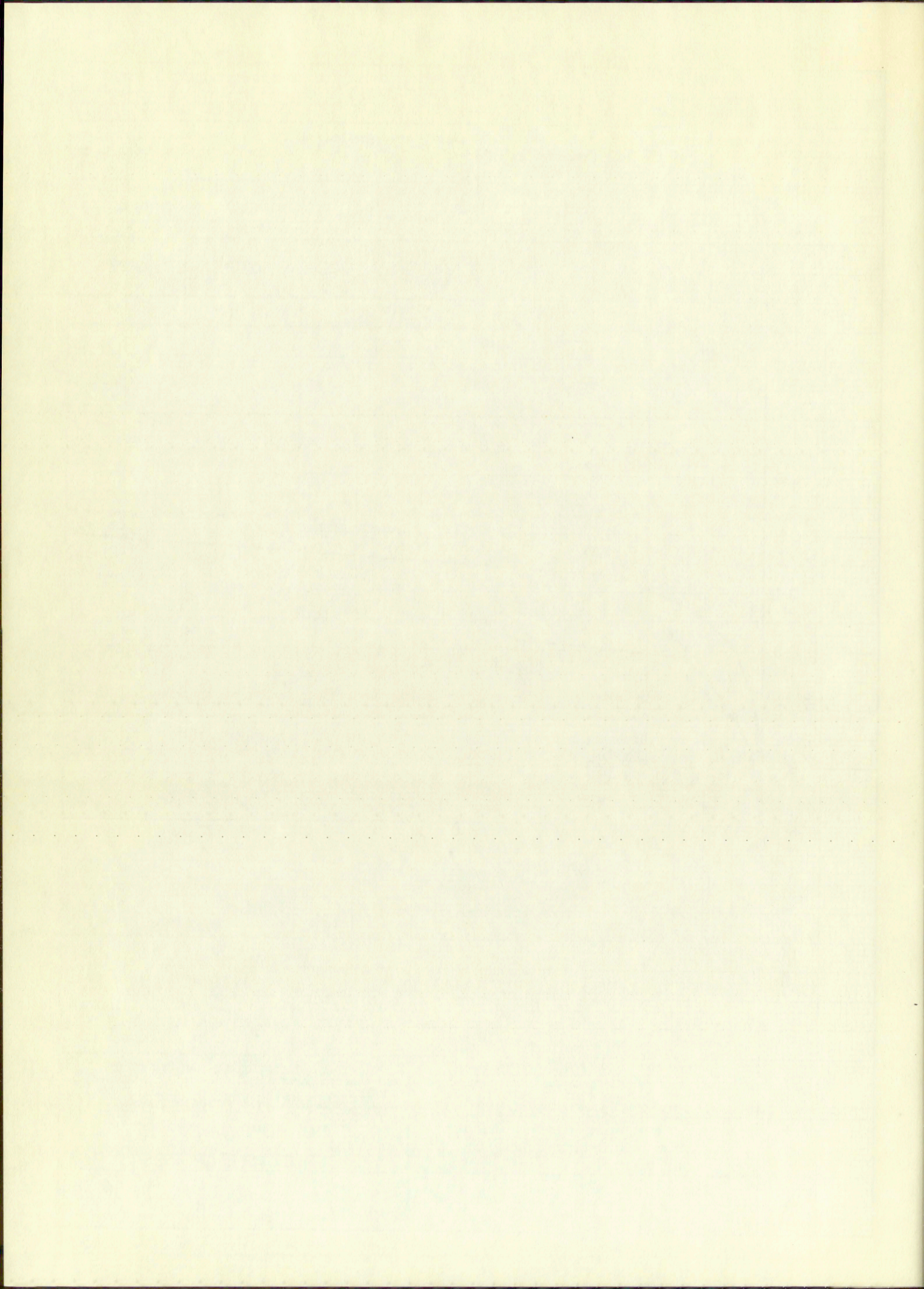


| TEST DATA | NORMAL LOAD — T/S.F. | |
|----------------------------------|----------------------|--------|
| | 0.1805 | 0.5305 |
| PERCENT OF MOIST. BEFORE TEST | 19.0 | 18.5 |
| PERCENT OF MOIST. AFTER TEST | 18.8 | 18.4 |
| DRY WT. LBS. PER CU. FT. | 107.8 | 106.9 |
| TEST NO. | 1 | 2 |



LOS ESTEROS DAMSITE
EMBANKMENT MATERIAL
BORROW AREA "B"
DIRECT SHEAR TEST

DATE: 17 JULY 1957



U. S. STANDARD SIEVE SIZE

NO. 200

NO. 40

NO. 10

NO. 4

3/4 IN.

