

AD-A032 943

ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISS F/G 8/13  
FIELD TESTS OF THE CUCARACHA FORMATION, PANAMA CANAL 1942-1946.(U)  
MAY 74 C K SMITH, R J LUTTON

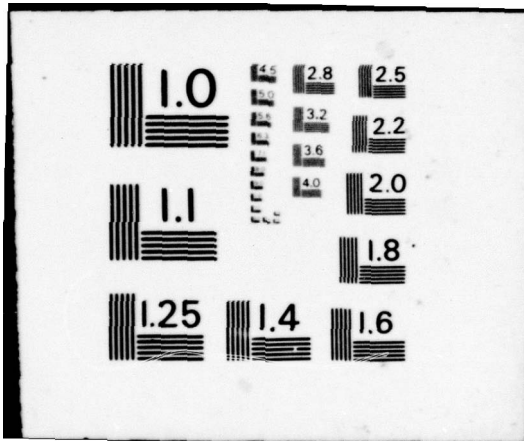
UNCLASSIFIED

WES-MP-S-74-16

NL

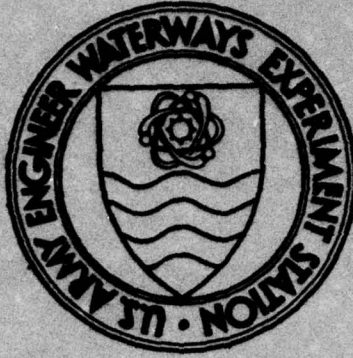
1 of 2  
ADA032943





91750 0

ADA 032943



2

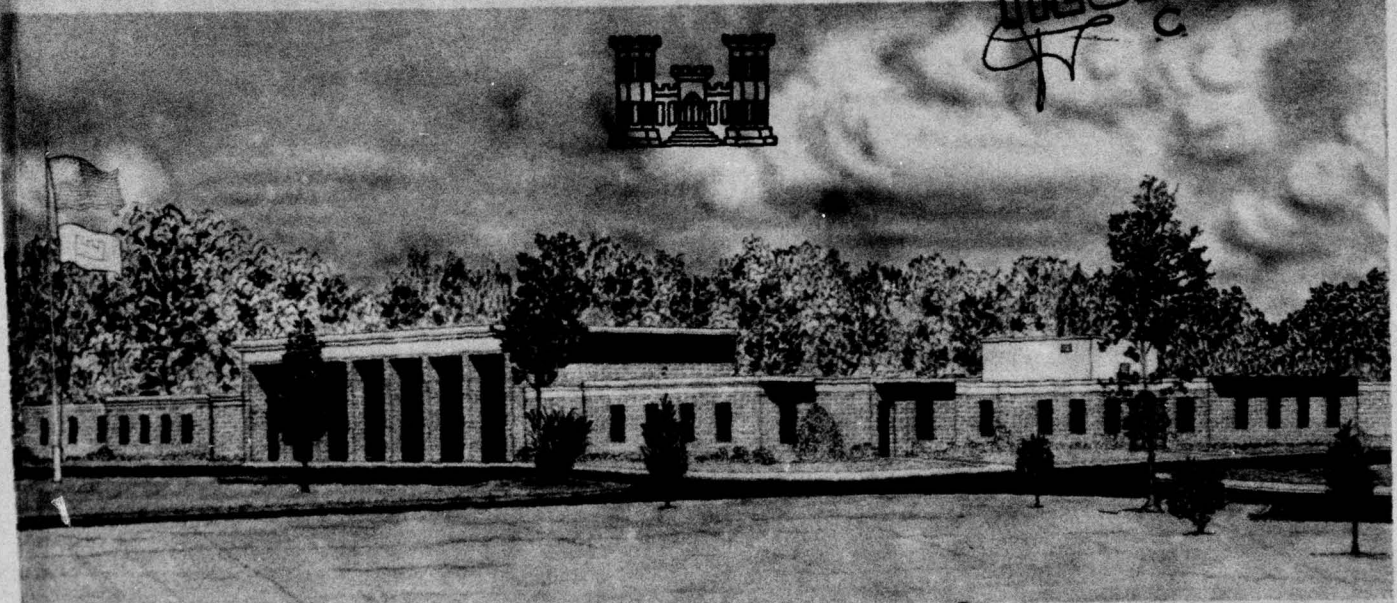
MISCELLANEOUS PAPER S-74-16 ✓

# FIELD TESTS OF THE CUCARACHA FORMATION, PANAMA CANAL, 1942-1946

by

C. K. Smith, R. J. Lutton

DDC  
RECEIVED  
DEC 3 1976  
REGULATED  
FC



May 1974

Copy available to DDC does ~~not~~ permit fully legible reproduction

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station  
Soils and Pavements Laboratory  
Vicksburg, Mississippi

APPROVED FOR PUBLIC RELEASE; DISTRIBUTION UNLIMITED

Destroy this report when no longer needed. Do not return  
it to the originator.

The findings in this report are not to be construed as an official  
Department of the Army position unless so designated  
by other authorized documents.

14 WES-MP-S-74-16



6

MISCELLANEOUS PAPER S-74-16

**FIELD TESTS OF THE CUCARACHA FORMATION, PANAMA CANAL, 1942-1946.**

by

C. K. Smith, R. J. Lutton

9 Final Rept.

DDC  
DEC 3 1976  
RECEIVED

10 Carneal & K. Smith  
Richard J. Lutton



12 124 p.

11  
May 74

ACCESSION for	
NTIS	White Section <input checked="" type="checkbox"/>
DDC	Buff Section <input type="checkbox"/>
UNIVERSITY MICROFILMS	<input type="checkbox"/>
JSTOR	
BY	
DISTRIBUTION/AVAILABILITY CODES	
Dist.	APPL. SEC. or SPECIAL
R	

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station  
Soils and Pavements Laboratory  
Vicksburg, Mississippi

ARMY-MRC VICKSBURG, MISS.

APPROVED FOR PUBLIC RELEASE; DISTRIBUTION UNLIMITED

038100

AB

THE CONTENTS OF THIS REPORT ARE NOT TO  
BE USED FOR ADVERTISING, PUBLICATION,  
OR PROMOTIONAL PURPOSES. CITATION OF  
TRADE NAMES DOES NOT CONSTITUTE AN OF-  
FICIAL ENDORSEMENT OR APPROVAL OF THE  
USE OF SUCH COMMERCIAL PRODUCTS.

## FOREWORD

Preparation and publication of this report were funded by redistribution of funds from Engineering Studies projects assigned to the U. S. Army Engineer Waterways Experiment Station (WES) in accordance with authorization by letter from the Office, Chief of Engineers (OCE) (DAEN-CWE-S), dated 29 November 1971, Subject: "ES 546, Panama Canal Slope Studies."

Special acknowledgement is due to various individuals in the Special Engineering Division, The Panama Canal Company, who were responsible for the planning, executing, and reporting of the field studies described herein. Persons involved in the studies conducted in connection with the Third Locks Project in the early 1940's were:

- a. Pedro Miguel Test Pit No. 3. Mr. E. Montford Fucik, Senior Engineer in charge of the Soil Mechanics Section, was in general charge of the investigation. Mr. A. C. Flach, Engineer, supervised the field and laboratory testing. Mr. C. F. Vincent wrote the report.
- b. Cucaracha Foundation Test. Mr. E. E. Abbott, Chief, Special Engineering Division, was the supervising engineer until sometime in 1945, when he was succeeded by Mr. Russell L. Klotz. Mr. T. F. Thompson, Chief, Geology Section, selected the site, analyzed the subsurface conditions, correlated the site with the prototype location, and took record photographs during the test program. Most field observations were made by Messrs. Maurice Eggleston and Maurice Teewinkle, Engineers. Mr. Robert L. Tracy, Engineer, assembled and analyzed the data and prepared the report.

With the exception of Part II, preparation of the text and selection of accompanying photos and figures were accomplished by Carneal K. Smith under Contract DACW39-72-C-0041 with WES. Part II was prepared

by Dr. R. J. Lutton, Engineering Geology Division, WES. Coordination of contract and WES work, review of the draft report, and incorporation of the authors' WES and OCE comments in the final report were accomplished by Mr. J. R. Compton, former Chief, Soil Mechanics Division, under the general supervision of Mr. J. P. Sale, Chief, Soils and Pavements Laboratory.

During the course of the WES work, Directors of WES were BG E. D. Peixotto, CE, and COL G. H. Hilt, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

	<u>Page</u>
FOREWORD . . . . .	v
CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT . . . . .	ix
SUMMARY . . . . .	xi
PART I: INTRODUCTION . . . . .	1
Background . . . . .	1
Tests Prior to 1942 . . . . .	2
Purpose and Scope of Tests from 1942 to 1946 . . . . .	2
PART II: GEOLOGY . . . . .	5
Early Interpretation of Geology . . . . .	5
Possible Misidentification of Strata . . . . .	10
Consequence of Possible Misidentification . . . . .	12
Details of Test Pit No. 3 . . . . .	13
Details of the Cucaracha Foundation Test Base . . . . .	15
PART III: PRE-1942 TESTS AND EARLY DESIGN VALUES FOR BEARING CAPACITY, MODULUS OF ELASTICITY, AND SHEAR STRENGTH . . . . .	22
PART IV: PEDRO MIGUEL TEST PIT NO. 3 . . . . .	23
Field Shear Tests . . . . .	23
Laboratory Shear Tests . . . . .	31
Bearing Tests . . . . .	34
Field Bearing Tests . . . . .	35
Laboratory Bearing Tests . . . . .	51
Conclusions and Recommendations of November, 1942 . . . . .	51
PART V: LARGE-SCALE CUCARACHA FOUNDATION TEST . . . . .	53
General . . . . .	53
Methods of Obtaining Test Data . . . . .	55
Settlement Data . . . . .	56
Uplift Tests . . . . .	65
Base Pressures . . . . .	67
Subsurface Settlement . . . . .	71
Deflection of Structure Base . . . . .	71
Settlement of Circumjacent Shale Surface . . . . .	71
PART VI: RECOMMENDATIONS OF CONSULTANTS . . . . .	76

CONTENTS

	<u>Page</u>
PART VII: CONCLUSIONS AND RECOMMENDATIONS . . . . .	77
Conclusions . . . . .	77
Recommendations . . . . .	79
LITERATURE CITED . . . . .	80
TABLES 1-7	
PHOTOS 1-7	
APPENDIX A: ORIGINAL GEOLOGICAL LOGS OF BORINGS IN THE VICINITY OF THE TEST STRUCTURE AND TEST PIT NO. 3	

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
inches	2.54	centimeters
feet	0.3048	meters
miles (U. S. statute)	1.609344	kilometers
square feet	0.092903	square meters
gallons (U. S. liquid) per minute	3.785412	cubic decimeters per minute
tons (2000 lb)	9.0718474	metric tons
kips (force)	4.448222	kilonewtons
tons (force)	8.896444	kilonewtons
kips (force) per square foot	47.880283	kilonewtons per square meter
tons (force) per minute	8.896444	kilonewtons per minute
tons (force) per square foot	95.760567	kilonewtons per square meter
pounds (force) per square inch	0.6894757	newtons per square centimeter

## SUMMARY

↓  
This report describes and presents the results of field studies of shales of the Cucaracha formation conducted during 1942-1946 by the Special Engineering Division of The Panama Canal Company in connection with the design of the Third Locks Project.

Geology of the area in the vicinity of the then-proposed New Pedro Miguel Lock where the field studies were made, as presented in this report, was based on a review of geological studies available at the time of the tests supplemented by more recent studies and exploration of the Cucaracha formation by geologists of The Panama Canal Company and the U. S. Army Engineer Waterways Experiment Station.)

The two principal field studies discussed in the report are:

- a. Pedro Miguel Test Pit No. 3. The pit was excavated in 1942. Field shear and bearing tests were performed in a drift on Cucaracha clay shale, and laboratory shear and bearing tests were made for comparison.
- b. Cucaracha Foundation Test. This test, which was initiated in early 1944, consisted of a large-scale bearing test of a concrete base 40 by 50 ft in plan and 10 ft thick. The base, which simulated that of a lock wall monolith, was loaded in stages over a period of about 9 months with steel plate and concrete to produce maximum pressures of 12.5 tsf at the toe and 7.5 tsf at the heel. Maximum pressures were maintained for about 5 months and then reduced to an average pressure of 6 tsf, which was maintained for 300 days before concluding the program in April 1946.

Measurements were made of structure settlements, settlements adjacent to the structure up to 40 ft away, and pressures immediately beneath the base of the structure.) Attempts to measure pressures induced at various depths beneath the structure were not successful.

→ Recommendations made by a board of consultants in February 1945 following review of the test results are included in this report.

The data indicate that settlement of large structures on Cucaracha clay shales could be higher than anticipated in 1945. It is recommended that analyses be made of the test data using present-day analytical methods.

FIELD TESTS OF THE CUCARACHA FORMATION

PANAMA CANAL, 1942-1946

PART I: INTRODUCTION

Background

1. In 1936, Congress authorized studies to be made to investigate expanding and improving the Panama Canal. As a result of these studies, Governor C. S. Ridley in February 1939 recommended that construction of locks be started within 10 or 12 years. The project was to incorporate locks of greater dimensions than had formerly existed. The new locks were to be located at some distance from the old structures with channels connecting the new sites with existing waterways. In August 1939, Congress authorized the construction of a third set of locks, each 140 ft wide\* and 1200 ft long. This project became known as the Third Locks Project. Construction was suspended in May 1942 when it became apparent that the new locks could not be completed before the end of World War II due to conflicting demands for men and materials. At that time, excavation for the Gatun and Miraflores Third Locks had been substantially completed, but excavation for the third lock at Pedro Miguel and work on actual lock construction had not commenced.

2. In the years since the suspension of the Third Locks Project, the construction of a third set of locks has been abandoned in favor of plans for a sea level canal due to the vulnerability of locks to enemy attack and sabotage. Many possible routes for a sea level canal have been studied in Nicaragua, Panama, and Colombia. However, a route through the Canal Zone and vicinity appears at present to have advantage over the other routes. If a sea level canal were excavated in the Canal Zone and vicinity or if a new terminal lake lock canal were built,

---

\* A table of factors for converting British units of measurement to metric units is presented on page ix.

the data on the Cucaracha formation from the Third Locks Project would become extremely valuable. Although there has been some testing of the Cucaracha in recent years, the bulk of the data on the formation is from the years 1942-1946. Even so, the Cucaracha foundation test is of interest to the engineering profession. It is probably the largest highly instrumented bearing test that has ever been performed. A comprehensive report on this test has never been made available to the engineering profession. There were also many small diameter bearing tests performed in Pedro Miguel Test Pit No. 3 on the Cucaracha formation, and laboratory tests were performed on samples from Test Pit No. 3. Since this pit is located close to the Cucaracha Foundation Test site the data have been included in this report along with the results of the large-scale bearing test.

#### Tests Prior to 1942

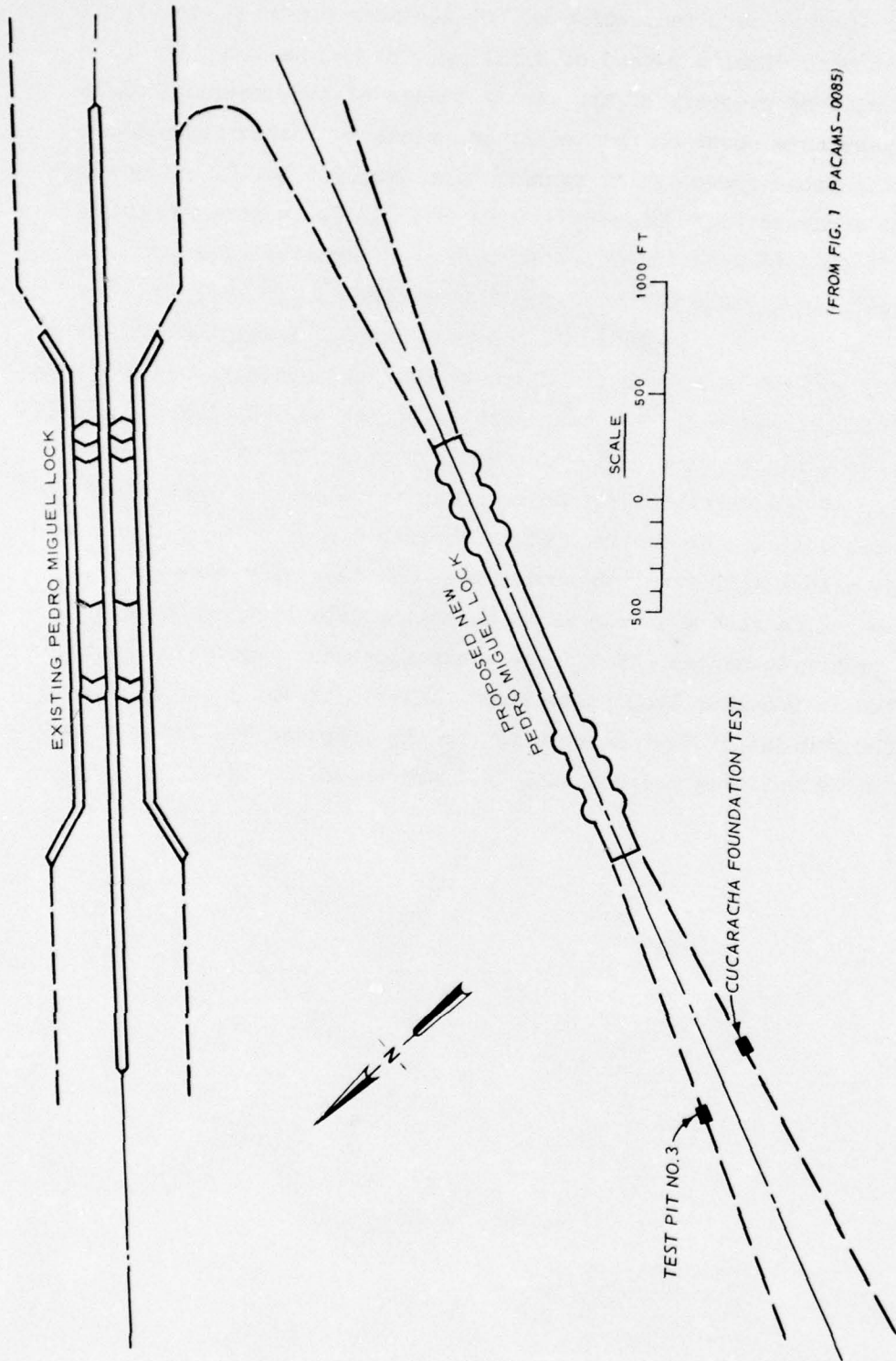
3. The first field tests on the Cucaracha formation for the Third Locks Project were conducted in Pedro Miguel Test Pit No. 2. Test Pit No. 2 was located in the lower approach area of the proposed New Pedro Miguel Lock. The Cucaracha formation in this location is a dark gray, moderately slickensided sandy shale. Since a large part of the Cucaracha formation consists of slickensided clay shale, it was decided to excavate another test pit so that tests might be made on the more slickensided phases of the Cucaracha clay shale.

#### Purpose and Scope of Tests from 1942 to 1946

4. In December 1941, the Soils Mechanics Section of the Special Engineering Division, Panama Canal Company, reviewed the recommended bearing and shear strength values for the materials in the Pedro Miguel area. The recommended values were in part based on the results from the tests in Pedro Miguel Test Pit No. 2 and from laboratory tests on cores from drilling with water. Drilling with water resulted in damage to the cores and poor recovery. Consequently, laboratory tests

on small samples were indicative of the stronger phases of the formation. At this time, a method of drilling with mud had been tried which permitted recovery of the weaker phases of the Cucaracha shale. A few laboratory tests on the weaker materials indicated strengths lower than those shown in the results from Test Pit No. 2. It was decided to excavate Pedro Miguel Test Pit No. 3. Tests were performed in the drift to determine the shear strength and the bearing capacity of the slickensided phase of Cucaracha clay shale.

5. Field tests at Pedro Miguel Test Pit No. 3 resulted in lower values of shearing strength and ultimate bearing capacity than had been previously recommended. All test results seemed to indicate the desirability of a large-scale test which would average the variable characteristics of the Cucaracha formation and give values applicable for structural design. It was decided to perform a bearing test that would simulate a lock wall monolith prototype. The test structure would be instrumented in such a manner as to determine data that could be used in the prototype design. The large-scale Cucaracha Foundation Test was initiated in December 1943. Locations of Test Pit No. 3 and the Cucaracha Foundation Test in relation to the proposed New Pedro Miguel Lock and the existing Pedro Miguel Lock are shown in fig. 1



(FROM FIG. 1 PACAMS-0085)

Fig. 1. Location of Test Pit No. 3 and the Cucaracha Foundation Test with respect to the existing and proposed locks

## PART II: GEOLOGY

6. Test Pit No. 3 and the Cucaracha Foundation Test site were located in the old valley of the Rio Grande about 4000 ft due west of the existing Pedro Miguel Lock. Near this point, the Rio Sierpe and Quebrada Conga enter the main valley from the southwest. Most of the Rio Grande runoff had been diverted directly into the Canal upstream so that the valley here carried only a fraction of that at the natural condition. Considerable dredge spoil had been dumped in the valley, modifying to some degree the configuration of the valley bottom.

7. Early geological studies were confined to areas within a few hundred feet of the banks of the Canal, and no detailed geological information was available in the vicinity of the test sites prior to 1940. Essentially all of the information on local geology was collected during the Third Locks Project (fig. 2) from 1940-1945; numerous borings\* were made along the lines of the right and left walls of the proposed New Pedro Miguel Lock, extending southeastward from this area.

8. Test Pit No. 3 (excavated in 1942) and the foundation test in 1944 were located on the lines of the right and left lock walls, respectively. Five borings were made near corners of the foundation test base to establish geological details immediately below. The locations of exploratory borings for the lock walls and test excavations and of the additional borings at the foundation test base are shown in fig. 3. Revised geological cross sections through the two test sites are given in fig. 4.