3 IN.

100

90

80

70

60

50

40

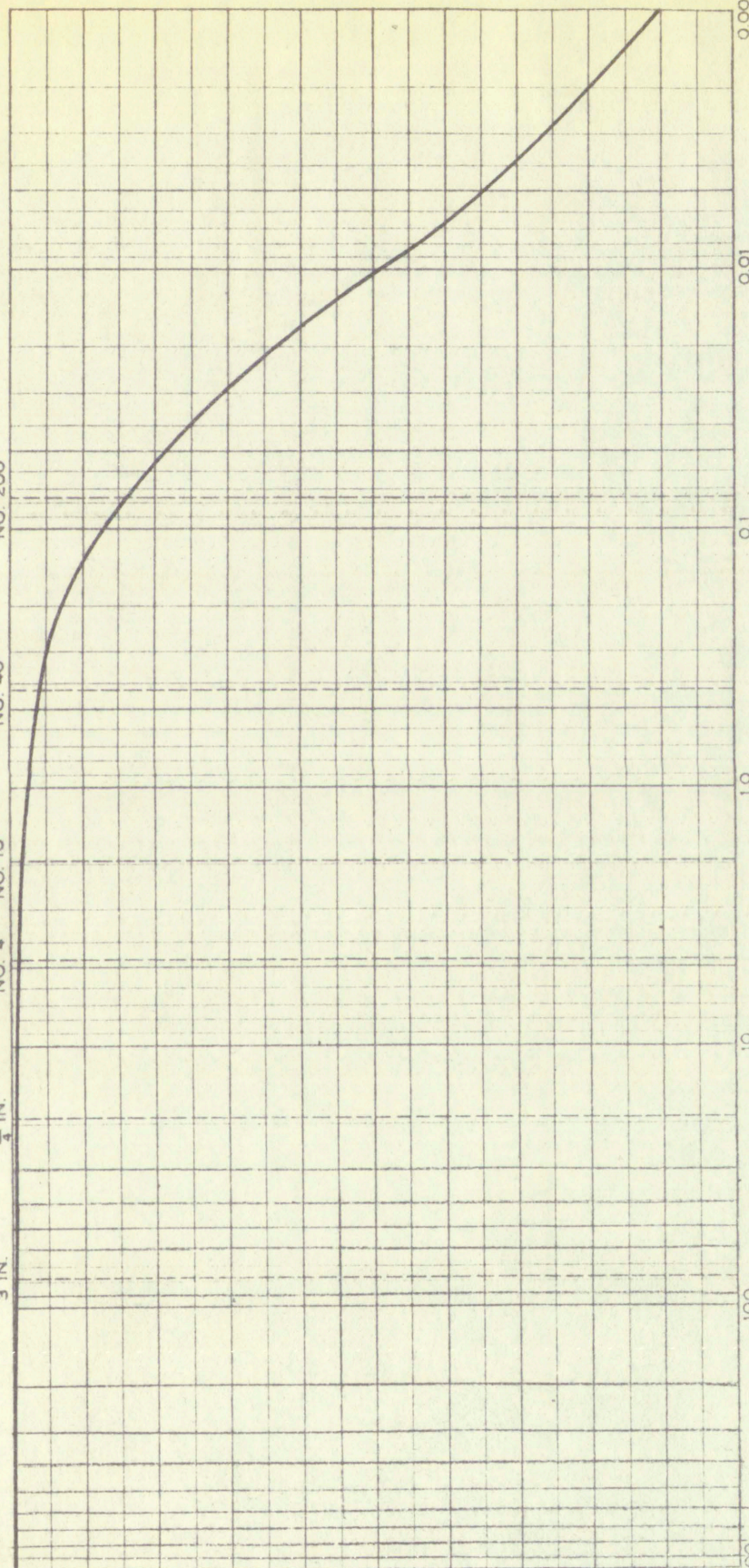
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