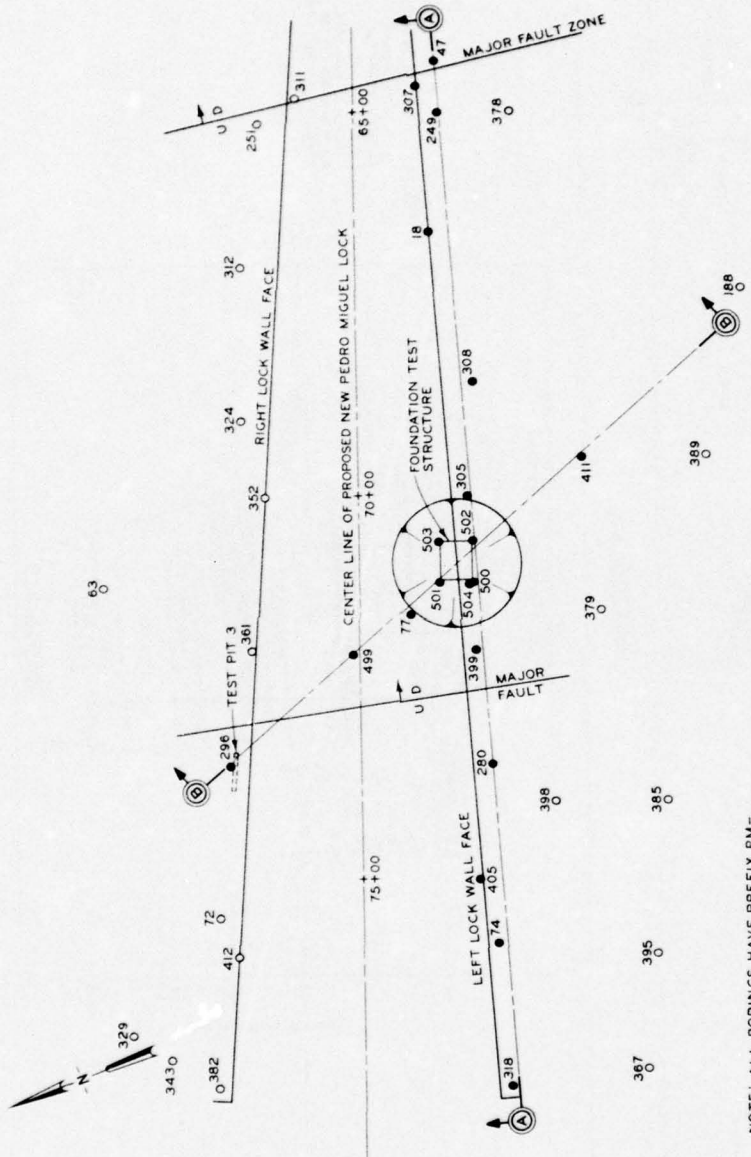
### Early Interpretation of Geology

9. Fig. 2 shows the early interpretation of geology along the design lock walls as deduced from core borings through 1942. The following general description of the formations is taken from Flach.<sup>1</sup> Rocks occurring beneath the north end of the proposed New Pedro Miguel

---

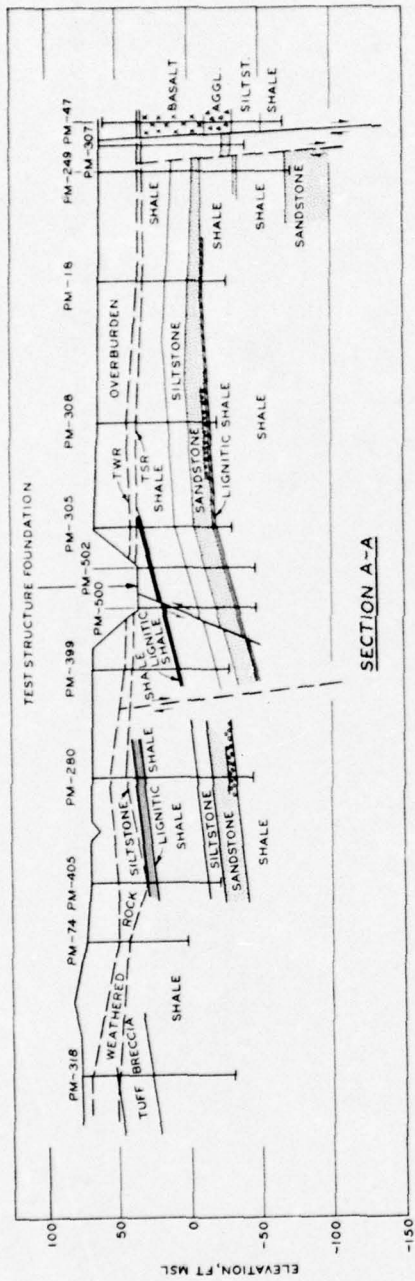
\* Selected logs of borings are given in Appendix A.



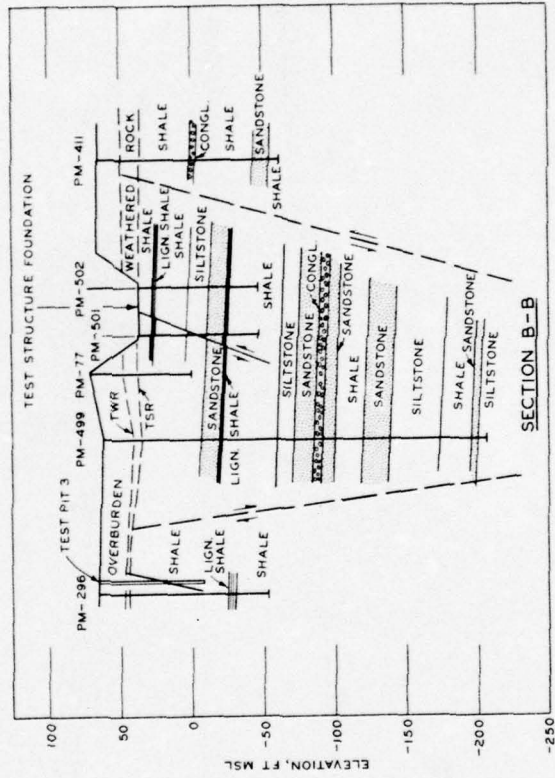


NOTE: ALL BORINGS HAVE PREFIX PM-  
BORING LOCATIONS INDICATED BY O OR ●  
SOLID CIRCLES INDICATE BORINGS USED IN  
LOGS OF BORINGS PM-77-296, -305, -399,  
-499 AND -500 THROUGH -504 ARE GIVEN  
IN APPENDIX A

Fig. 3. Locations of test sites, major faults, borings through 1946, and geologic sections



SECTION A-A



SECTION B-B

NOTE: SEE FIG. 3 FOR LOCATIONS.  
 VERTICAL: HORIZONTAL SCALE  
 EXAGGERATION IS 2:1 AS IN  
 FIG. 2.  
 SECTION A-A IS PREFERRED  
 OVER EARLIER INTERPRETATIONS  
 ALONG SAME LINE IN FIG. 2

Fig. 4. Geological sections of test sites.

Lock may be divided into two major groups: (a) igneous-related rocks, consisting of hard, dense basalt and agglomerate, and (b) sedimentary rocks, including rather hard, cemented sandstone and conglomerate, and comparatively weak shale, regarded at that time as belonging to the Culebra and Cucaracha formations.

10. Strengths differ greatly, ranging from very strong, massive igneous rock to the unpredictable, weak, slickensided shale beds. Faults, folds, and joint systems complicate the structural features and further weaken the rocks. A reasonable degree of correlation of the strata has been attained through extensive core boring combined with surface study, but excavation would no doubt reveal details otherwise missed, particularly in the softer phases of the sedimentary beds.

11. Beds regarded as Culebra formation are composed essentially of shale, sandstone and conglomerate, with scattered calcareous veins and concretions in the sandstone and conglomerate. Fossils indicate a marine environment of deposition. The shale is essentially well-consolidated tuffaceous siltstone, extensively laminated with sandy partings, and it slakes readily on exposure. The sandstone and conglomerate beds are lenticular, generally hard and well cemented. The supposed Culebra formation lies well below foundation grade at the north end of the proposed lock.

12. Beds regarded as Cucaracha formation are the weakest of all foundation materials in the vicinity, and yet they would form most of the foundation of the north end of the proposed lock. Disturbances have especially weakened the tuffaceous shale phase, which comprises about 50 to 75 percent of the entire formation. Sandstone, conglomerate, and an ash flow constitute the remainder. These harder rocks show little effect of the general disturbances. Hard beds are lenticular and were probably cushioned by the very extensive and more easily distorted shales which surround them. The ash flow, a welded tuff breccia, was usually regarded as unique and used widely for stratigraphic correlation. It is a strong rock but is characterized by well-developed jointing. From an engineering standpoint the ash flow

was insignificant since no part of the proposed New Pedro Miguel Lock came in direct contact with it.

13. The typical shale is medium hard, rather brittle, soapy (highly bentonitic) clay rock originally consolidated under a high overburden pressure. Subsequent alterations of the component materials have contributed to the creation of an intricate system of slickensides which lessens the overall strength. Weaker zones of soft, crushed shale (gouge) are present locally in certain areas and are believed to be directly related to faults although some gouge-like material has been revealed by borings in other areas supposedly not near such zones.

14. Since the major proportion of the supposed Cucaracha formation is shale, little importance was assigned to the presence of harder and stronger sandstone and conglomerate beds in strengthening the formation as a foundation for the new lock. This applied especially to the north approach wall areas (fig. 2), which would supposedly have been founded on the middle portion of the Cucaracha formation, with abundant weak, slickensided shale and only minor sandstone and conglomerate. The south approach area, at the other end of the proposed lock, was supposedly underlain by the lower portion of the Cucaracha, which is characterized by its stronger, more continuous beds of sandstone and the presence of basal conglomerate immediately above the Culebra formation.

#### Possible Misidentification of Strata

15. In view of the renown of the Cucaracha shale and in turn the significance of the in situ tests described herein, it is appropriate to critically review the basis for previous identification of test site material as the Cucaracha. It was pointed out above that very little information on geology in the immediate vicinity had been assembled prior to the boring program. The boring logs were and still are the main source of information on the geology. At the time of the boring program, from 1940 to 1945, D. F. MacDonald as consultant was strongly influencing geological investigations in the Canal Zone.

MacDonald had worked there in about 1912-1914 during construction for the Isthmian Canal Commission and at that time had established stratigraphic nomenclature that remains to the present.

16. Geologists of the Special Engineering Division of The Panama Canal Company apparently found the old stratigraphic sequence to be adequate for logging the units they were encountering in the new borings. In boring logs and reports of the Third Locks Project, somewhat subjective identifications of formations by names have been made. There are enough discrepancies or anomalies to leave a degree of uncertainty.

17. The crux of the problem centers on the individuality of the Cucaracha, Culebra, and La Boca formations. These three sedimentary formations are recognized today in the Canal banks and each has in the past been confused with the others. Salient points that need to be resolved or at least thoroughly considered are as follows:

- a. The Culebra, Cucaracha, and La Boca formations lie stratigraphically in the lower Miocene<sup>2</sup> and represent a relatively small portion of geological history.
- b. Although the three formations are dominated by sandstone, shale, and siltstone, respectively, all three rock types are present in each of the formations.
- c. The distinction between clayey siltstone and silty shale may vary from observer to observer, and there were at least three geologists logging borings in this area.
- d. The fossil content of some of the boring cores was used to support stratigraphic identification, but it seems unlikely that an adequate analysis was made to find subtle faunal differences that would be required to distinguish such similar and nearly contemporaneous formations.
- e. The La Boca formation was equated to the upper Culebra formation in 1942.
- f. Siltstone and shale encountered in borings 2200 ft northwest at Cartegena South Extension slide of 1964 were classified as La Boca formation by R. H. Stewart of The Panama Canal Company. Other beds once classified as Culebra or Cucaracha near the Pedro Miguel Locks are now regarded as La Boca formation.
- g. Several of the boring logs indicate atypical strata. Boring PM-499 encountered fragments of oyster shells high in the unit that was called Cucaracha shale, and

a lower fossiliferous zone also considered within the Cucaracha was designated as the "False Culebra." Similarly, oyster shell fragments were found at a relatively high level in borings PM-500 to -504 below the foundation test base.

18. On the other hand, two features of the strata near the test sites seem to support the identification as Cucaracha formation. Some of the strata appear to be composed of alternating sandstone, siltstone, and shale in a graded bed relationship, i.e., coarse sandstone grading upward to siltstone and then shale, followed by a sharp change to sandstone again. The second supporting feature is the presence of so-called ash flow, as in boring PM-318 (fig. 4). If this tuff breccia can be shown to be the unique ash flow found in the Cucaracha formation 1 mile to the north along the Canal, the identification as Cucaracha formation will be confirmed. The Cucaracha ash flow is said by Thompson<sup>3</sup> to outcrop in the valley of Rio Majellon several hundred feet west of the foundation test site, but until the unit has been followed more or less persistently to a connection with the known Cucaracha ash flow along the Canal, there is still room for doubt.

#### Consequence of Possible Misidentification

19. Descriptions of the shale at the test sites clearly establish its similarity to shale of the Cucaracha formation. It was noted above that the age in any case is probably very close, so geological preconsolidation must be essentially the same. The environment of deposition is about the same for the Culebra, lower Cucaracha, and La Boca formations, with all being volcanic-derived marine sedimentary rocks except for a few intervals marking terrestrial conditions. It seems best for the present to regard the in situ test results as representative of tuffaceous shale like that in the Cucaracha formation. The chances are rather good that the test sites were indeed located in the lower half of the Cucaracha, but this has not been proven to date.

Details of Test Pit No. 3

20. According to Vincent<sup>4</sup>, Test Pit No. 3 was located at station 73+35, 161 ft right of the axis of the proposed New Pedro Miguel Lock (fig. 2), near boring PM-296. This location was chosen after a careful study had been made of the cores taken from borings in the north approach wall areas. Boring PM-296 indicated a considerable depth of material which was representative of the weaker phase of the supposed Cucaracha formation and therefore very desirable for testing. The materials encountered in the vertical shaft (fig. 5) were as follows:

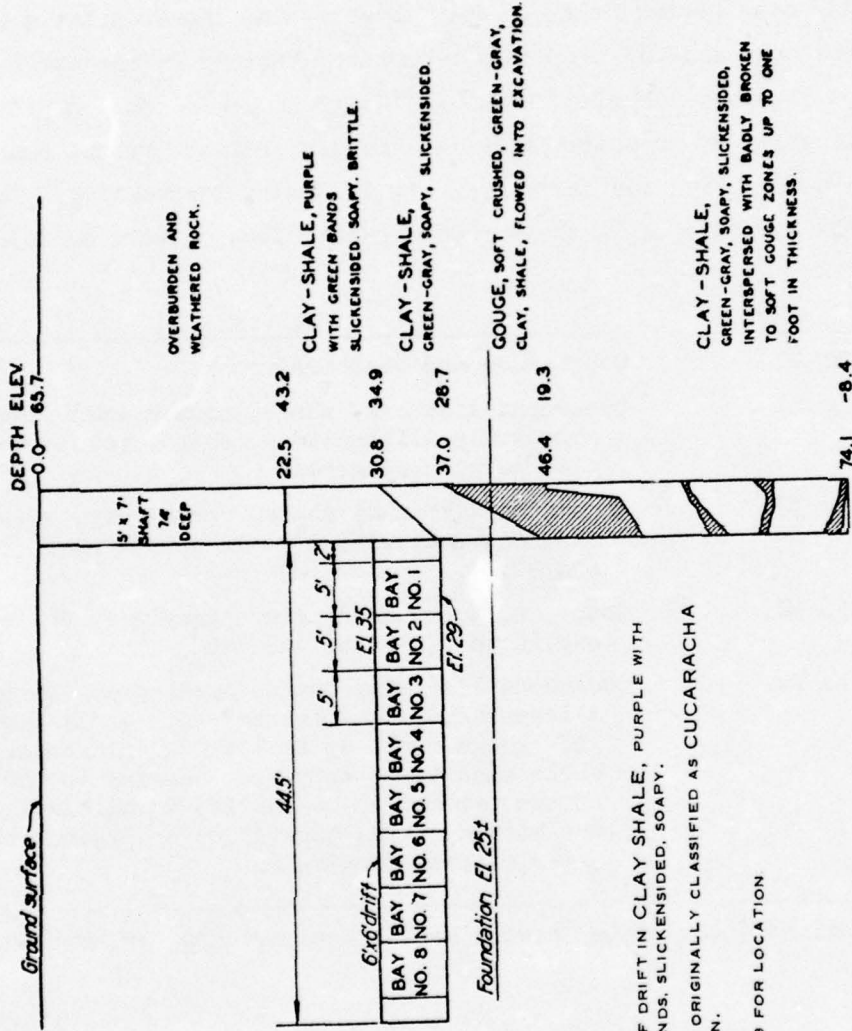
<u>Depth, ft (Ground Elev Is +65.7*)</u>	<u>Materials</u>
0 to 22	Overburden and weathered rock
22 to 31	Cucaracha-like clay shale, medium hard, dense but abundantly slickensided, soapy, purple to mottled purple and green
31 to 37	Cucaracha-like clay shale, green-gray, soapy, slickensided, with a gouge zone between 30.5 and 32.0
37 to 52	Gouge, soft, crushed, green-gray clay shale, soft enough to flow into the pit
52 to 74	Cucaracha-like clay shale, green-gray, soapy, and slickensided, interspersed with badly broken, soft gouge zones up to 1 ft in thickness. The whole mass is so soft that lagging in the sides of the pit had to be carried to within 1 ft of the bottom of the excavation to prevent the material from flowing in

---

\* Elevations (elev) cited herein are in feet referred to mean sea level.

21. A drift (fig. 5) was excavated between depths of 30 and 36 ft in the north wall of the shaft. The drift was 6 by 6 ft and extended 45 ft back into the shale. This location was selected to allow testing of the very slickensided purple shale at about the foundation level of the proposed lock. As can be seen from the section in fig. 5, the material tested appears in the shaft between depths of 22 and 31 ft and

PM - TP-3  
 STA. 73+35 (P-8-C AXIS)  
 161' R.



NOTE:  
 BOTTOM OF DRIFT IN CLAY SHALE, PURPLE WITH GREEN BANDS. SLICKENSIDED. SOAPY.  
 ALL ROCK ORIGINALLY CLASSIFIED AS CUCARACHA FORMATION.

SEE FIG. 3 FOR LOCATION

Fig. 5. Sectional view of Test Pit No. 3

dips downward quite steeply in the direction of the drift, providing a depth of material below the floor of the drift sufficient for the field tests to be considered to have been performed on material of infinite thickness.

22. As the drift was dug, it was timbered with frames of 8- by 8-in. beams set approximately 5 ft apart. It was necessary to carry lagging overhead along with the advance to prevent chunks of the rock from falling in. Pieces of this material frequently broke off to form domes in the ceiling, and prior to testing grout was forced into the spaces between the timber roof and the rock so that the ceiling would provide a good resistance for the jack loads applied in the field tests.

23. During excavation of the pit, moderate water inflows were encountered. Most water issued from gougy fractures, but an appreciable contribution came from seemingly tight slickensided surfaces.<sup>5</sup> The maximum pumpage from the pit was in excess of 50 gpm with inflows spaced rather uniformly downward to the bottom.

24. An attempt was made to dig another drift between depths of 64 and 70 ft in the green-gray shale. This drift had to be abandoned after 2-1/2 ft had been dug because the amount of cave-in endangered the entire shaft.

#### Details of the Cucaracha Foundation Test Base

25. Subsequent to the Cucaracha Foundation Test, core borings were drilled 3 ft from each corner of the concrete test base on the extension of a line bisecting the corner angle. An additional boring was drilled after completion of the first four to determine to what extent the water had been released by drilling. Locations are shown in fig. 6. Drilling was from a platform about 4 ft above the floor of the excavation. Stratigraphic intervals are given in the logs (Appendix A) in feet below the platform reference point (depths) and in feet referred to mean sea level (elevations).

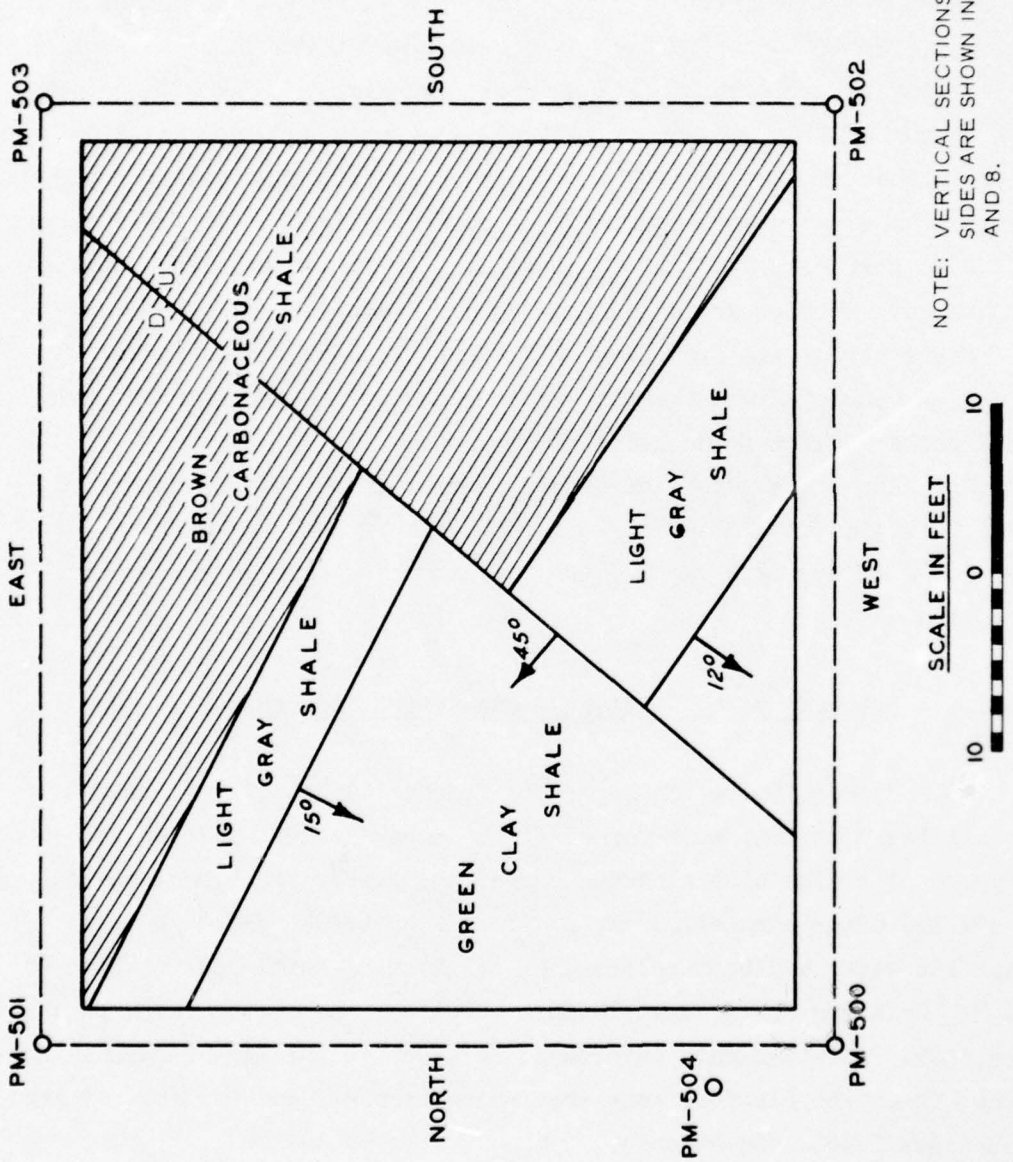


Fig. 6. Geologic plan of test structure base

#### Drilling of posttest borings

26. Drilling procedure was like that used during the Third Locks Project exploration where Cucaracha or similar strata were known to be present. Carboloy-set bits were employed exclusively with a 10-ft core barrel recovering NX (2-1/8-in.-diam) core samples. The use of a commercial bentonite drilling mud assisted in recovering a high percentage of the softer materials. Core recovery from the 5 borings averaged 92 percent, with a range of 88 to 97 percent. Cores were covered with wetted burlap to minimize drying. Samples were selected for laboratory testing and future inspection upon removal from the core barrel and were coated with wax to preserve their natural water content. Each day's recovery was removed to a high humidity room for permanent storage. Where pieces of sufficient length for testing could be obtained, samples were taken from consecutive 10-ft depths at each boring. If marked lithologic differences occurred within any 10-ft section, typical samples of each material were selected. The waxed samples were also stored in the high humidity room. The drill inspector selected and coated the samples, noted unusual drilling conditions, and recorded pertinent information that would not be apparent from an examination of the cores alone.

27. On the completion of each boring, it was filled with a coarse sand (passing the No. 16 screen and retained on the No. 30) to within 4 ft of the top. Sand was introduced to the bottom of the hole by means of a 2-in. pipe that was progressively raised as the drill mud was forced out. A grout cap was poured into the top 4 ft to prevent rising water from washing the sand from the hole.

28. During the drilling of three of the borings (PM-501, -502, and -504) the drilling mud was lost and reappeared at the surface at several points, in the case of PM-504, 40 ft from the casing head. Lost mud usually returned to the surface by way of visible cracks in the rock or at the top of settlement point borings. In PM-501, artesian water was encountered (about 2/3 gpm) at elev -10.0, and a constant flow at this rate continued to the ultimate depth of the boring at elev -47.0. Some difficulty was experienced in sealing off this flow

after the boring was filled with sand.

Geology of foundation test base

29. Strata encountered below the test base could be correlated from boring to boring. A minor fault that extended from the southeast corner to about 15 ft south of the northwest corner (fig. 6) had maximum offset of about 3 ft and a zone of crushing less than 1 ft in width. Bedding dipped northwest at an inclination of about 15 deg. The strike, as measured at the subgrade prior to placing the concrete base, was N36°W. Figs. 7 and 8 show the apparent dip of the beds along vertical sections 2 ft outside of the base of the structure, as well as the location and effect of the small fault. Descriptions of the beds are also shown in these figures.

30. A carbonaceous shale exposed on the test base (fig. 6) was the weakest member, and its presence was believed by Thompson<sup>3</sup> to be the prime factor in the settlement observed (see Part V). Beds of this character occur as lenses scattered throughout the Cucaracha formation along the Panama Canal, but aggregate only about 6 percent of its total thickness and are generally less than 6 ft thick. Stresses created by regional folding have been relieved by slippage along bedding laminae with resultant mashing. This slippage is locally reflected by small seams or lenses of soft, brown carbonaceous clay found at various locations within the beds but without large lateral extent. These interbedded gouge seams are soft and plastic, and because of their considerable organic content absorb much water after release of confining pressure.

31. A light gray shale immediately overlying the carbonaceous shale was believed to have been originally of the same character as the green shale which overlies it; the difference in color was attributed to the bleaching effect of the carbonaceous bed. The somewhat softer, disturbed nature was thought to be due to the effects of slippages along the contact with the underlying carbonaceous member.

32. Green shale, three beds of which were encountered (figs. 7 and 8), was similar to typical shale of the Cucaracha. Strength of the green shale varies depending on slickenside spacing and degree of

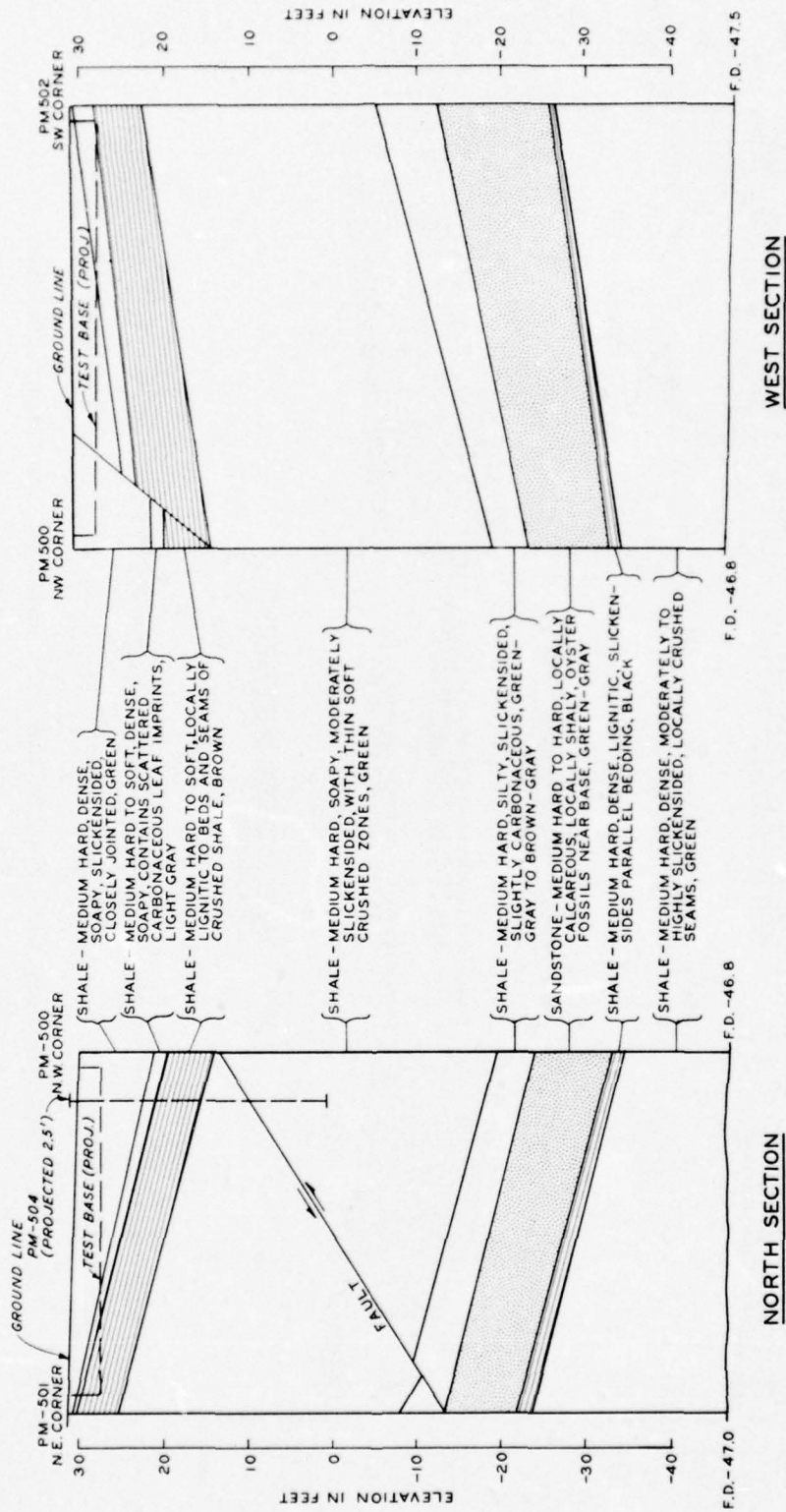


Fig. 7. Geologic sections of north and west sides of test structure base

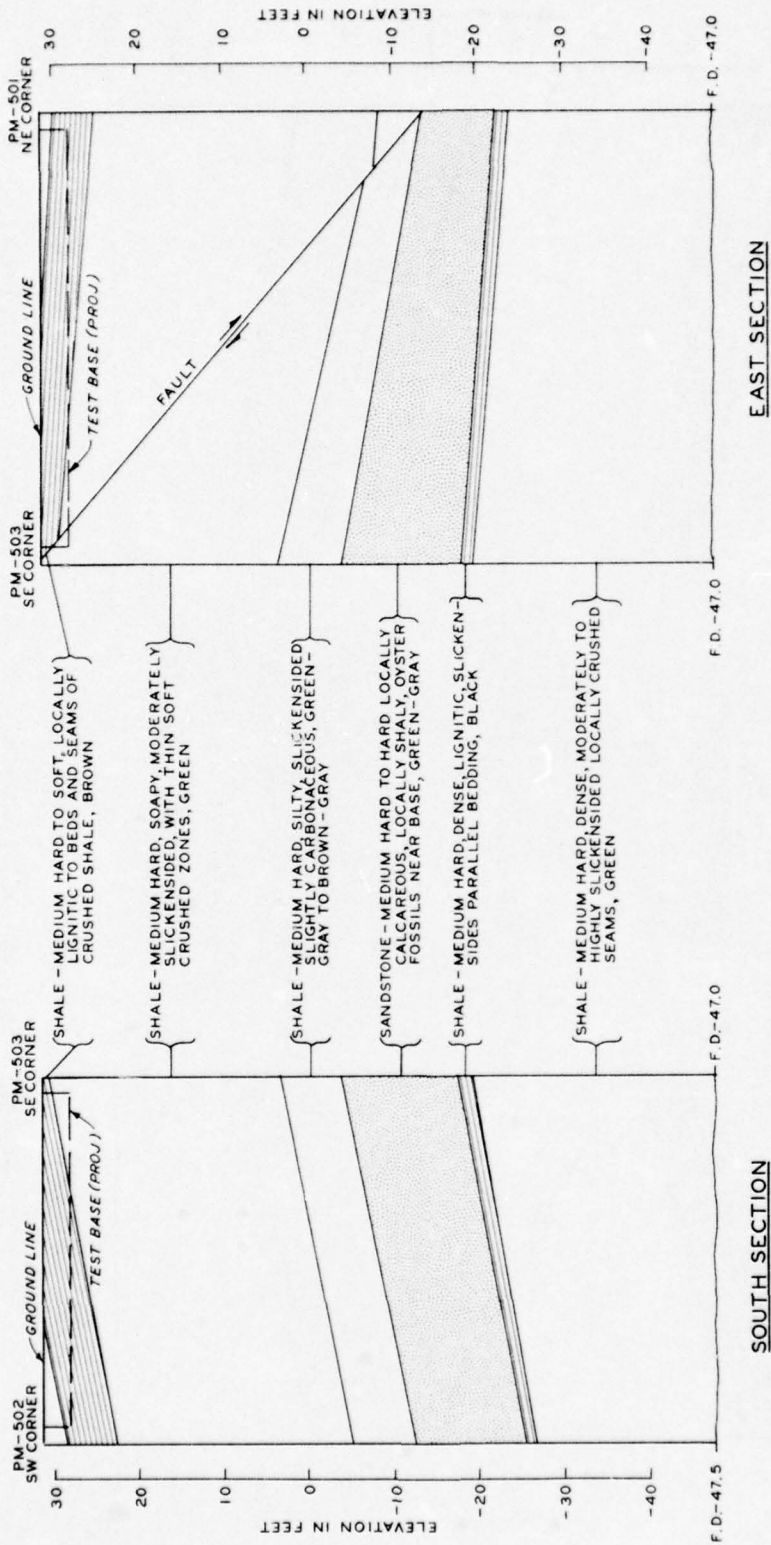


Fig. 8. Geologic sections of south and east sides of test structure base

fracturing subsequently by "distributed" fault action. Thin, soft, light gray gouge seams commonly occurring within the beds resulted from compression and differential movement along minor faults. Generally, core samples of appreciable length parted on slickensides, and samples tested in the laboratory often failed partially or completely along slickensides. From this evidence, it was deduced that the most highly slickensided strata, if other considerations are the same, are the weakest of the unfaulted, green or chocolate-colored shales; and conversely, those which have wider spacing of slickensides are stronger. This deduction has been borne out by the many tests.

33. The green-gray sandstone bed shown on figs. 7 and 8 has a maximum thickness of about 12-1/2 ft. The rock is variably calcareous and argillaceous and accordingly has a considerable range in strength. Its subsurface trend along the right wall of the proposed New Pedro Miguel Lock (fig. 6) can be traced 650 ft southeast, and it thins considerably in that direction. This bed is the strongest revealed in foundation test borings and is comparable in strength to similar units occurring in the lower third of the Cucaracha formation and the upper portion of the Culebra. Thickness variations beneath the test site reflected the presence or absence of shaly interbeds. A layer rich in oyster shells was used as a reference in determining the subsurface structure.

34. A "coaly" shale layer immediately below the sandstone is an impure form of lignite and, when dried, can be burned. It is essentially carbonized wood, the original fibrous structure of which was evident in some of the core samples. Slickensides along bedding were notable. No pockets of gougy clay were revealed by the borings, and the general aspect of this lignitic shale suggested that it is stronger than the carbonaceous shale located 50 ft above, probably equal in bearing strength to unfaulted green shale.

35. No evidence of crushing or movement of the rock was noted that could be attributed to loading of the test structure. The samples were similar in all respects to those recovered from other nearby exploratory borings.

PART III: PRE-1942 TESTS AND EARLY DESIGN VALUES FOR BEARING CAPACITY, MODULUS OF ELASTICITY, AND SHEAR STRENGTH

36. Early Third Locks Project design values proposed an allowable bearing pressure of 10 tsf and a modulus of elasticity of 70,000 psi for the Cucaracha formation. Early values adopted for shear strength for use in lock wall design were cohesion  $c$  of 1.8 tsf (25 psi) and the coefficient of friction  $\tan \phi$  of 0.42. Field bearing tests and laboratory triaxial and unconfined compression tests on the Cucaracha dark gray moderately slickensided sandy shale encountered in Pedro Miguel Test Pit No. 2 gave values for ultimate strength that varied from 60 to 105 tsf and modulus of elasticity that varied from 23,200 to 245,000 psi. Early tests were made on cores obtained by drilling with water, which resulted in damage to the cores and poor recovery. The later use of drilling mud, although it increased the percentage of core recovery approximately 30 percent, did not enable recovery of the weakest phases of the Cucaracha. Consequently, laboratory tests on small samples were indicative of strengths of the stronger phases of the formation only. Furthermore, samples tended to shear or fail on weak slickensided faces thereby producing highly irregular results.

37. A special type of bearing tests was developed in an effort to devise a suitable laboratory test for the Cucaracha shale. A 3-in.-diam plate was loaded on the surface of a block of shale about 10 in. in diameter and 8 to 12 in. deep. The sample was confined by grouting it in a steel cylinder. These tests resulted in values for ultimate strength ranging from 28 to 110 tsf and values of modulus of elasticity ranging from 12,500 to 102,000 psi. These tests confirmed the observation that samples loaded slowly failed at lower pressures than those loaded rapidly. In the light of this evidence and since a large part of the Cucaracha formation consists of slickensided clay shale, it was decided to excavate Test Pit No. 3 at the location shown in fig. 1 so that shear and bearing tests might be made on the more slickensided phases of the Cucaracha clay shale.

#### PART IV: PEDRO MIGUEL TEST PIT NO. 3\*

38. Materials encountered in the shaft of Test Pit No. 3 and in the drift excavated at depths between 30 and 36 ft are described in Part II. Field shear tests and field bearing tests were performed on the floor of the drift between the timber support frames as shown in fig. 5. The test location is identified by the number of the bay in which it was located.

39. For comparison with the results of the field shear tests, laboratory tests were performed on 2-in.-diam specimens of Cucaracha clay shales obtained from blocks taken in the test pit, and from various borings. The laboratory tests consisted of triaxial compression tests, friction tests on precut polished surfaces, and unconfined compression tests. Laboratory bearing tests were performed on undisturbed block samples for comparison with the field bearing tests.

#### Field Shear Tests

##### Uses of apparatus and equipment

40. A 1-ft cube of concrete was poured on a grooved surface of the rock, and a normal load was applied to the block by placing a 50-ton-capacity hydraulic jack on it and jacking against the ceiling of the drift. In order to distribute the jack pressure evenly over the entire area of the concrete block, a steel plate was grouted to the top of the block. Roller bearings were placed between the jack and the plate to allow free horizontal movement of the block.

41. The lateral load was obtained with another 50-ton-capacity hydraulic jack pushing against a thrust bar fastened to the block with stud bolts set in concrete. This thrust bar was designed to permit the lateral jack pressure to be applied in the same plane as the base of the test block. As a resistance for this jack, a small concrete slab with a vertical face was poured against the wall of the drift. A

---

\* Material for this part of the report is taken largely from Vincent.<sup>4</sup>

swivel head was placed between the jack and the wall to provide a perfect bearing between the jack piston and the thrust bar.

42. Jack loads were measured with a pressure gage in the oil reservoir. These gages were calibrated with the jacks before being used in the field and again at the conclusion of the test. The latter calibration was used for the computations. Movements of the concrete blocks during application of the normal load and during the shear tests were measured with micrometer gages supported from channels fastened to the timbering of the drift.

#### Test procedures

43. The site for each test was prepared by first removing all disturbed material from the floor of the drift and then making a smooth and level surface by cutting horizontally through the clay shale with a hand ice saw. The smooth surface was then grooved by using a saw-toothed scraper made especially for this purpose. This scraper had seven sharpened teeth designed to make grooves with a triangular cross section 1 in. deep and 2 in. apart. When these grooves were finished, the clay shale surrounding the 1-sq-ft area was cut to the level of the bottom of the grooves.

44. When the site had been prepared as described above a concrete form box was placed over the grooved surface. A 1-in.-thick layer of grout was spread in the bottom of the form and tamped. Then the box was filled with concrete and thoroughly rodded, and the top was smoothed off with a trowel. Immediately prior to testing, a small, 1- by 1-in. channel was cut into the clay shale around the base of the block. This was done to remove any lateral restraint around the boundary of the joint between the rock and the concrete.

45. Normal loads were applied to the blocks in increments of 1 tsf per 2 min. In all the tests except that of the first block, the settlement of the block under each increment of load was recorded. In tests of blocks 3, 4, 5, and 6, two or more cycles of load and release, in increments, were applied prior to the actual application of the shearing load. It was hoped that the settlement data so obtained would be valuable in the analysis of the bearing tests. Results of the

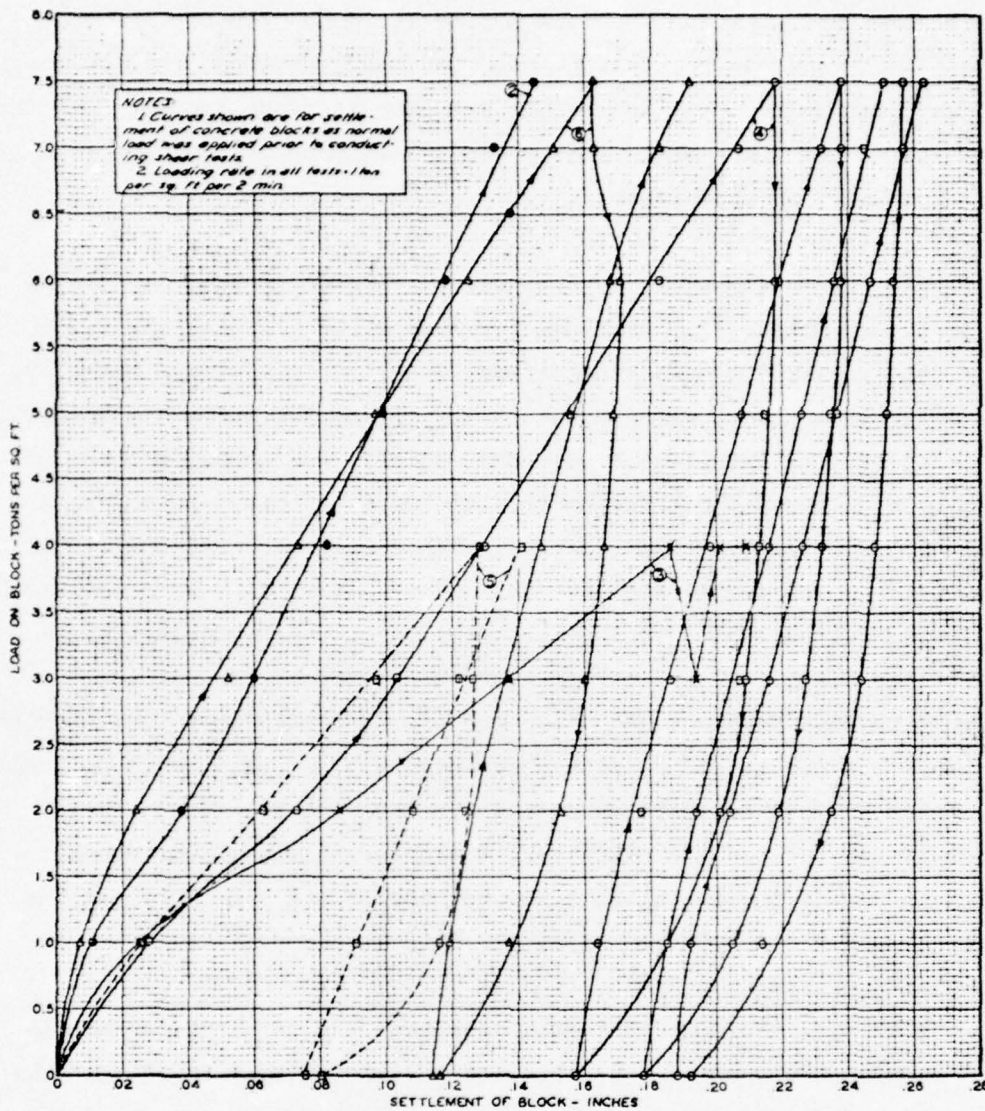


Fig. 9. Load versus settlement of concrete blocks prior to conducting field shear tests, Pedro Miguel Test Pit No. 3

settlement tests are shown in fig. 9 and table 1.

46. When the normal load had been applied, the lateral pressure was gradually increased in increments until failure had been reached. The loading rate, which was arbitrarily determined, was about 0.25 ton/min. Readings of movement of the block became increasingly greater with increases in pressure, up to a peak, when the horizontal

force necessary to keep the block moving began to drop off. When the block had failed and a friction value had been obtained, the horizontal pressure was released in increments and then increased again until the block moved out once more. This procedure afforded a check of the friction value after failure.

#### Test results

47. Six shear tests were conducted on grooved surfaces of the shale in the field. Two of these tests were run at 4 tsf and the other four were run at 7.5-tsf normal stress load. In all of the tests, failure occurred in the Cucaracha but at varying depths below the bottom of the grooves. Failure, which was attained very gradually, was considered to be the point at which the horizontal force necessary to maintain continuous movement of the block began to decrease. This shear stress defined what was termed the "ultimate shear strength." When the block had moved out for some distance after failure, the pressure was released and again applied in increments until the block once more started to move continuously while the pressure was maintained constant. The point on the second cycle beyond which the pressure could no longer be increased defined what was termed the "sliding friction value." Figs. 10 and 11 show the results of the observations of horizontal movement versus horizontal load for the tests, and Fig. 12 shows the ultimate shear strength and sliding friction values.

#### Discussion of test results

48. Fig. 12 shows that good agreement was obtained between the two tests at 4-tsf normal load but the ultimate strengths of the four tests at 7.5-tsf normal load varied over a large range. An analysis of the variations in the material and of the irregularities in the type of failure produced in each test was made in an effort to determine which, if any, of the tests could be omitted as not representative. A summary of the factors considered is presented in the following paragraphs.