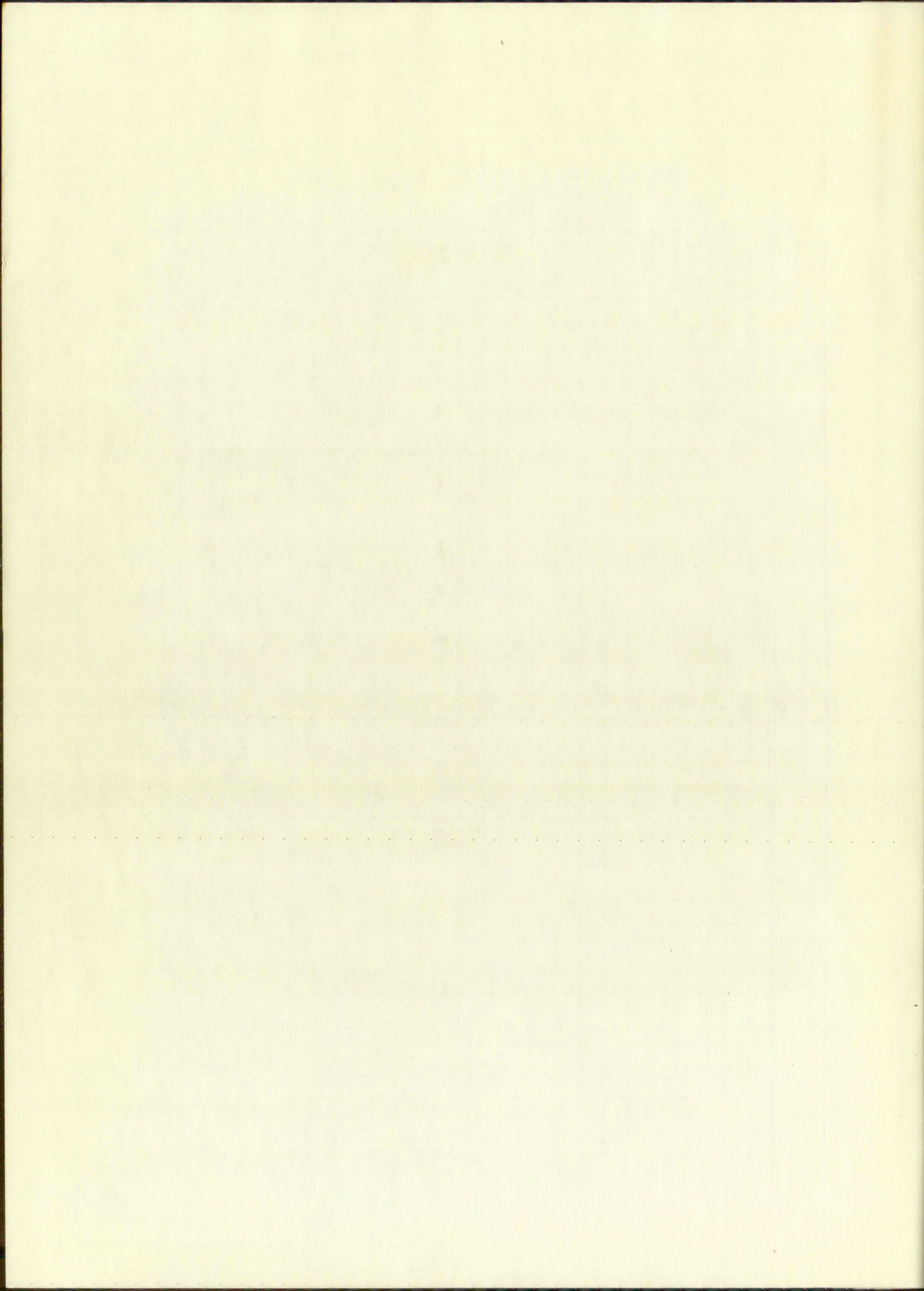
20

10

0

PERCENT FINER BY WEIGHT





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