49. Shear test 1, bay 3. The material in which the grooves were cut was predominantly green in color, with some purple being present, and was harder than that in any other bay, with concretion up to 3/8 in. in diameter. Slickensides in the surface were few and minor. However,

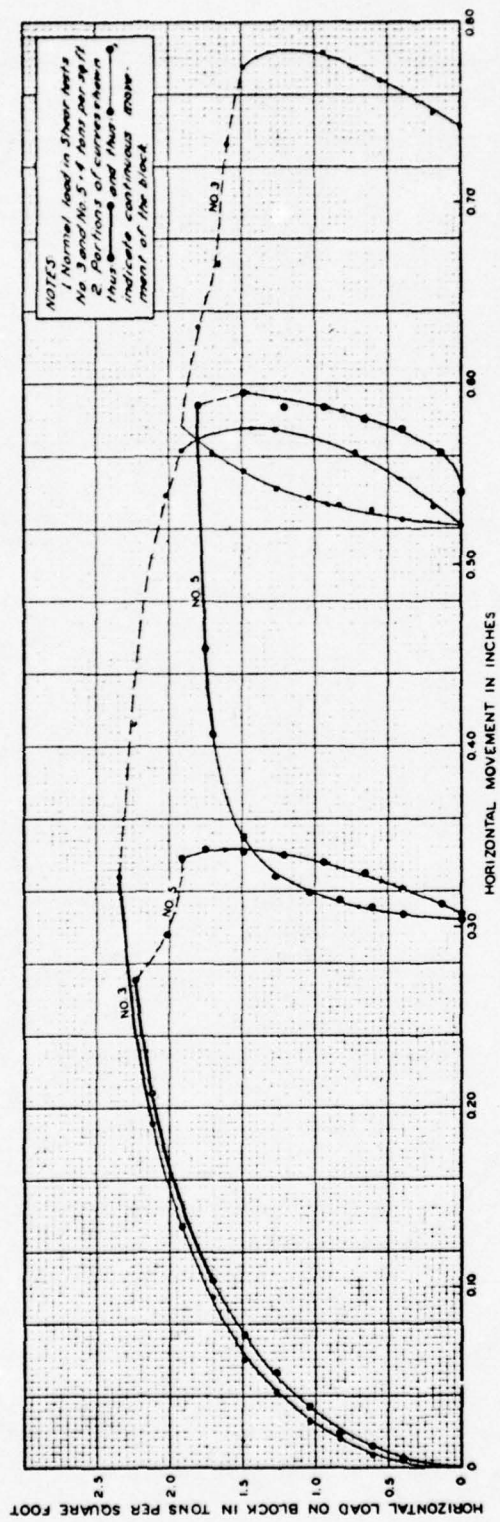


Fig. 10. Load versus movement, field shear tests 3 and 5, Pedro Miguel Test Pit No. 3

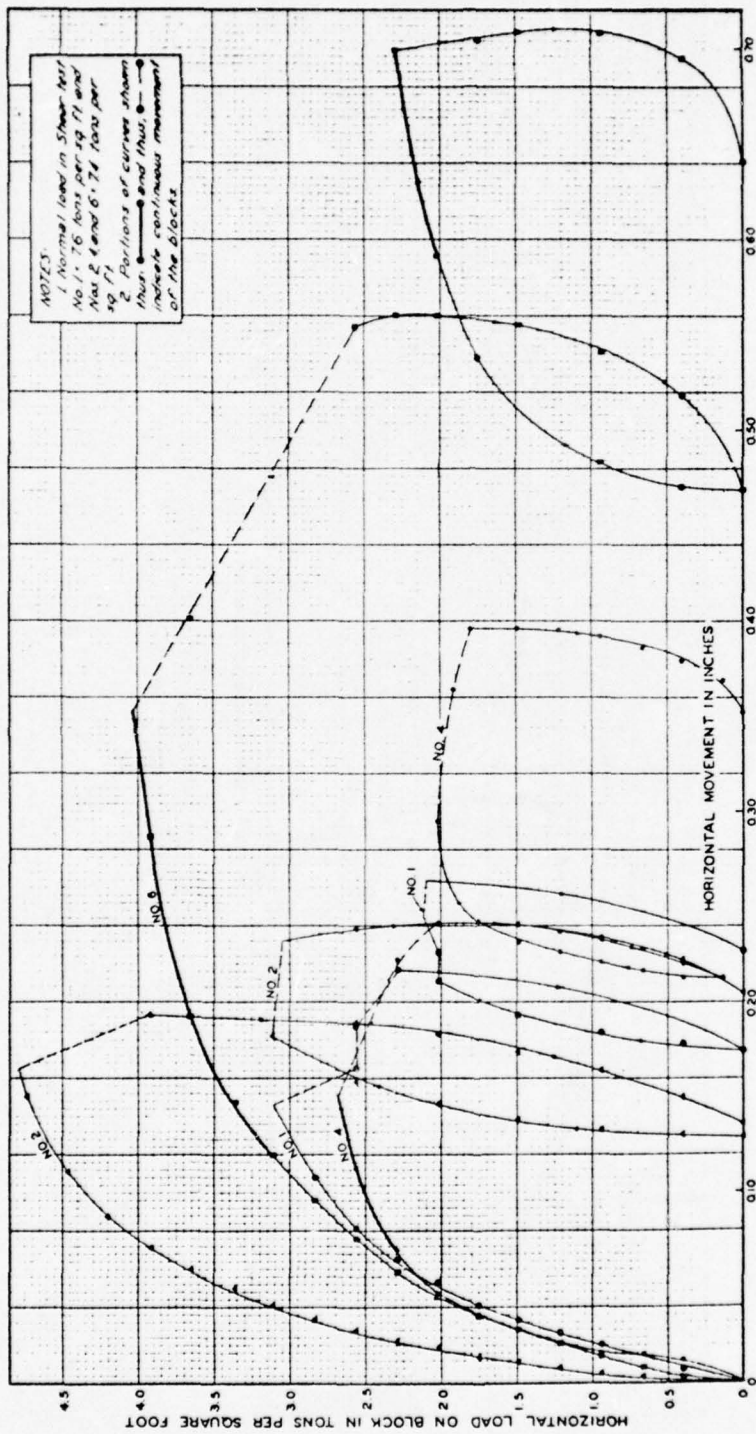


Fig. 11. Load versus movement, field shear tests 1, 2, 4, and 6, Pedro Miguel Test Pit No. 3

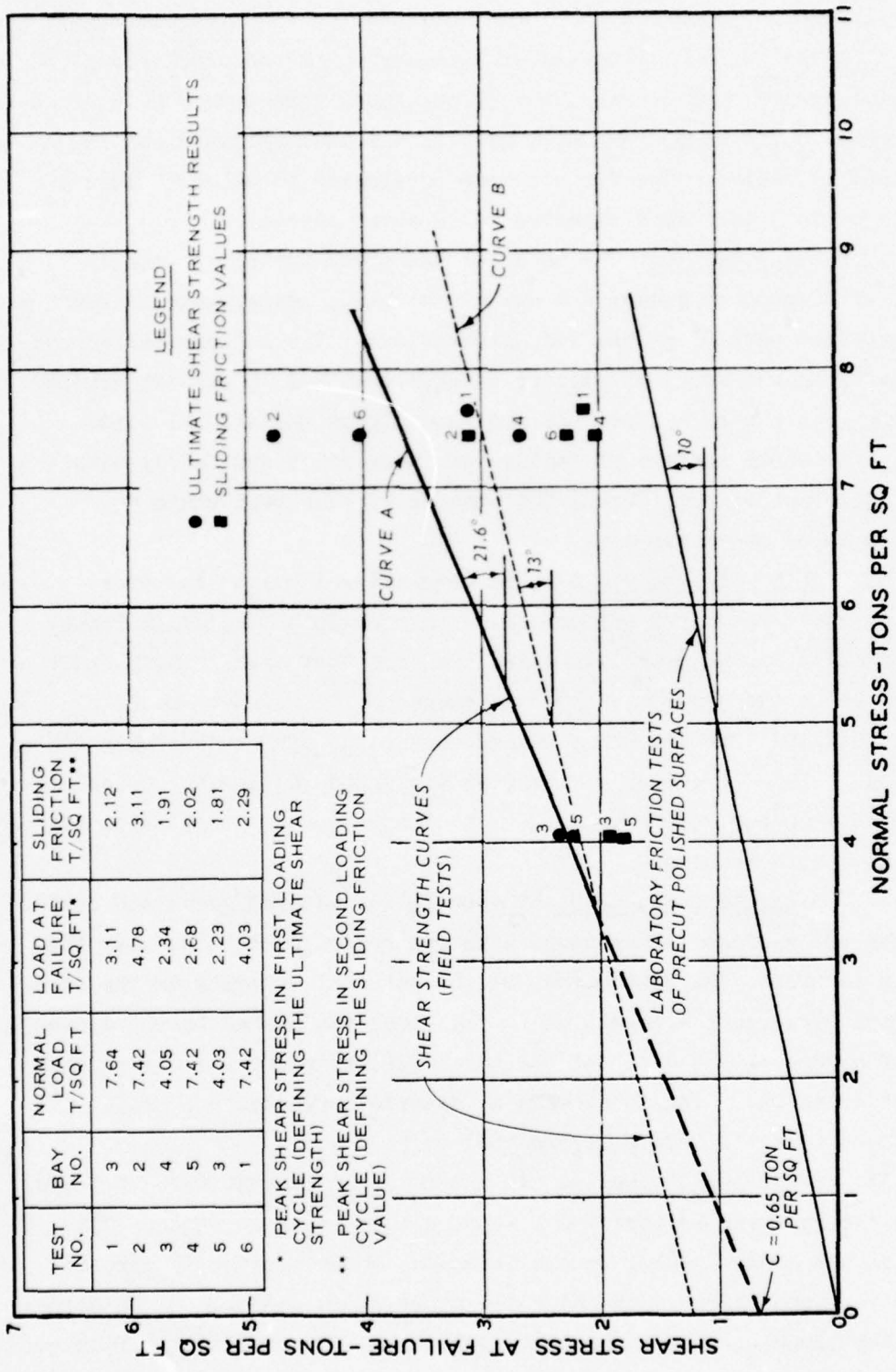


Fig. 12. Shear strength curves for field tests and laboratory friction tests, Pedro Miguel Test Pit No. 3

the failure plane occurred at a slickenside below this lens of harder material. The bottom face of material adhering to the block along which failure took place was about 30 percent slickensided; but, since failure was quite deep, the total mass of material affected was fairly large and extensive. The factors were considered to balance, so the results of this test were expected to be about average.

50. Shear test 2. The material was predominately purple in color, with spots of green, and was medium hard, homogeneous in texture, and contained several pronounced slickensides. The failure surface below the block was about 65 percent slickensided and 35 percent crushed. The block was pushed against the moderate dip of the natural slide plane. The whole surface of failure was very rough and fairly extensive. All factors considered, the results of this test would be expected to be above average.

51. Shear test 3, bay 4. The material was purple in color, slightly softer and more uniform than that in any of the other tests, and contained two major slickensides near the test area. The failure surface under the block was about 40 percent slickensided and 60 percent broken. The block was pushed across the direction of dip of the rock. The entire mass of material disturbed was large. Consideration of all these factors would lead to the expectation that this test would give high results.

52. Shear test 4, bay 5. The shale was mottled green and purple in color and average in hardness, with one major slickenside near the grooved surface. The bottom face of the material adhering to the block was about 90 percent slickensided. The block was pushed in the direction of a very slight dip, and the total failure plane was quite rough but not extensive. The total mass of material affected was small. The results of this test would be expected to be low.

53. Shear test 5, bay 3. It should be noted that this test was run in the same bay as test 1 but about a foot lower in depth. The material was predominately purple in color, about average in hardness with a few concretions, and had a few minor slickensides. Slickensides under the block were approximately horizontal. The failure surface was

about 90 percent through slickensides and 10 percent crushed. The whole failure plane was very rough but shallow so that the mass of material disturbed was small. These variations would indicate that an average value for the test results would be anticipated.

54. Shear test 6, bay 1. The material in this bay was predominately purple in color and a little harder than average, with concretions up to 1/4 in., and had several minor slickensides in the surface. The major slickensides near the block were nearly vertical so that the failure surface was only 20 percent through slickensides and the remainder was broken and crushed. The entire failure plane was rough, medium deep, and extensive. These factors would lead to the expectation that this test would produce very high results.

55. It is important to note that the location of the failure in every test was governed by the position of the slickensides under the block. The path of least shearing resistance was along the slickensides even when this slickenside was 5 or 6 in. below the level of the application of the lateral thrust.

56. From the examination of the materials and the failure surfaces it would be expected that, of the tests at the 4-tsf normal load, test 3 would be somewhat higher than test 5, whereas the actual results were about the same. Of the tests at the 7.5-tsf normal load, test 4 was, as expected, lower than test 1, but tests 2 and 6 were exactly reversed in order of magnitude though they were both, as expected, higher than tests 1 and 4. It was concluded that the range of values indicated by the results of the tests at the 7.5-tsf normal load is a characteristic of the mass in which the tests were performed. Therefore, each test was given equal weight in establishing the curve of ultimate strength for the field shear tests. This shear strength curve, which is shown in fig. 12, indicates a  $c$  of 0.65 tsf and a  $\phi$  of 21.6 deg.

#### Laboratory Shear Tests

57. Samples of Cucaracha clay shale from the proposed New Pedro

Miguel Lock approach area were obtained by preserving cores from the drill holes. Additional samples were obtained by drilling cylindrical specimens from irregular blocks taken from Pedro Miguel Test Pit No. 3.

#### Triaxial compression tests

58. Triaxial compression tests were made on 2-in.-diam by 5-in. cores. The shear strength curve obtained by averaging the results of 18 triaxial compression tests is shown in fig. 13. It should be pointed out that this curve is based on tests of sound clay shale samples which showed no evidence of slickensides and fractures. Consequently, it was expected that this strength would be higher than that found for the field tests in which slickensides and fractures were present to weaken the material.

#### Laboratory friction tests

59. Friction tests on the Cucaracha clay shale were made in the laboratory to compare the results with the values determined from the field tests. These tests were made on short lengths of 2-in.-diam cores. The surfaces to be tested were carefully polished with carborundum stones and by rubbing the shale surfaces together. For testing, the pieces were held in place in a direct shear box with plaster of paris. In some of the tests, a normal load was applied and the friction value was determined immediately. In others, the normal load was applied for a period of 1 hr prior to shearing. There was little difference in the results obtained from the two methods. The curve established from the results of the laboratory tests is shown in fig. 12. The friction angle was found to be about 10 deg.

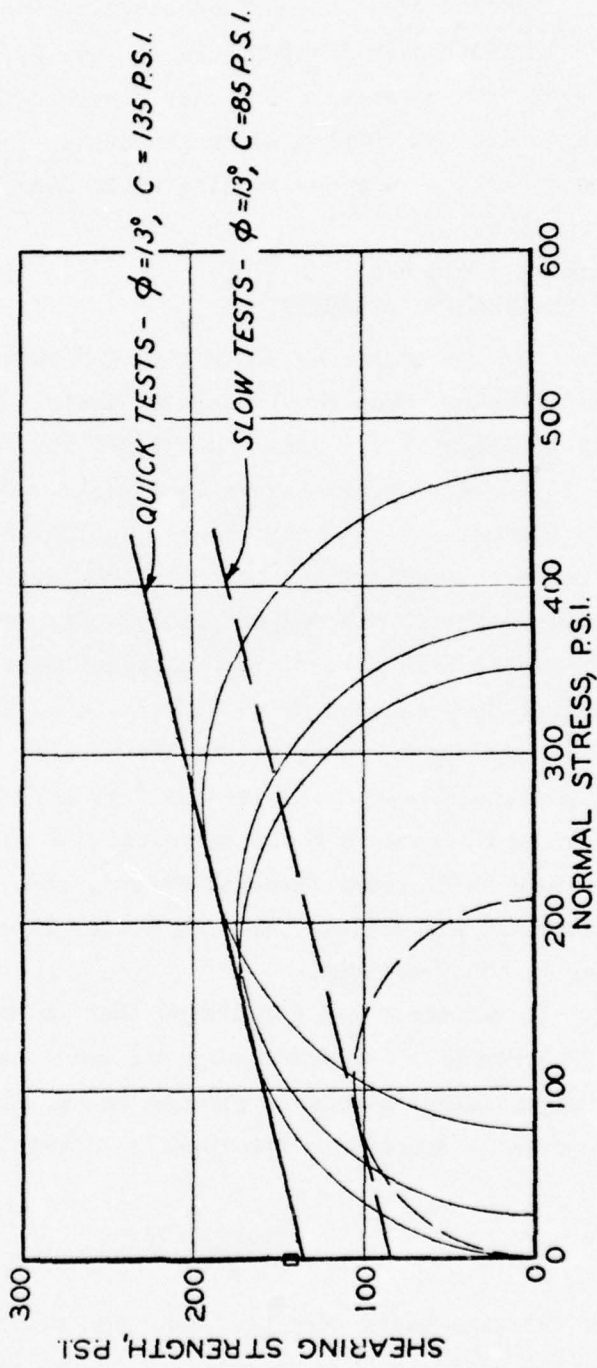
#### Unconfined compression tests

60. The samples tested were drill cores of Cucaracha clay shale approximately 2 in. in diameter by 5 in. long. The specimens were loaded to failure either by very slow or by quick application of load. For the slow tests, loads were applied in increments of about 15 psi per day, while in the quick tests the rate used was about 15 psi per minute. The purpose of running both quick and slow tests was to ascertain whether there was the same considerable reduction in the ultimate strength under the slower application of load in this

SUMMARY OF TESTS

LATERAL STRESS P. S. I.	LOADING RATE	ULTIMATE STRENGTH				NO. TESTS
		MAX. P. S. I.	MIN. P. S. I.	MEAN P. S. I.	AVE. P. S. I.	
0	SLOW	463	69	210	214	20
0	QUICK	550	138	338	349	14
25	"	623	147	310	374	9
75	"	609	317	449	468	9
SINGLE SHEAR	"	215	78	151	144	5

NOTES:  
 FULL DEFORMATION UNDER EACH INCREMENT OF LOAD IN SLOW TESTS  
 AVERAGE STRENGTHS USED IN MOHR'S CIRCLE PLOTS.  $\phi = 13^\circ$  ASSUMED FOR SLOW TESTS  
 AVERAGE OF SINGLE SHEAR TESTS SHOWN THUS  $\phi$



type of test as had been noticed for the field and laboratory bearing values.

61. Results of 34 unconfined compression tests (14 quick and 20 slow) are summarized in tabular form in fig. 13, and average values are plotted as Mohr's circles in the same figure. Test data included in the averages are for samples generally better than the average clay shale and do not include any results which were known to be influenced by slickensides.

Discussion of field and  
laboratory shear test results

62. Fig. 13 shows the shear strength curve of the Cucaracha shale as determined from the laboratory tests. The values of 13 deg for the angle of internal friction and 135 psi for cohesion are at variance with the field test results shown in fig. 12. The laboratory data should be higher since the only tests considered for the final analysis were on Cucaracha shale which on the average was slightly harder than the test pit material and did not include any test in which the specimen was known to have failed along slickensides. If it is assumed that the effect of the slickenside would only be to lower cohesion, then the angle of 13 deg might be applied to the results of the field shear tests. This would produce curve B in fig. 12. Such a curve, if used for design, would in effect disregard tests 2 and 6. However, no reason could be found to discount these two tests, and, more important, the field shear test was believed to be a better representation of the conditions in the prototype.

63. It was therefore considered that an angle of internal friction of 21.5 deg and a cohesion of 9 psi were the most accurate ultimate shear strength values available for use in analyzing foundation bearing capacity of walls founded on the type of material tested.

Bearing Tests

64. Bearing tests were performed for two purposes: First, to determine the ultimate bearing capacity of the foundation material, and

second, to determine the settlement that could be expected for structures founded on clay shale. It was considered that the bearing capacity might govern the design of the lock walls if the settlements under the allowable bearing capacity as determined from the tests were found to be small. If the settlements were found to be excessive for loads less than the ultimate bearing capacity, the design would be modified so that the loads would not produce settlements greater than the structure would withstand.

### Field Bearing Tests

#### Use of apparatus and equipment

65. The apparatus used to determine the bearing characteristics of the shale consisted essentially of (a) a rigid circular plate placed on a prepared shale surface rock, (b) a hydraulic jack for applying the desired load on the plate, and (c) a system of micrometer gages to indicate the amount of settlement. The plates used in this series of tests were 8, 16, and 40 in. in diameter. The largest of these (40 in.) was a truncated cone-shaped casting 12 in. high. The other two were of forged steel, each 3 in. high with four holes in the sides drilled and tapped for a 1-in. bolt.

66. The load on the plate was applied with a 150-ton-capacity hydraulic jack which was placed directly on the plate. A length of heavy I-beam was used as a thrust beam between the jack and the ceiling of the drift. Jack loads were measured with pressure gages in the oil line of the jack. Plate settlement was measured with micrometer dial gages supported from channels grouted into the walls of the drift. In the case of the 40-in.-diam bearing plate, dial stems bore directly on the outer edge of the plate, but for the smaller sizes, the base of the jack covered the plate so that the dials could not rest directly on it. To correct this, bolts were fastened to the 8- and 16-in.-diam plates to support steel strips set in an upright position to act as bearing points.

#### Test procedure

67. A smooth level surface of the shale was prepared by making a horizontal cut with an ice saw. The bearing plate was placed on the shale with a thin layer of neat cement grout to fill any small irregularities in the surface.

68. Two types of bearing tests were conducted in this series: quick tests and slow tests. In the quick tests, the load was applied in increments fairly rapidly up to some maximum and then released. For the slow tests, the loads were applied in increments allowing deformation under each application of load before adding the next increment. In all the tests, the load was applied in increments of 2.0 tsf.

69. The test program for the first test of each size of plate consisted of four cycles of quick loading to 10 tsf in increments of 2 tsf per 2 min and a release to zero load at the same rate. The final cycle on the 16- and 40-in.-diam plates and the first test of the 8-in.-diam plate consisted of an increase of pressure to 10 tsf as before, maintenance of the 10-tsf load until all appreciable movement had ceased, and then the increase of the pressure at a rate of 2 tsf per 2 min until the shale failed. In the case of the 40-in.-diam plate, the 10-tsf load was almost the capacity of the jack pressure system so no higher loads were applied.

70. In the second and third tests of the 8-in.-diam plate, after the four quick cycles of load and release were finished, the loading used was slow, being applied at the rate of 2 tsf for 24 hr up to the failure of the shale.

#### Test results

71. A total of five bearing tests on circular plates were performed in the field. Tables 2 and 3 are summaries of the results of these tests. Figs. 14-21 show the load versus settlement curves, and figs. 22-27 show the time versus settlement curves.

72. The ultimate bearing value of 10 tsf obtained from the second test of the 8-in.-diam plate is not considered to be representative of the mass, since it was performed in the same bay as the test of the 40-in.-diam plate. Since the 10-tsf load was maintained for 55

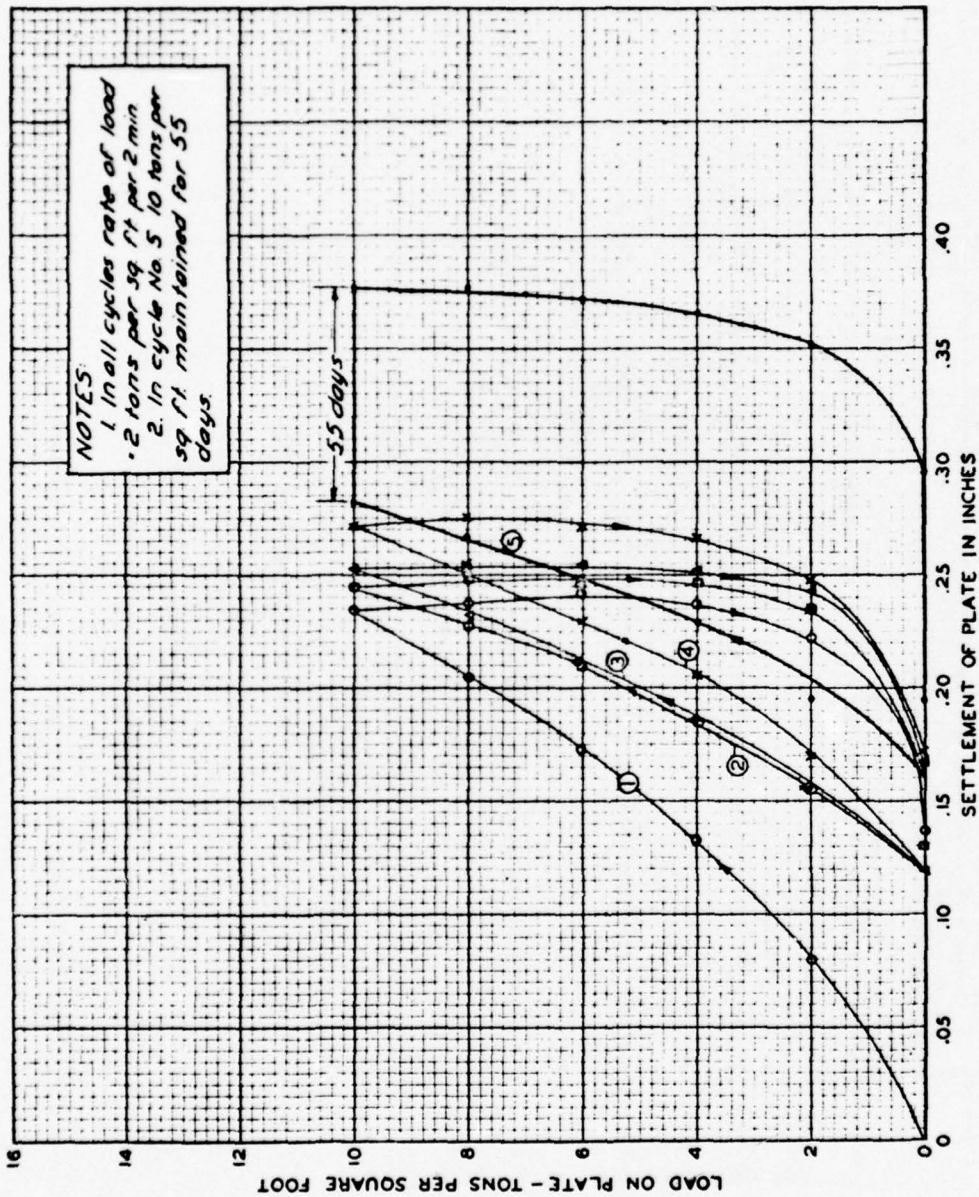


Fig. 14. Load versus settlement, 40-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3

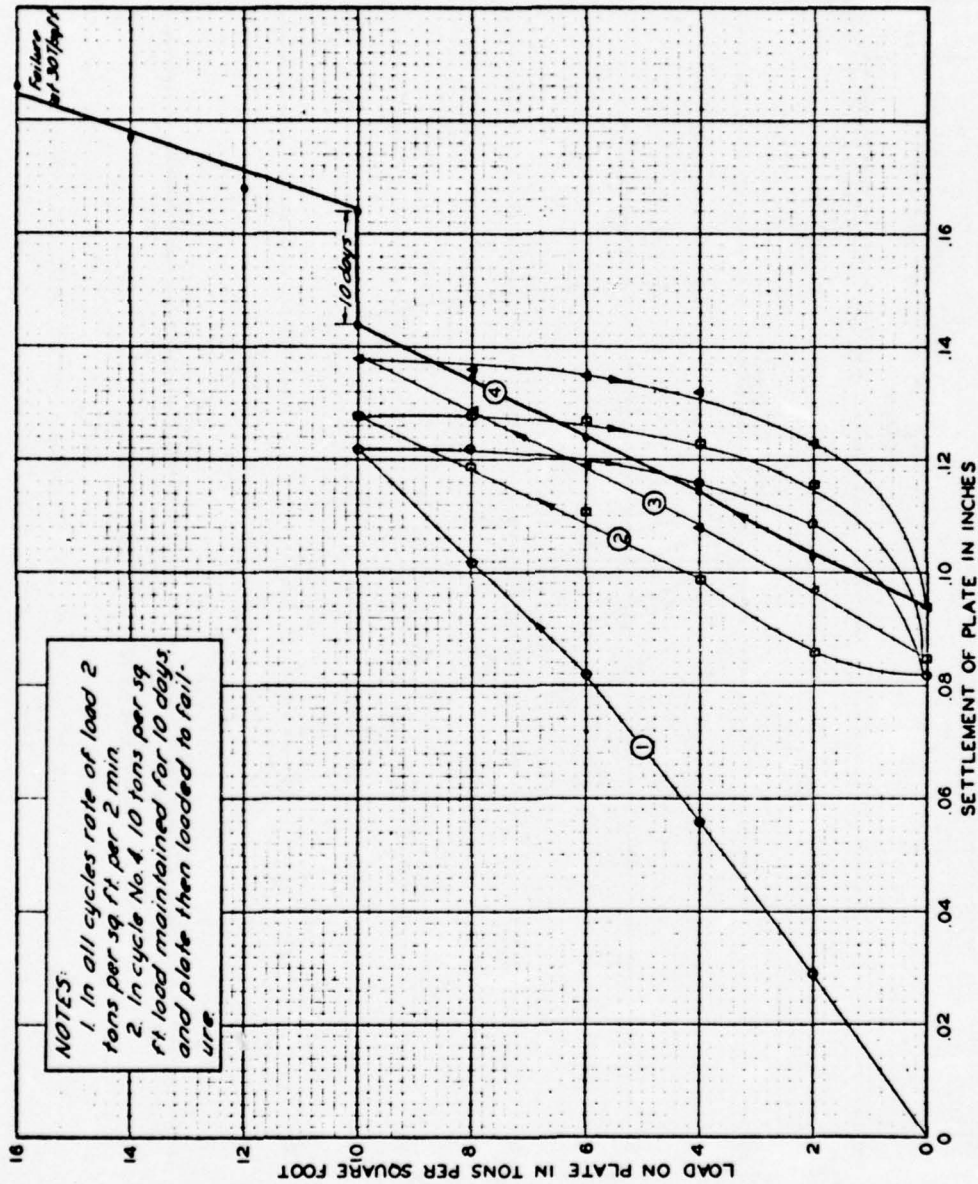
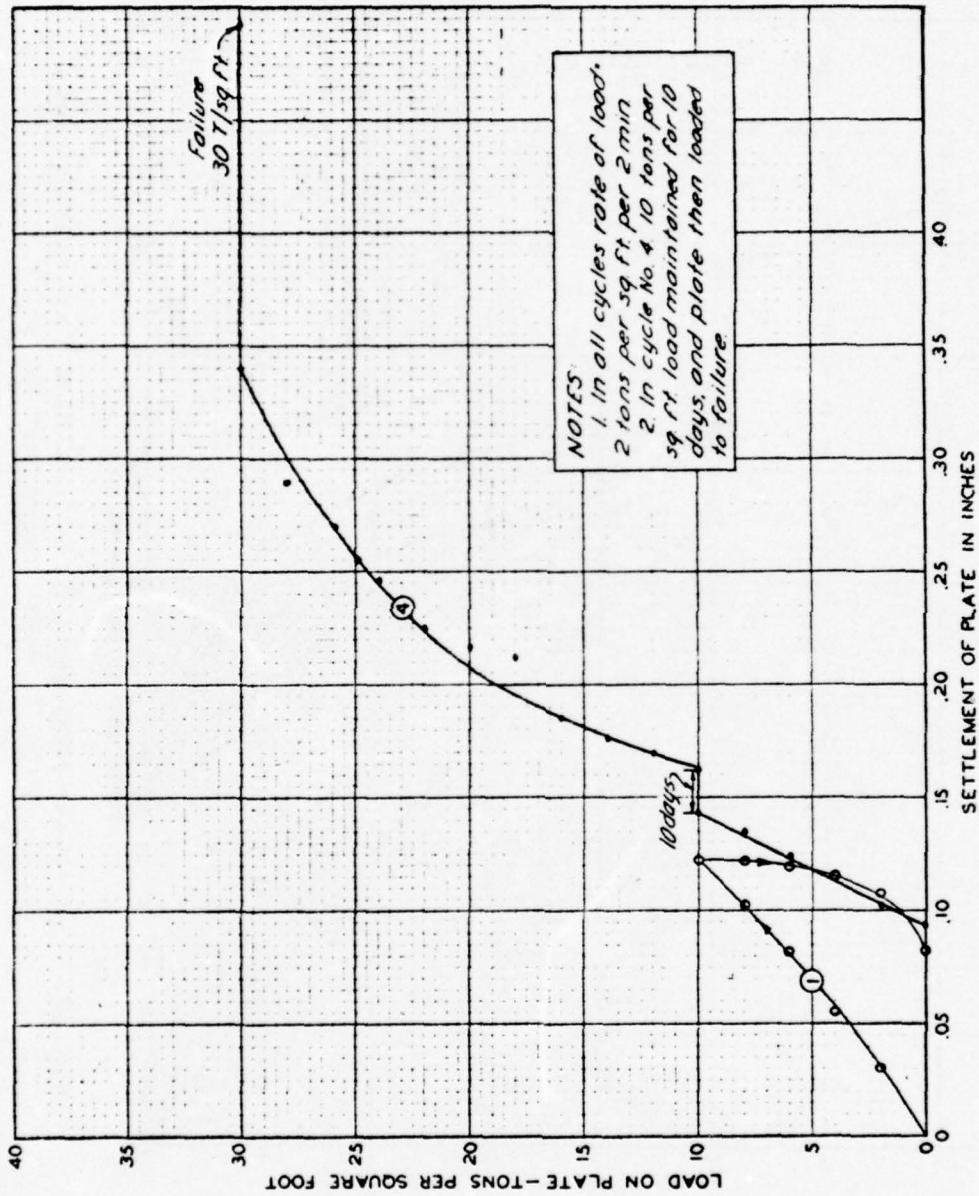


Fig. 15. Load versus settlement, 16-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3



NOTES:  
 1. In all cycles rate of load.  
 2 tons per sq ft per 2 min  
 2. In cycle No 4 10 tons per  
 sq ft load maintained for 10  
 days, and plate then loaded  
 to failure

Fig. 16. Load versus settlement, 16-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3

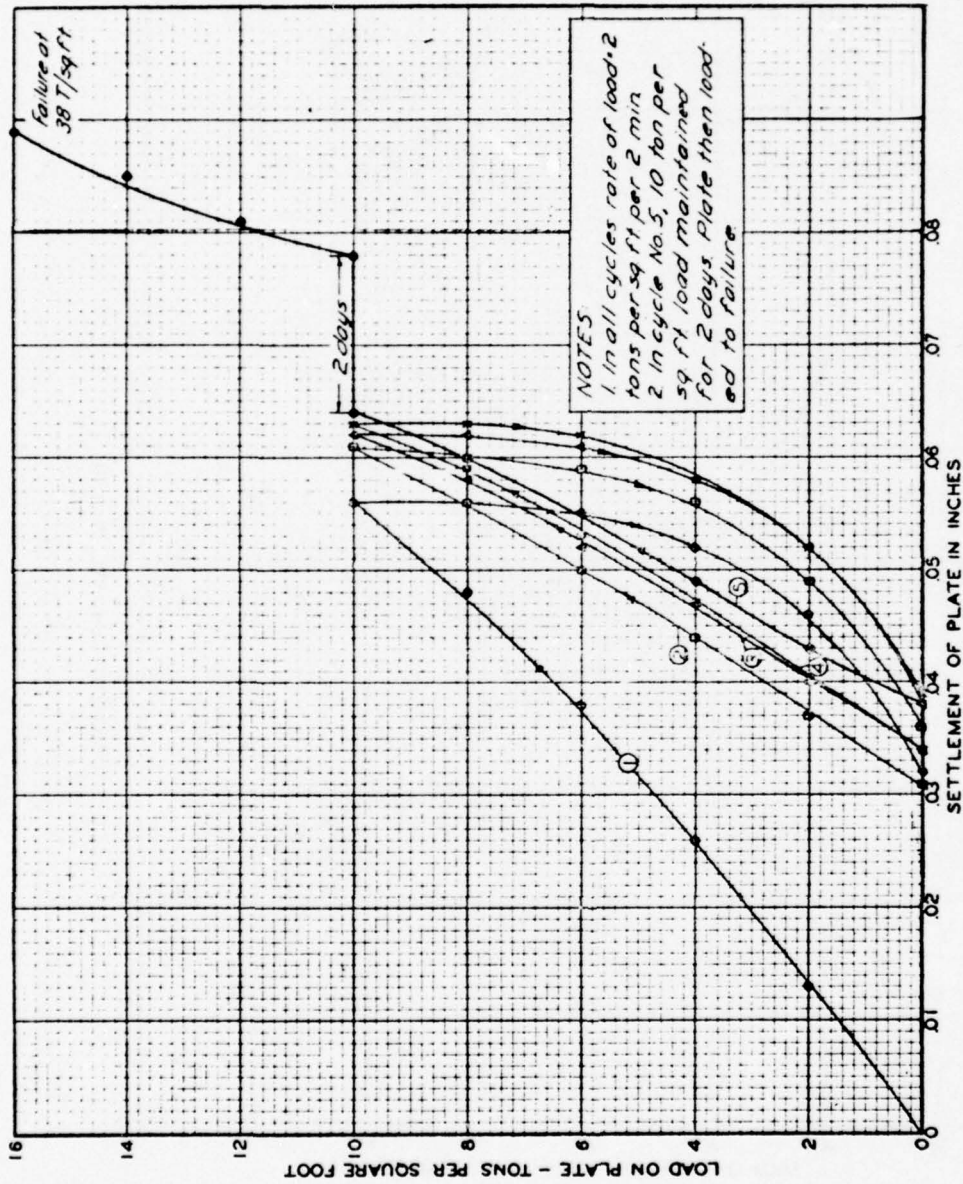


Fig. 17. Load versus settlement, first 8-in.-diam plate bearing test (cycles 1 and 5), Pedro Miguel Test Pit No. 3

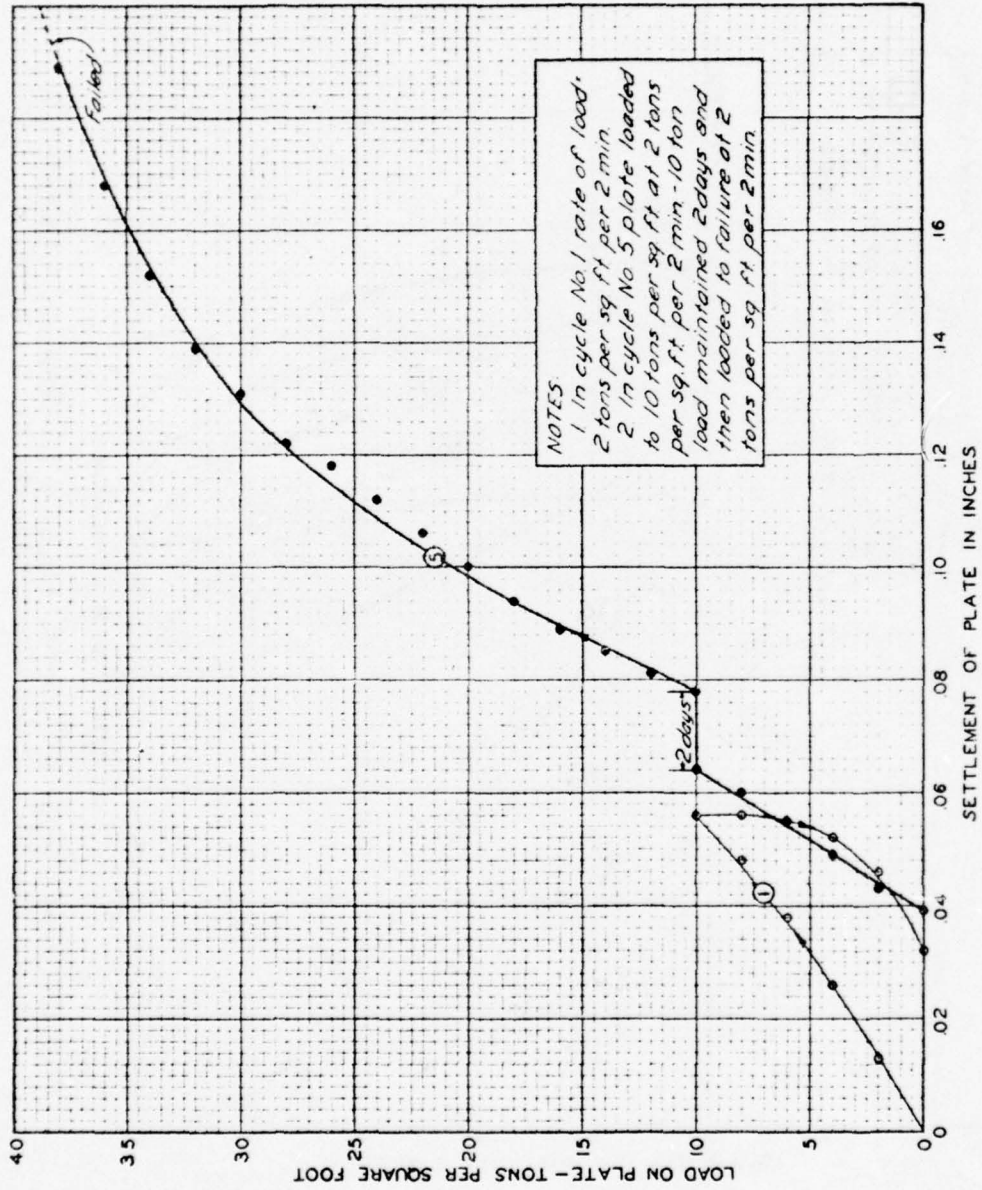


Fig. 18. Load versus settlement, first 8-in.-diam plate bearing test (cycles 1 and 5), Pedro Miguel Test Pit No. 3

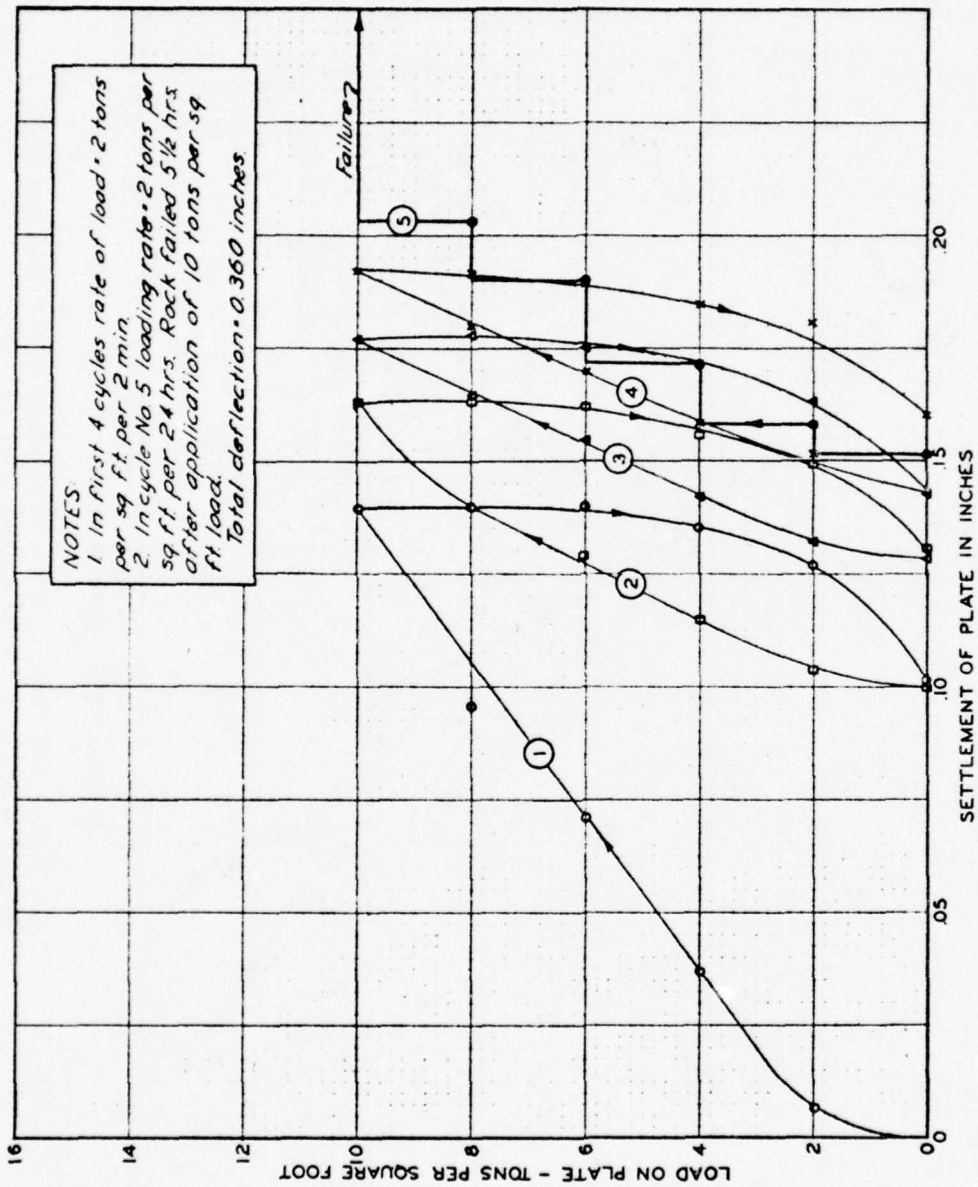


Fig. 19. Load versus settlement, second 8-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3

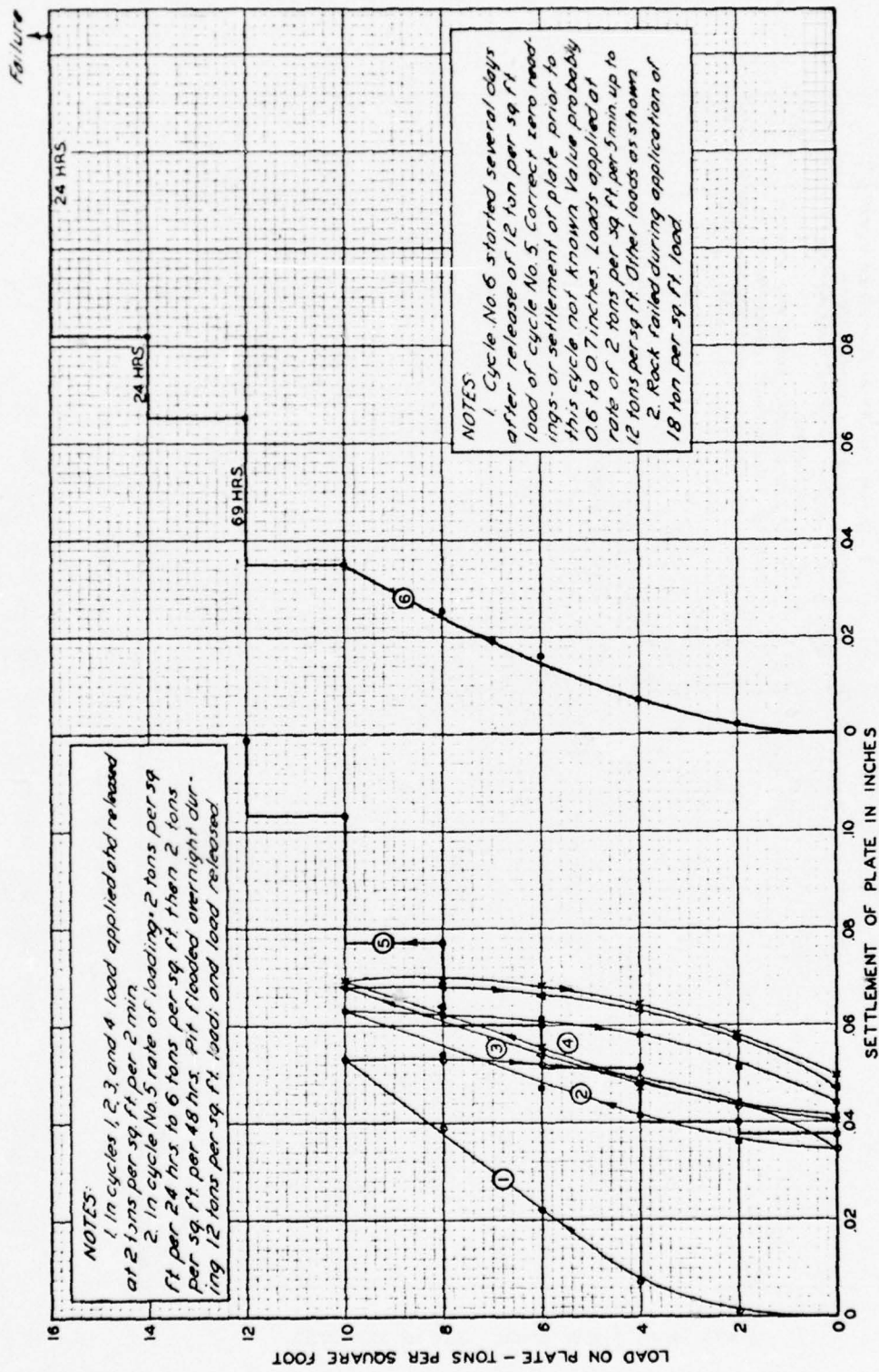


Fig. 20. Load versus settlement, third 8-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3

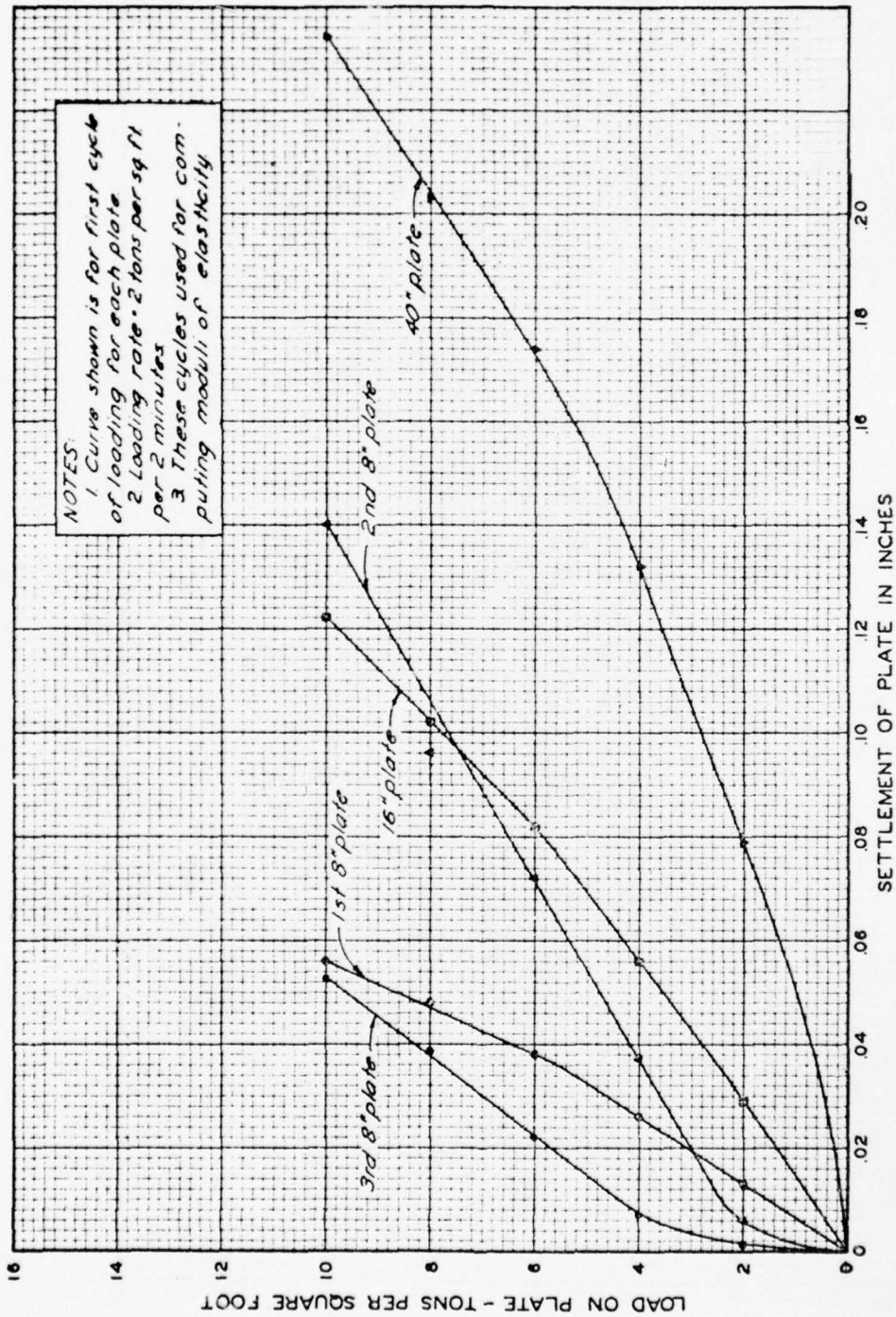


Fig. 21. Load versus settlement, first loading cycle for all field plate bearing tests, Pedro Miguel Test Pit No. 3

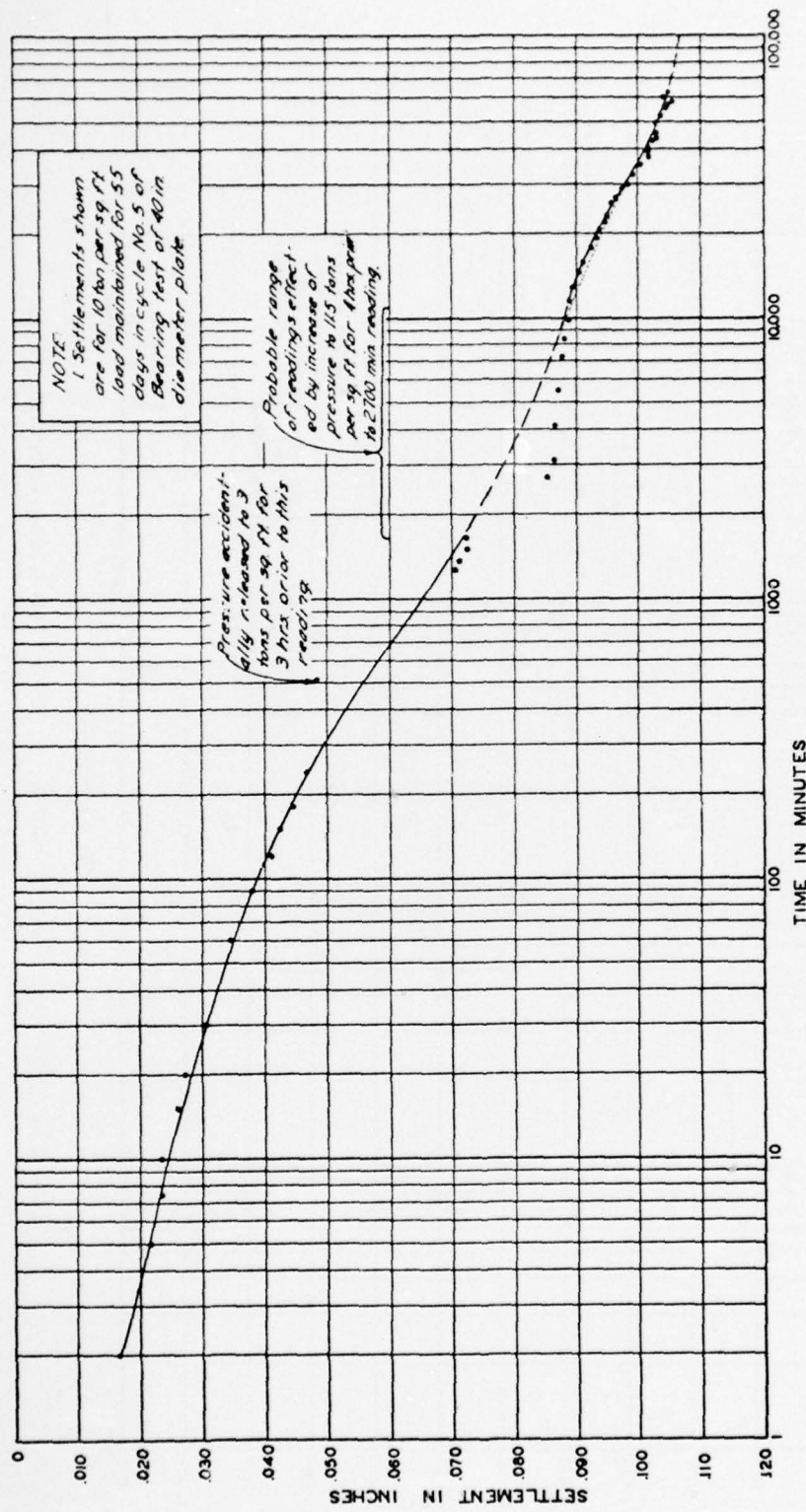


Fig. 22. Time-settlement curve, 40-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3

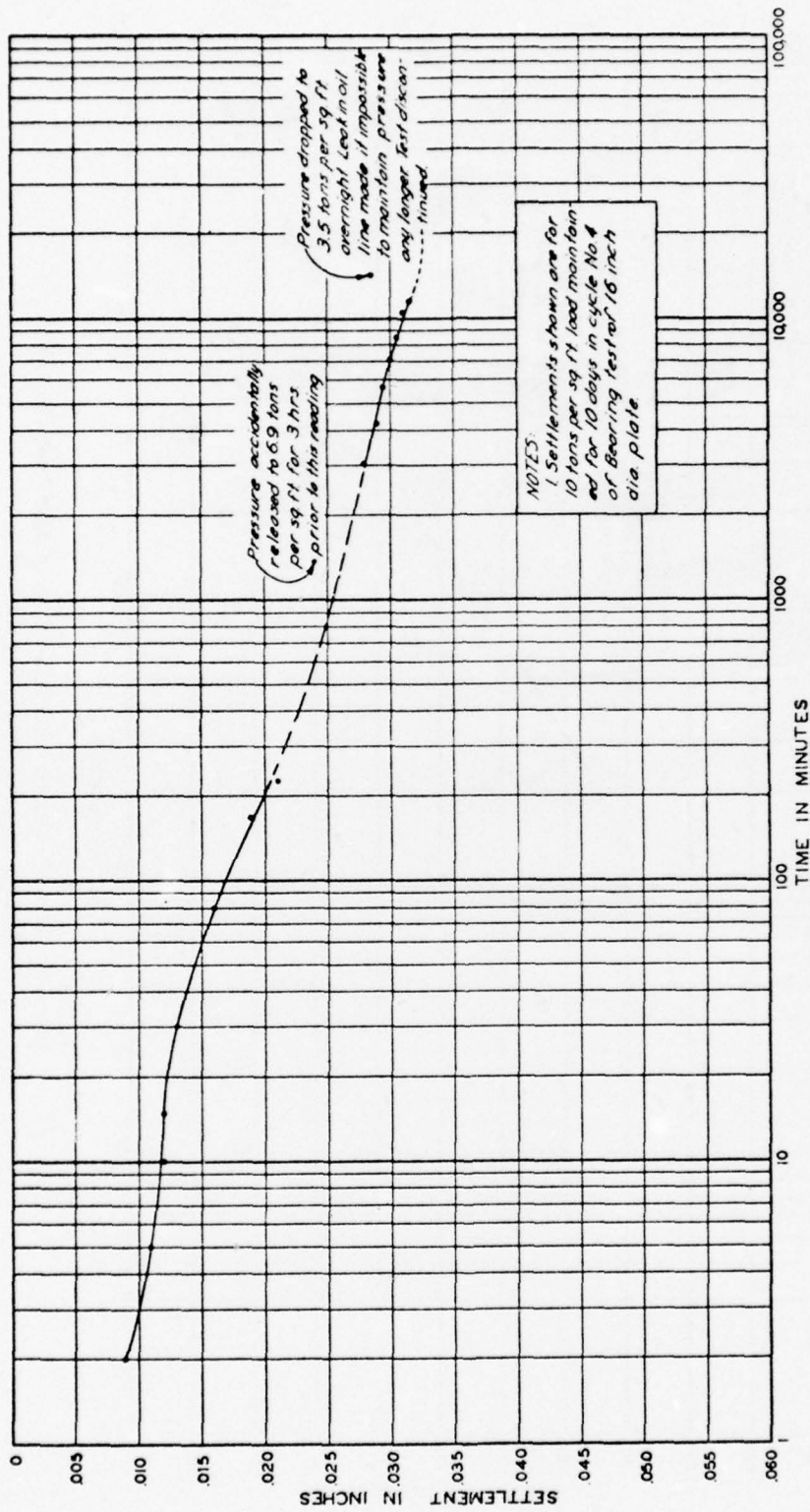


Fig. 23. Time-settlement curve, 16-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3

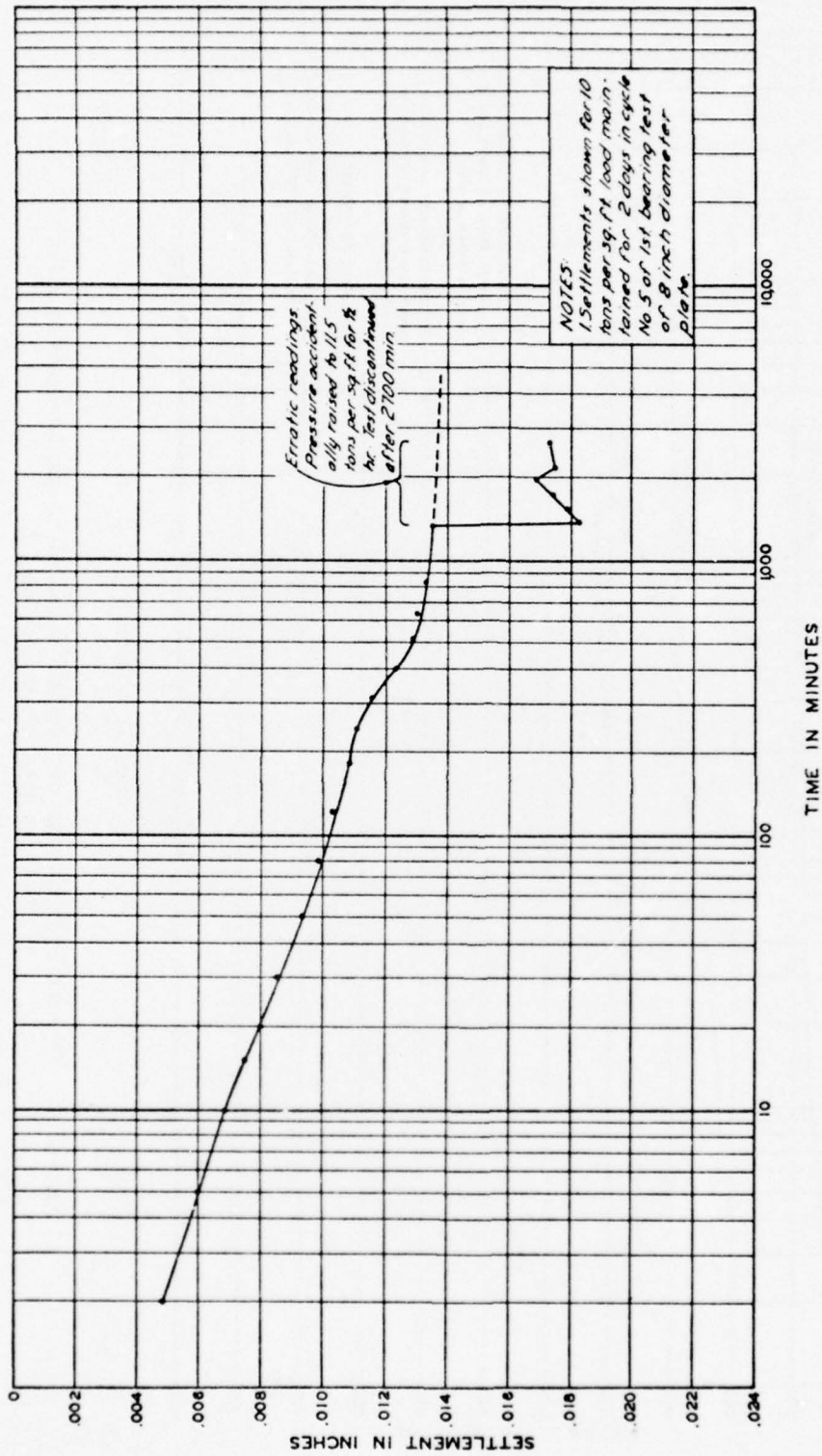


Fig. 24. Time-settlement curve, first 8-in.-diam plate bearing test,  
Pedro Miguel Test Pit No. 3

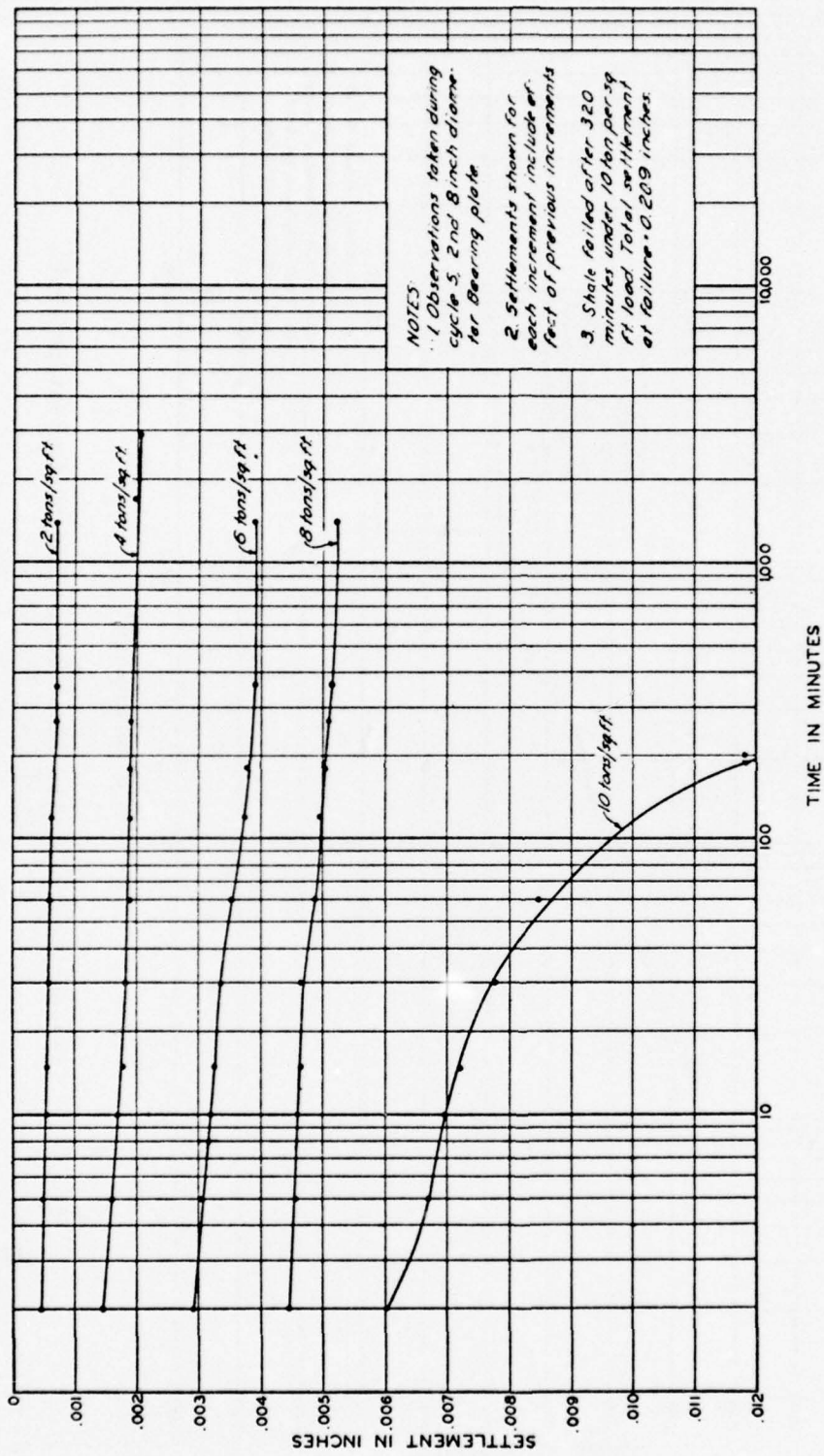


Fig. 25. Time-settlement curves, second 8-in.-diam plate bearing test, Pedro Miguel Test Pit No. 3

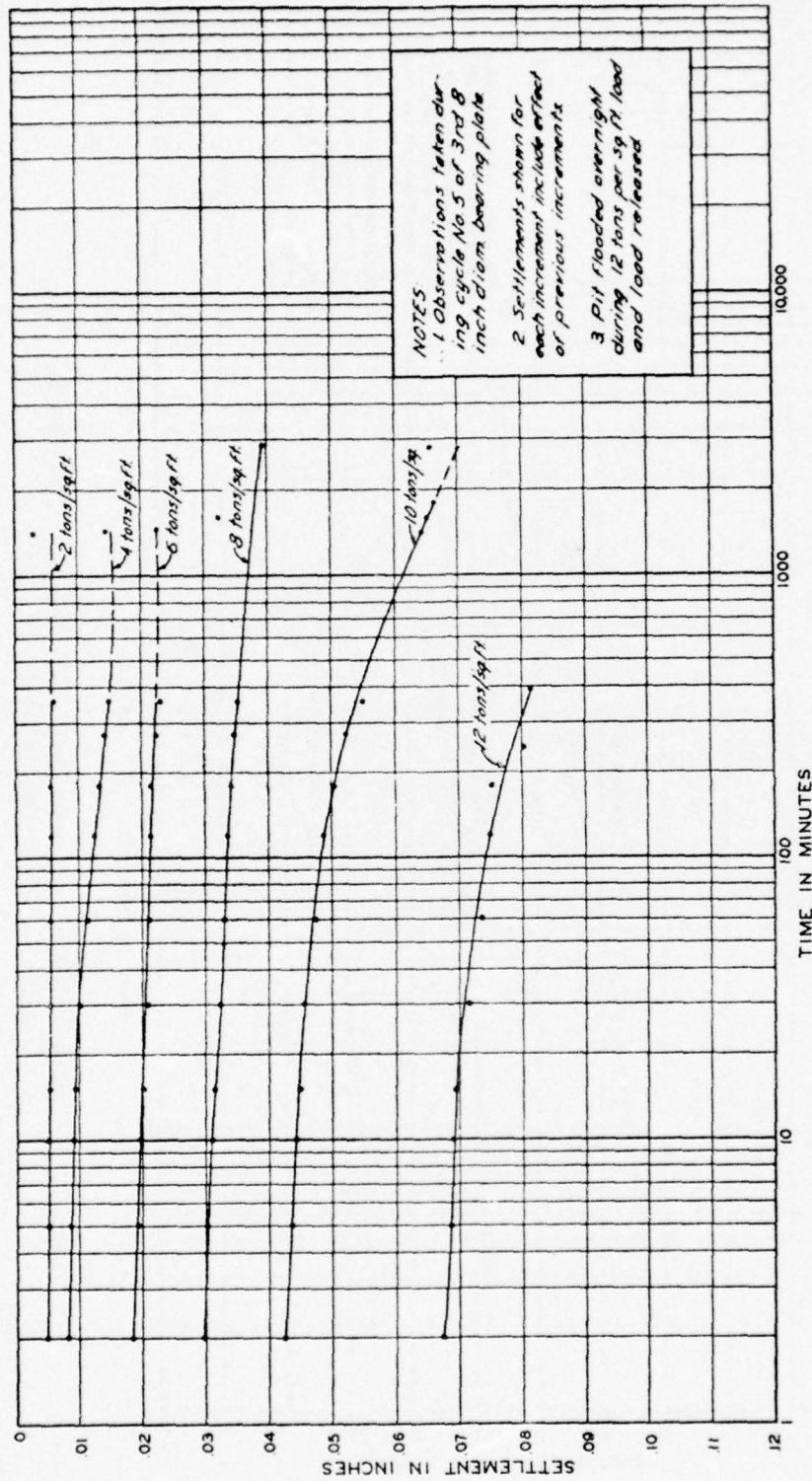


Fig. 26. Time-settlement curves, third 8-in.-diam plate bearing test (cycle 5), Pedro Miguel Test Pit No. 3

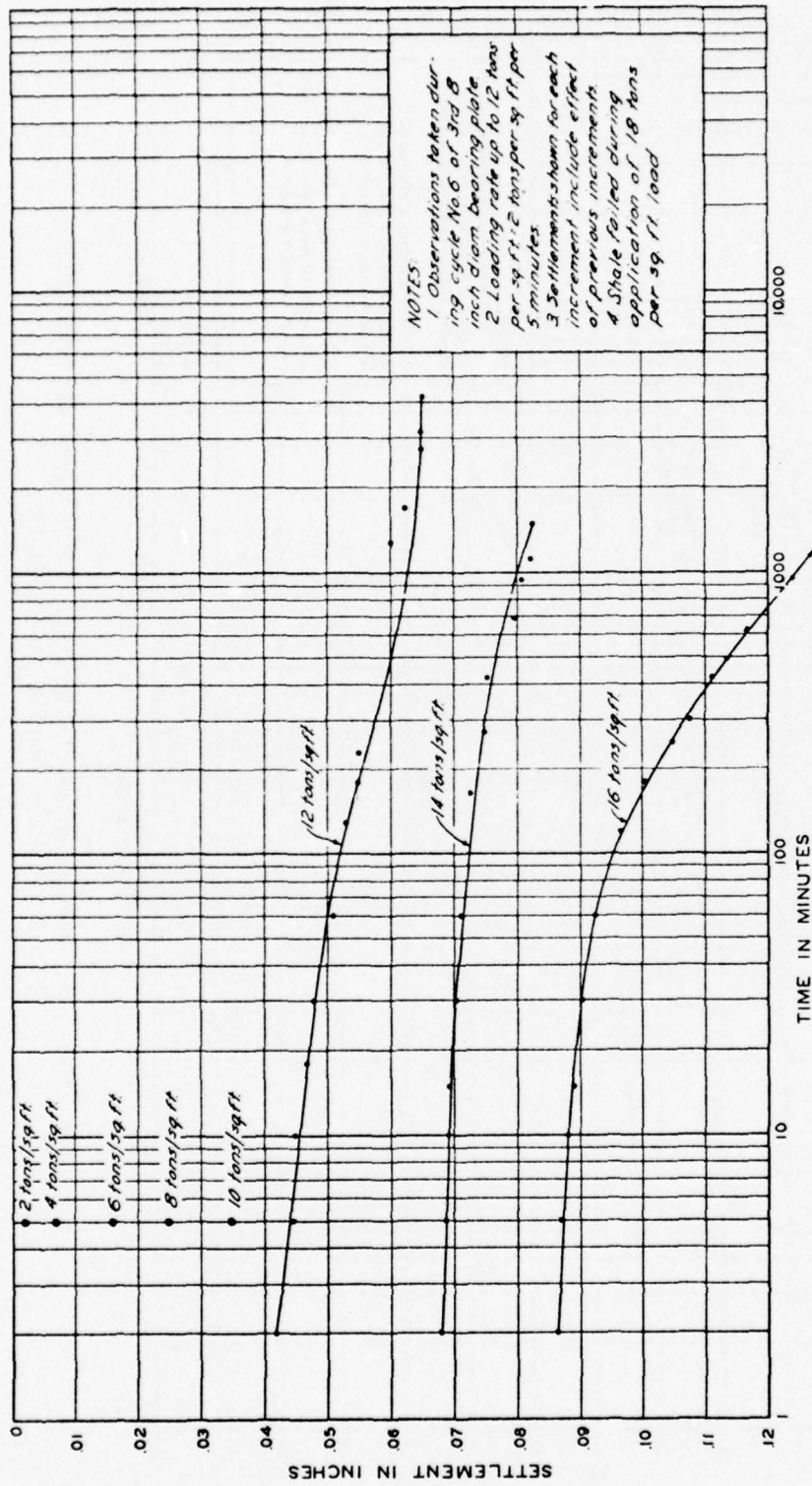


Fig. 27. Time-settlement curves, third 8-in.-diam plate bearing test (cycle 6), Pedro Miguel Test Pit No. 3

days on the larger plate without sign of failure of the shale, it seemed reasonable to conclude that the failure of the 8-in.-diam plate at the 10-tsf load was due to a locally weaker zone and certainly was not representative of the area of the bay in which it was run.

#### Laboratory Bearing Tests

73. Several bearing tests were conducted in the laboratory on Cucaracha shale taken from the test pit. These tests were made with 3-in.-diam bearing plates on smooth surfaces of approximately 10-in. cubes. Both quick and slow tests were made as in the field bearing tests. Results of the laboratory tests are shown in table 4, and load versus settlement curves are shown in fig. 28. The values when compared with the field bearing tests results in tables 2 and 3 are very high. However, it was considered that the fault was not in the laboratory testing but that the differences were due to the fact that the samples were not representative. The only blocks obtained for testing were of material considerably harder than the clay shale tested in the field. For this reason, the laboratory bearing test values were given little consideration in arriving at the recommended bearing capacity for the Cucaracha.

#### Conclusions and Recommendations of November, 1942

74. Based on a study of the results of the tests described above, the following conclusions and recommendations were made:

- a. Field shear tests on the Cucaracha clay shale indicate that the plane of failure is governed by the slickensides.
- b. Field shear tests results also indicate that an angle of internal friction  $\phi$  of 21.5 deg and a cohesion  $c$  of 0.65 tsf should be used in obtaining the shear friction factor for lock walls founded on the type of material tested.
- c. The ultimate bearing capacity of the Cucaracha shale indicated by this series of tests is 15 tsf.

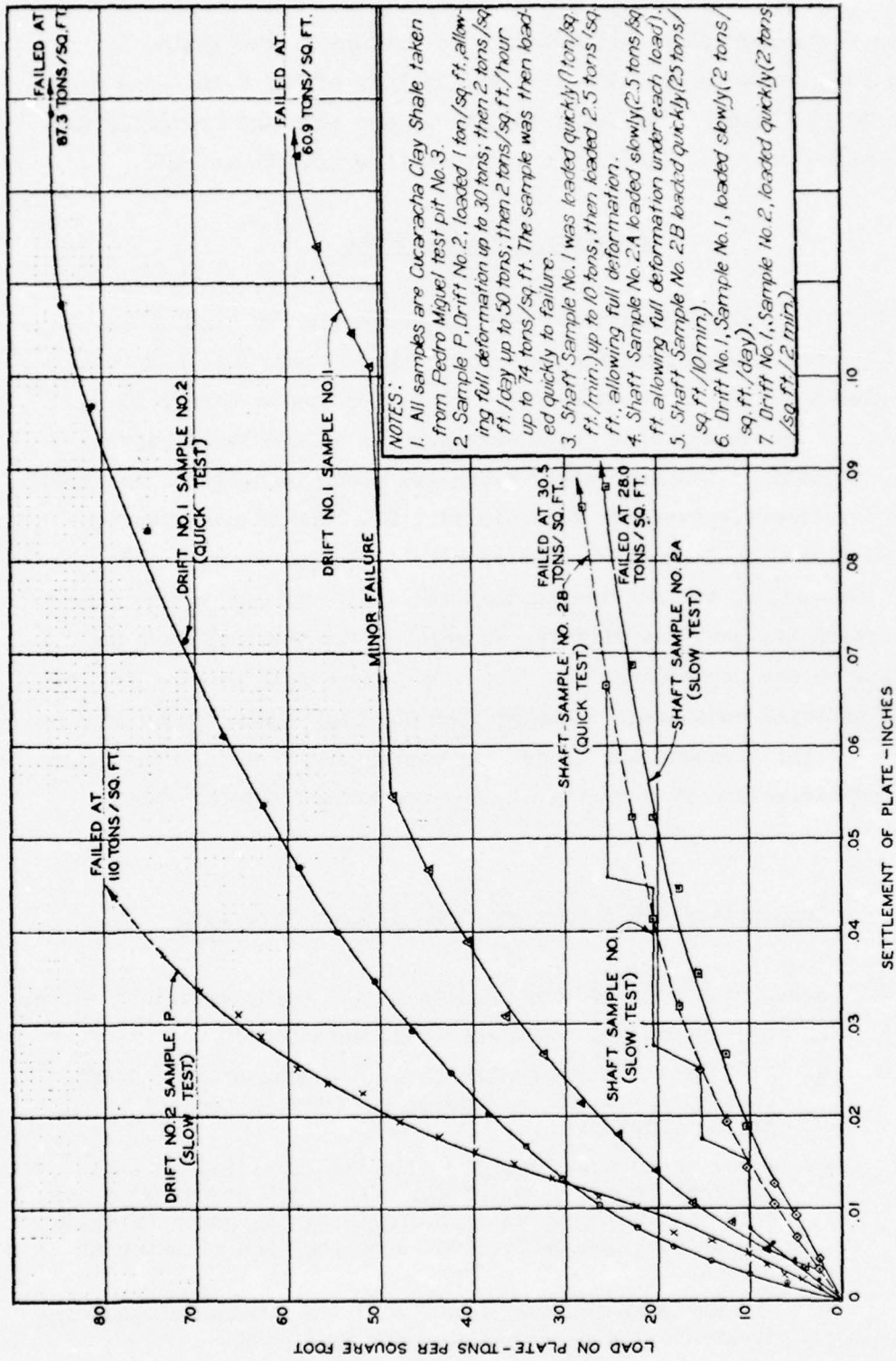


Fig. 28. Load versus settlement, laboratory plate bearing tests, Pedro Miguel Test Pit No. 3

## PART V: LARGE-SCALE CUCARACHA FOUNDATION TEST\*

### General

75. The foundation test was located in the upper approach area of the proposed New Pedro Miguel Lock, approximately 500 ft from Test Pit No. 3 (see fig. 1). Geology of the site is discussed in Part II.

76. After removal of 30 ft of overburden and exposure of the shale, the layer of soft carbonaceous shale was discovered (see photo 1). It was realized that the presence of this soft material would influence the data of the test, and consideration was given to lowering the base of the structure sufficiently to avoid it. However, since black carbonaceous layers of this type are present in the weaker zones of the Cucaracha and no reliable values of its strength had been determined, the decision was made to leave the material in place and the base of the structure was located 4 ft below the top of sound shale at elev 28. Details of the geology of the site are given in Part II.

### Description of structure

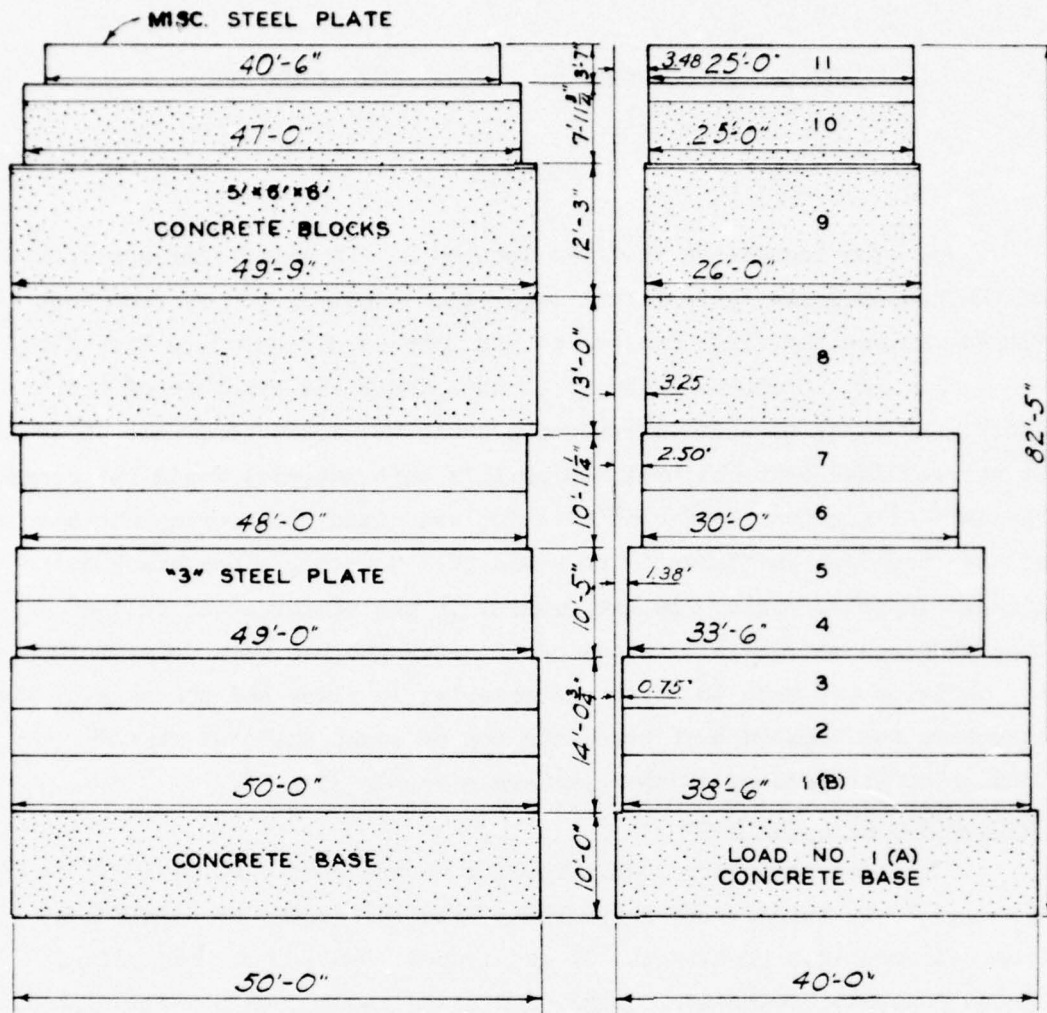
77. The structure consisted of a concrete base 40 by 50 ft in plan and 10 ft thick. The rectangular base was chosen to simulate a lock wall monolith prototype. It was loaded with 3-in. steel plate, 13-ton concrete breakwater blocks, and miscellaneous thin steel plates. Approximately 15,000 tons of 3-in. steel plate, 2500 tons of concrete breakwater blocks, and 1300 tons of miscellaneous thin plate were used to produce the final load (see fig. 29 and photo 2).

### Loading

78. Loads were applied uniformly until a theoretical unit foundation pressure of 4 tsf was obtained. The remainder of the load was applied so that the final load produced a theoretical straight-line distribution of base pressure ranging from 7.5 tsf along the east side to 12.5 tsf along the west side of the structure. Unit loads of 4 and 6 tsf were chosen for specific observation of long-time settlement because

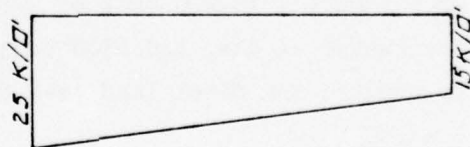
---

\* This material is taken largely from Tracy.<sup>6</sup>



ACCUMULATIVE BASE LOAD  
KIPS PER SQ. FT

LOAD NO.	TOE LOAD	HEEL LOAD	DATE STARTED	DATE COMPLETED
1(A)	1.50	1.50	2-14-44	3-7-44
1(B)	3.91	3.91	3-14-44	3-22-44
2	5.95	5.95	3-29-44	4-3-44
3	8.00	8.00	4-10-44	4-13-44
4	10.56	9.44	5-24-44	5-30-44
5	13.12	10.88	6-5-44	6-9-44
6	15.67	12.04	10-2-44	10-5-44
7	18.21	13.20	10-17-44	10-19-44
8	20.31	13.78	11-3-44	11-7-44
9	21.92	14.23	11-16-44	11-17-44
10	23.51	14.63	11-29-44	12-2-44
11	25.00	15.00	12-12-44	12-16-44



TOTAL BASE LOAD

Fig. 29. Test structure loading diagram

they were the proposed design values for lock structures to be built on the Cucaracha formation in the proposed Pacific Terminal Lake Anchorage Plan. The maximum unit load of 12.5 tsf represented conditions giving a factor of safety greater than 2 for the 6-tsf design loading.

79. A slow rate of loading automatically developed because of the size of the test. The rate was further decreased by scheduling 7-day rest intervals between each loading increment of 2000 tons (1-tsf unit load on the base). At the 4-tsf unit load, a long rest period of 28 days was maintained to determine consolidation characteristics of the formation. A longer rest period of 122 days was maintained at the 6-tsf unit load, and, under the final load, consolidation characteristics were observed for a period of 155 days. The structure was then partially unloaded for the purpose of observing rebound characteristics. When the average unit base pressure equaled 6 tsf, a final rest period was maintained for 300 days before allowing the test pit to fill with water on April 18, 1946.

#### Methods of Obtaining Test Data

##### Settlement of structure

80. Initially, settlement measurements were obtained by precise level methods using a reference bench mark located in agglomerate approximately 350 feet from the site. Scales mounted at the tops of 40-ft pipe standards at the four corners of the structure were used for reading settlement values. Early in the test, it was discovered that the accuracy of precise level measurements was not adequate for the measurement of consolidation characteristics. A static water reference system was installed between the bench mark and the corner pipe standards. Settlement measurements were obtained with vernier hook gages. The system consisted of short hook gage wells (6 in. long) installed in the corner pipe standards and connected by 1-in. galvanized iron water pipe to a hook gage well at the bench mark. The corner pipe standards were maintained full of water to equalize temperature conditions between top and bottom. Corrections were applied to compensate for

change of length of standard. Readings were correct within  $\pm 0.002$  in. Details of the hook gage assembly and the water system are shown in photos 3 and 4.

#### Differential settlement

81. Measurements of the differential of the circumjacent shale surface were obtained from bench marks grouted into the shale surface at intervals of 10, 20, and 40 ft from the sides of the test structures shown in fig. 30. Measurement of the movement of subsurface strata was obtained by using as bench marks steel rods grouted into the bottom of holes 10, 25, 50, and 75 ft deep as shown in fig. 30.

#### Contact and subsurface pressures

82. Pressures at the contact of the base of the structure and the underlying shale were measured with 6 Carlson stress meters grouted into the concrete base as shown in photo 5. Five Carlson stress meters were located at equal intervals across the transverse axis of the structure, and the sixth was located at the northwest corner. The locations are shown in fig. 30, and a cross section of the Carlson cell is shown in fig. 31. Unsuccessful attempts were made to measure substrata pressure by means of rubber cells placed at depths of 10, 25, 50, and 75 ft in 9-in. drill holes located at the center of the structure and 3 ft west of the west edge. The cells were 2-ft-long water-filled rubber cylinders connected to pressure gages at the ground surface. Spaces around and between the cells were filled with sand. Late in the test a piezometer tube was installed to measure pore water pressures, but the results were qualitative rather than quantitative.

### Settlement Data

#### Factors that influence settlement

83. Settlement of the test structure was influenced by the soft plastic carbonaceous layer which dipped below the northwest corner. The settlement at this corner was approximately 3-1/2 times the settlement observed at the southeast corner, which is underlain to a great depth with typical gray-green Cucaracha. The base of the structure

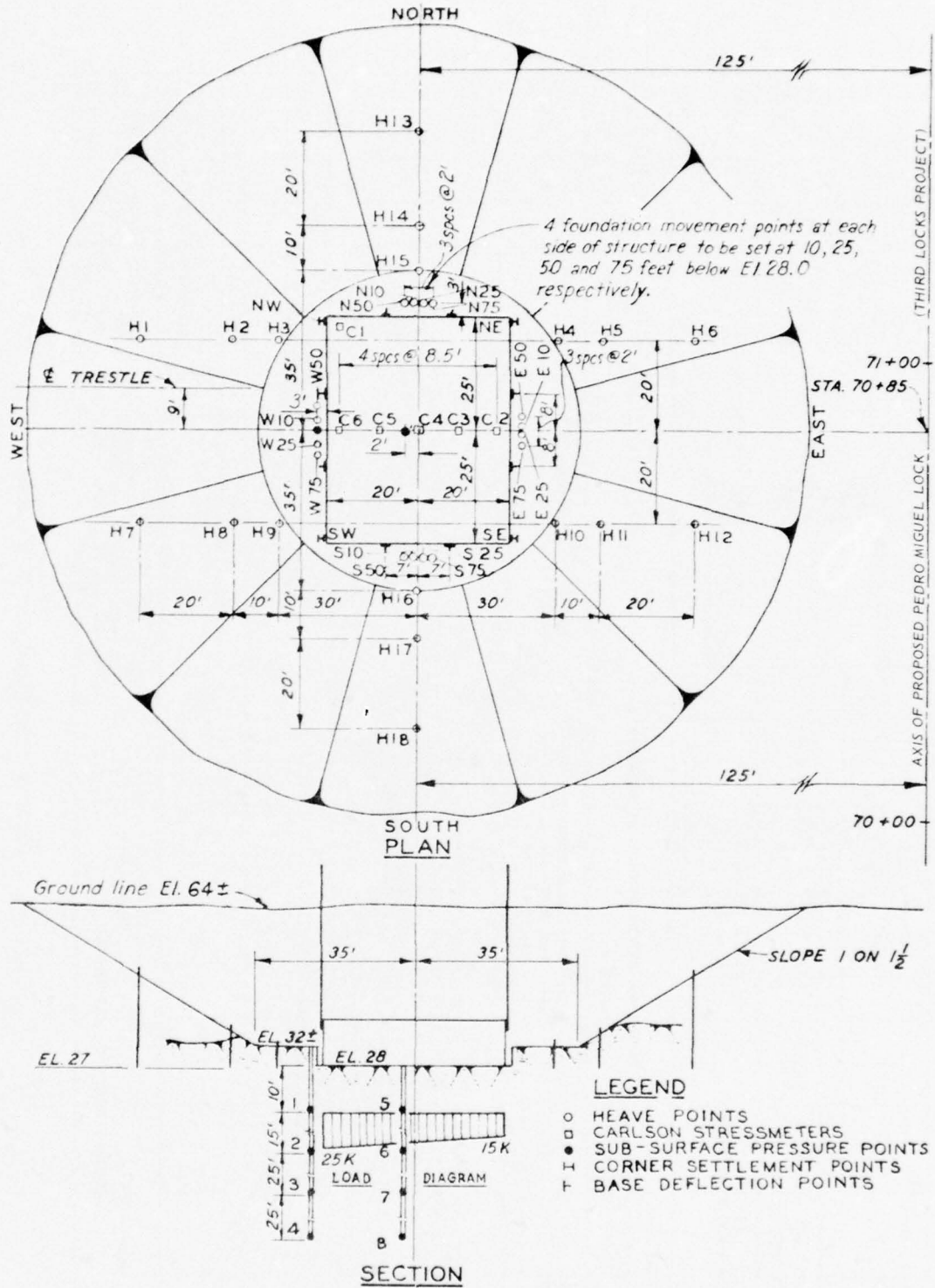


Fig. 30. Foundation plan and cross section with instrumentation locations

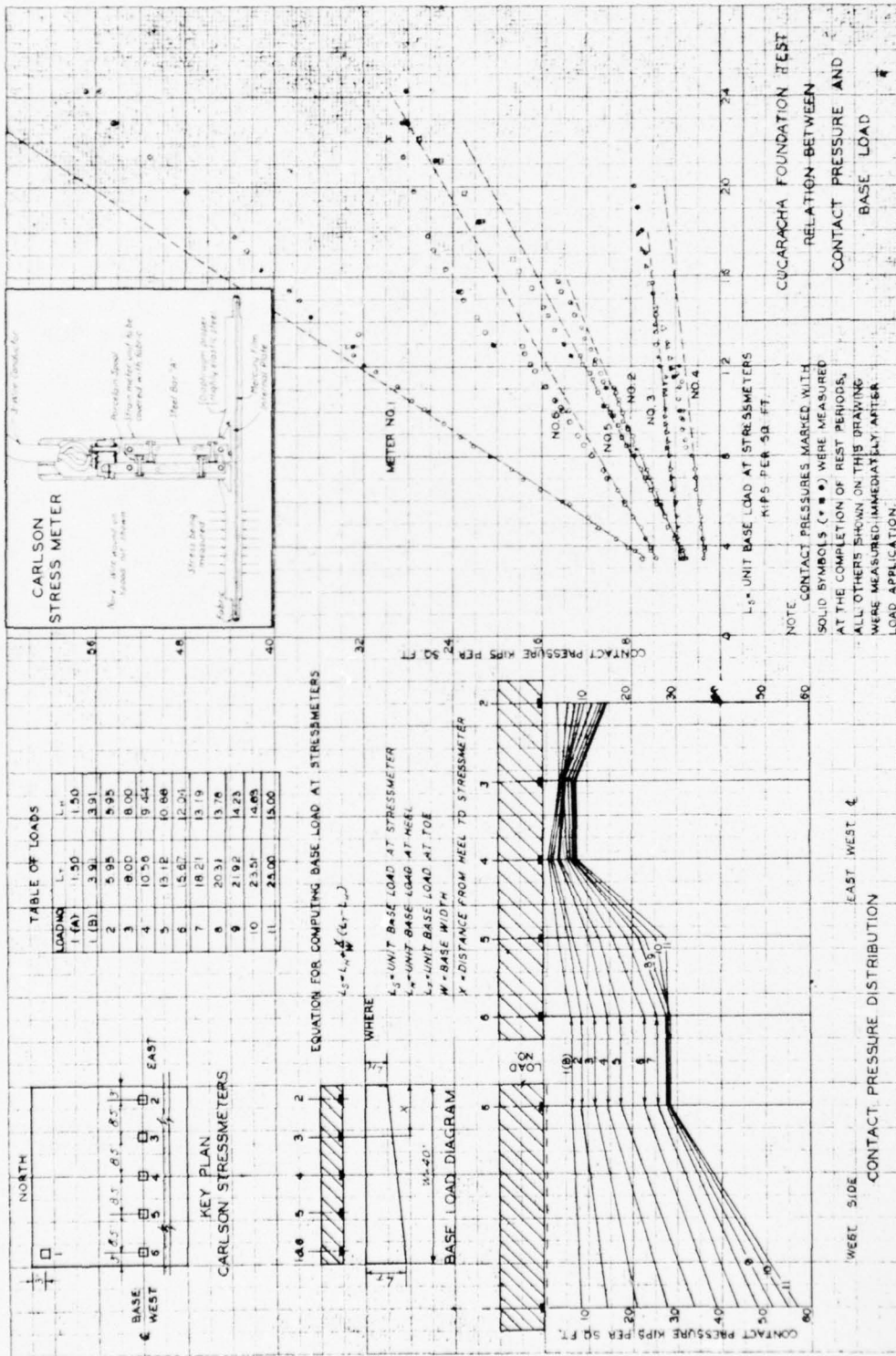


Fig. 31. Contact pressures and base loads

appeared to act at all times like a rigid plate, although late in the test a crack developed approximately at the transverse axis as shown in photos 6 and 7. There is some indication that a foundation failure may have occurred when the average load on the west side of the structure was approximately 18 ksf. Perimeter heave points adjacent to the west side of the structure show a rise after this time (November 1944, see fig. 43.) The evidence is partly disproved by the continuation of the rise of pressure indicated by the Carlson cells and by the rebound which occurred when the structure was unloaded.

#### Test results

84. The summary of the settlement data that follows is based on the average settlement of all four corners of the test structure. An arbitrary distinction was made between "immediate" settlement, that occurring at the time of load application, and "intermediate" settlement, that occurring during rest periods. Significant data of the test are summarized in tables 5 and 6. Settlement versus time and load versus time curves for the test are shown in figs. 32-34. A semilog plot of settlement versus time for the load 5 rest period is shown in fig. 35. The uplift test during this rest period with the omission of 13 days of the uplift test from the curve made it difficult to decide if the settlement was complete at the end of the 113-day rest period. Semilog plots of settlement versus time for the rest periods of loads 6-11 are shown in fig. 35. During rest period 11, drilling operations at the site caused an abnormal settlement rate to occur between the 19th and 41st days of the rest period. This abnormal settlement has been omitted on the plot shown in fig. 36.

85. When the structure was unloaded to an average base load of 12 ksf, a 300-day rest period followed to observe long-time rebound characteristics. Observed rebound data have been plotted on a semilog scale in fig. 36. The rebound was only 32.2 percent of the observed settlement under the same load. The test loading during rebound was held constant for a period approximately twice as long as the earlier settlement rest period.

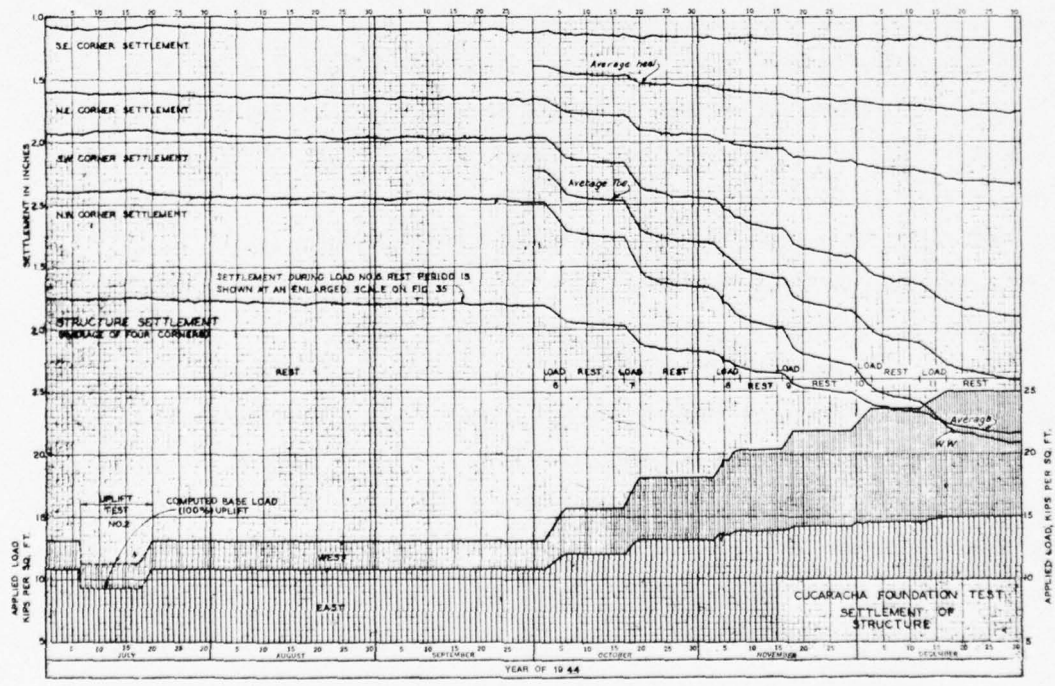
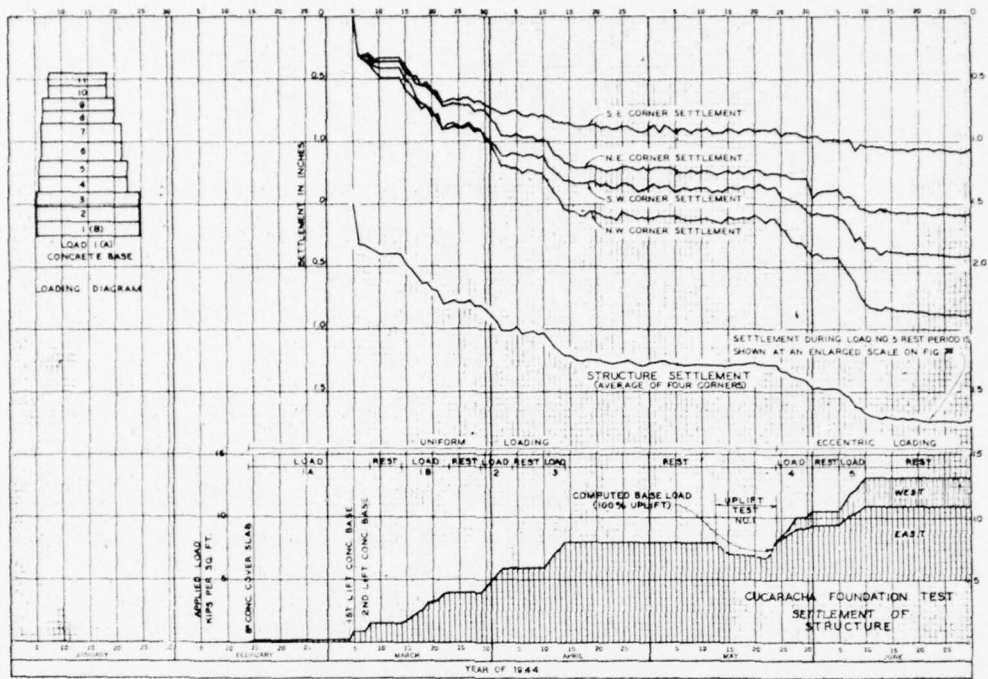


Fig. 32. Settlement of test structure during 1944

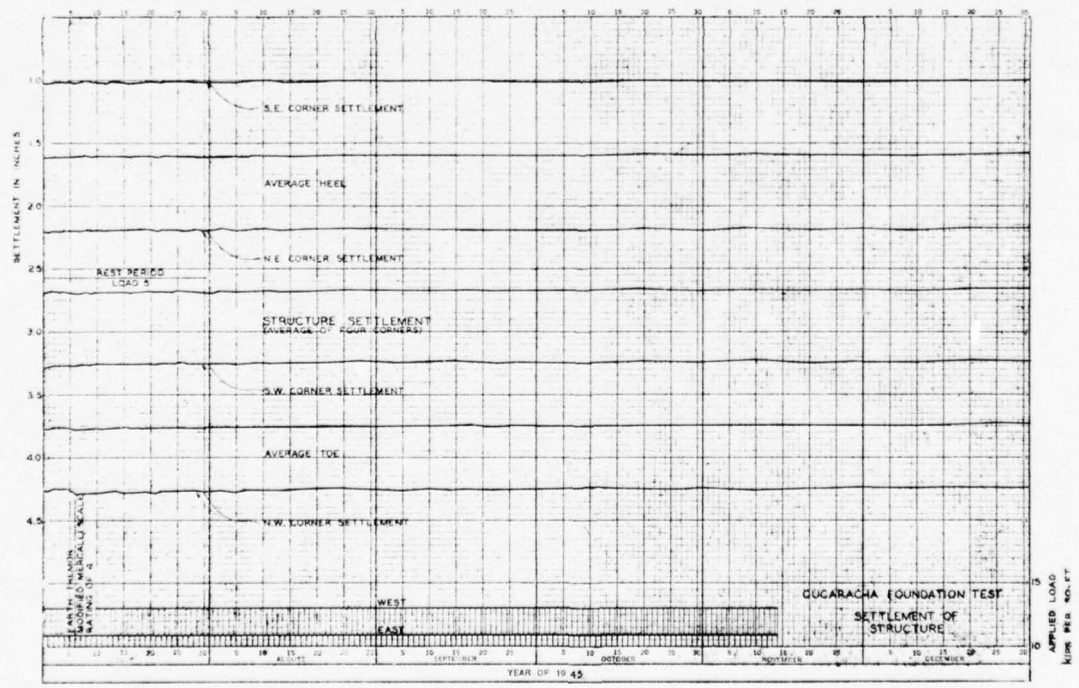
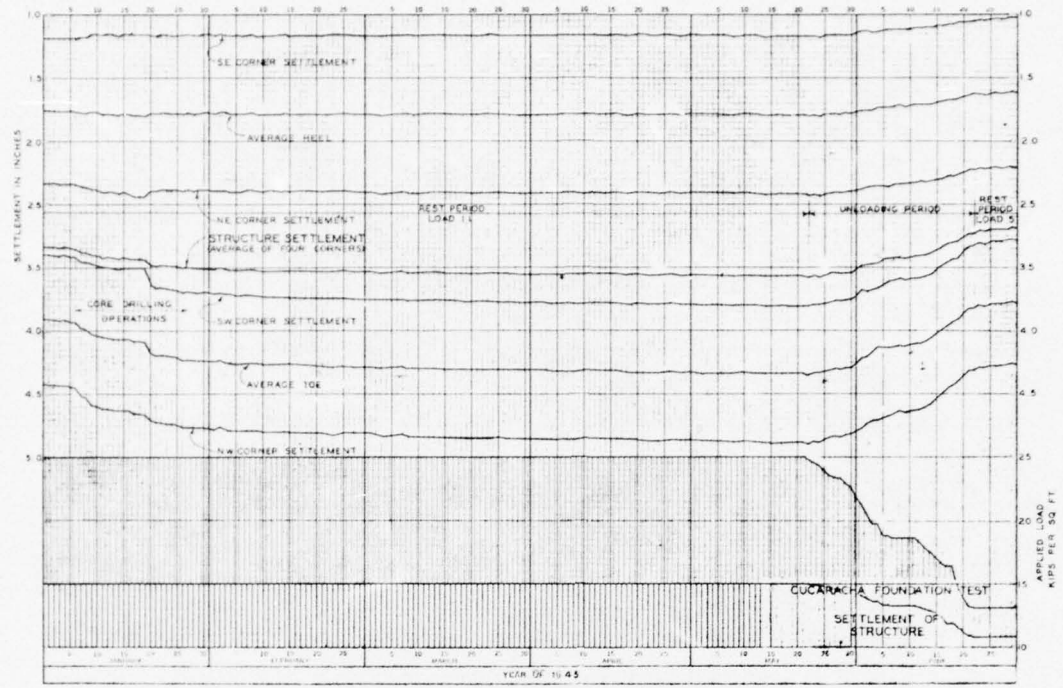


Fig. 33. Settlement of test structure during 1945

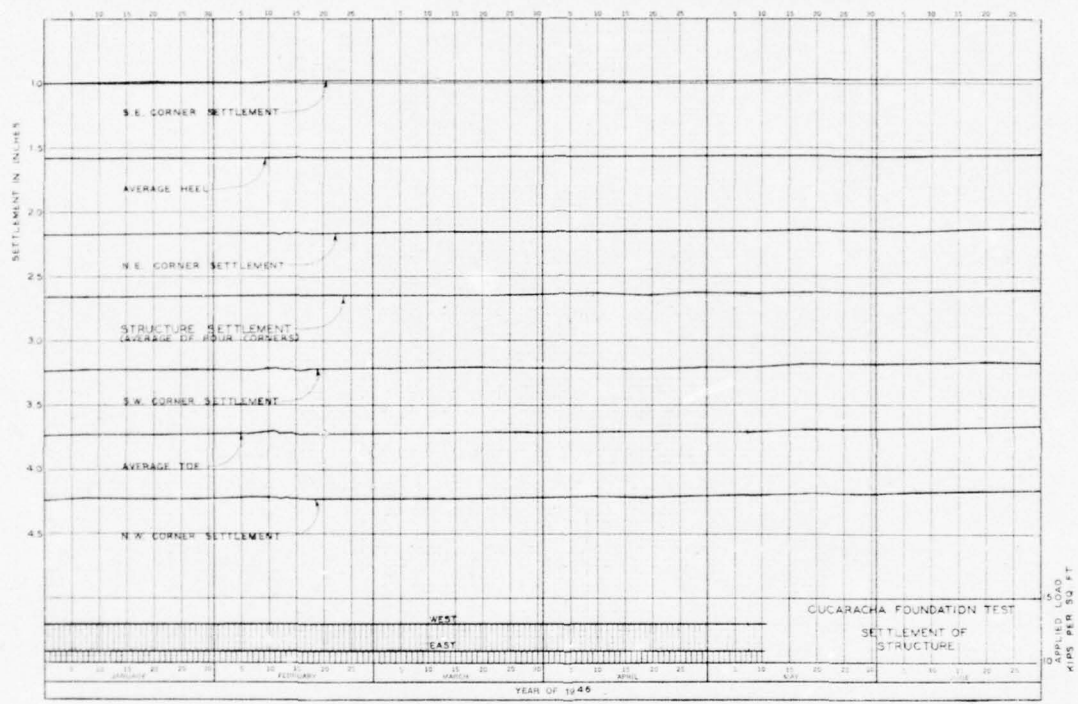


Fig. 34. Settlement of test structure, January-May 1946

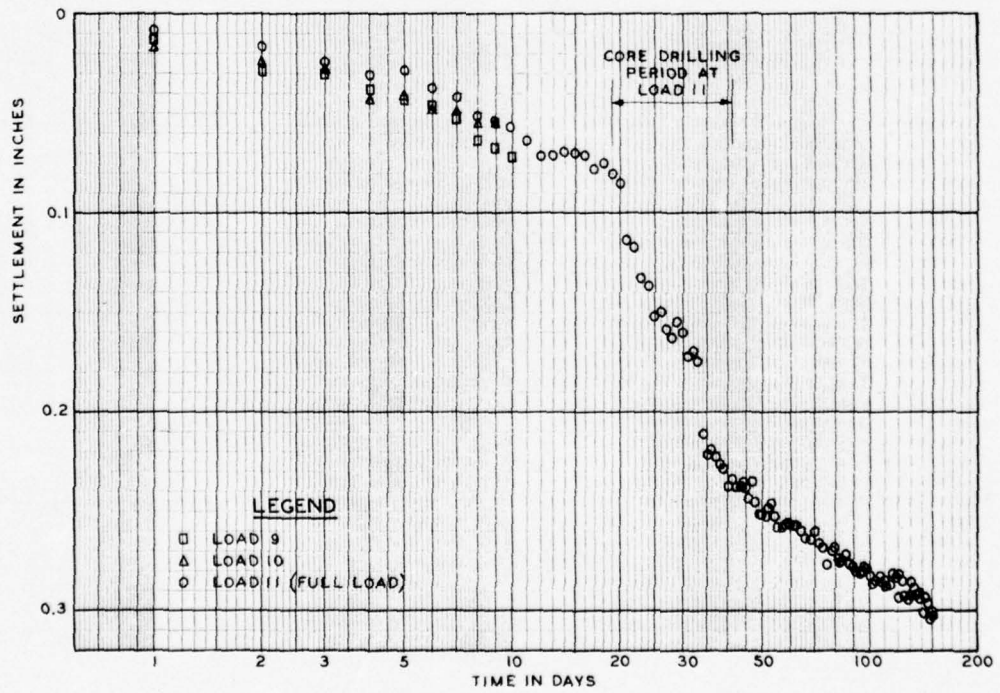
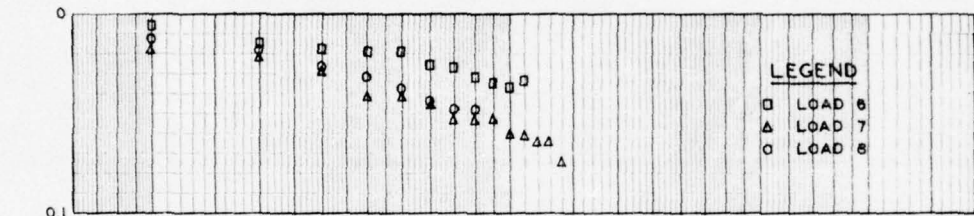
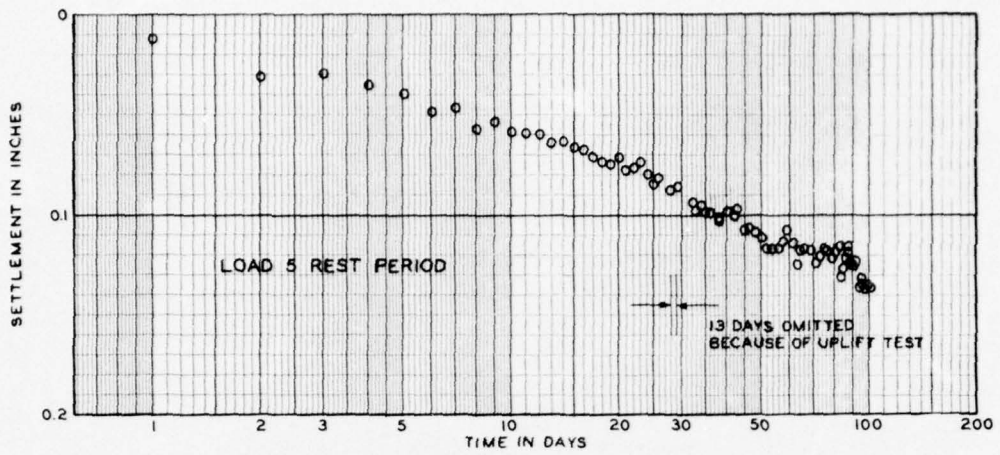


Fig. 35. Time-settlement curves for loads 5-11

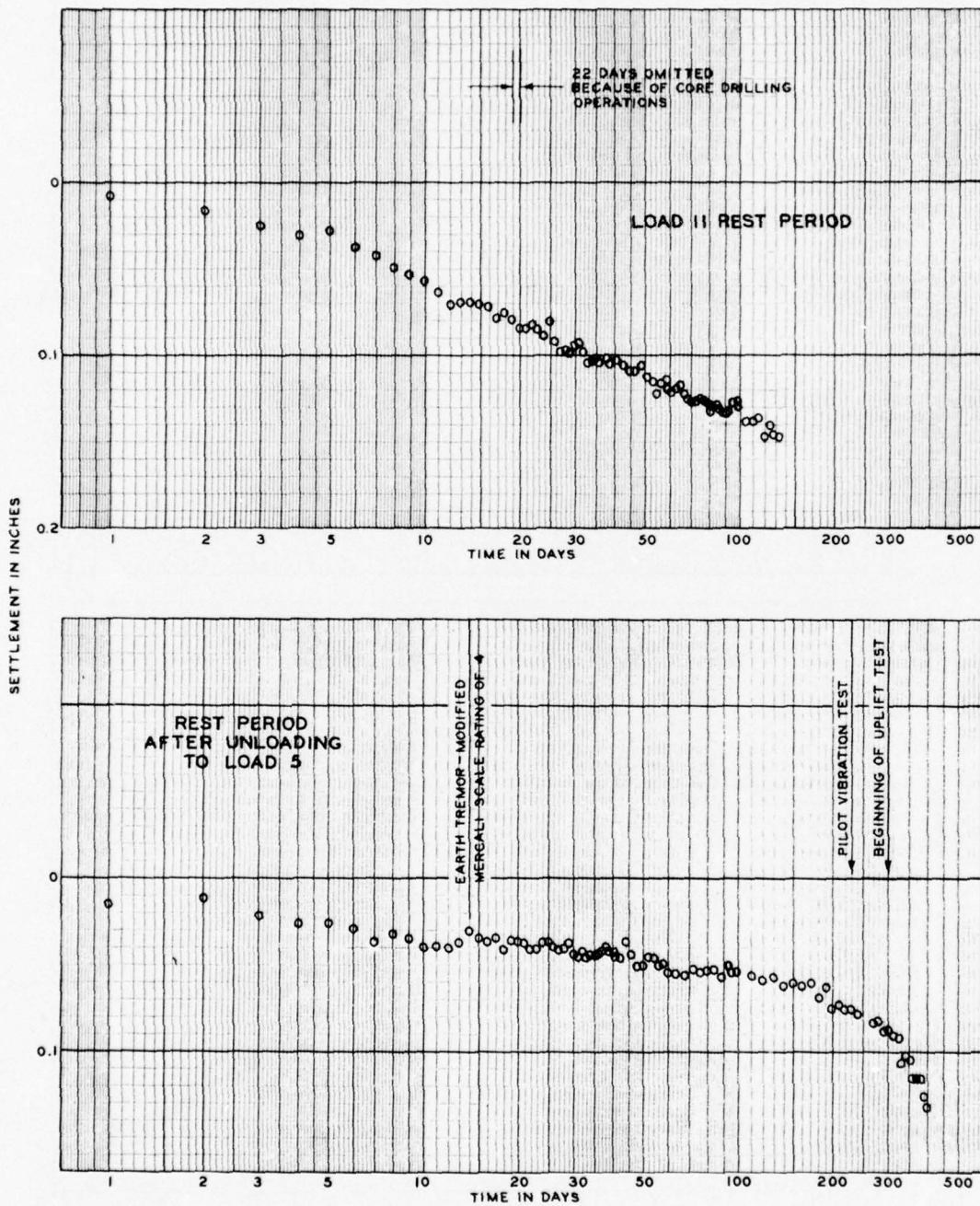


Fig. 36. Time-settlement curve for load 11 rest period and rebound after unloading to load 5

## Uplift Tests

### First uplift test

86. The structure was subjected to two uplift tests. The first test was during the period May 13-23, 1944, when the average base load upon the structure was 8 ksf. The test pit was flooded to a height of 24.4 ft above the base of the structure. Due to a shortage of water, 7 days were required for flooding the pit, and observations of structure rebound and contact pressure variation were made under varying load conditions. The relation between pool stage and substructure water pressures was observed by recording float well gages, the latter being connected to the drill hole for the subsurface pressure cells. During the test, the total movement of the structure was within the range of error of the precise level measurements being used at the time. Each Carlson stress meter registered a rise or fall of pressure when its reading prior to the test was less or greater, respectively, than the average unit load. Stress meter pressures under conditions of increased uplift are a combination of two separate effects. Total uplift over the base of the structure would cause a reduction of pressure on a stress meter equivalent to unloading the structure by a uniformly distributed load of equal amount. The stress meter indication of this reduction would, however, be modified by an increase of pressure caused by water which is locally in direct contact with the meter. The pressures observed during the uplift test are indicated in fig. 37.

### Second uplift test

87. The second uplift test was performed during the period July 6-20, 1944, when the average unit load upon the base of structure was 12 ksf. The pit was flooded to a point 37.1 ft above the base of structure by heavy rains on July 6. During this period, the structure gradually rose 0.015 in. The test was maintained until no further rise had been noted for a period of 4 days. Carlson stress meters indicated a change of contact pressure distribution during the test as shown in table 7.

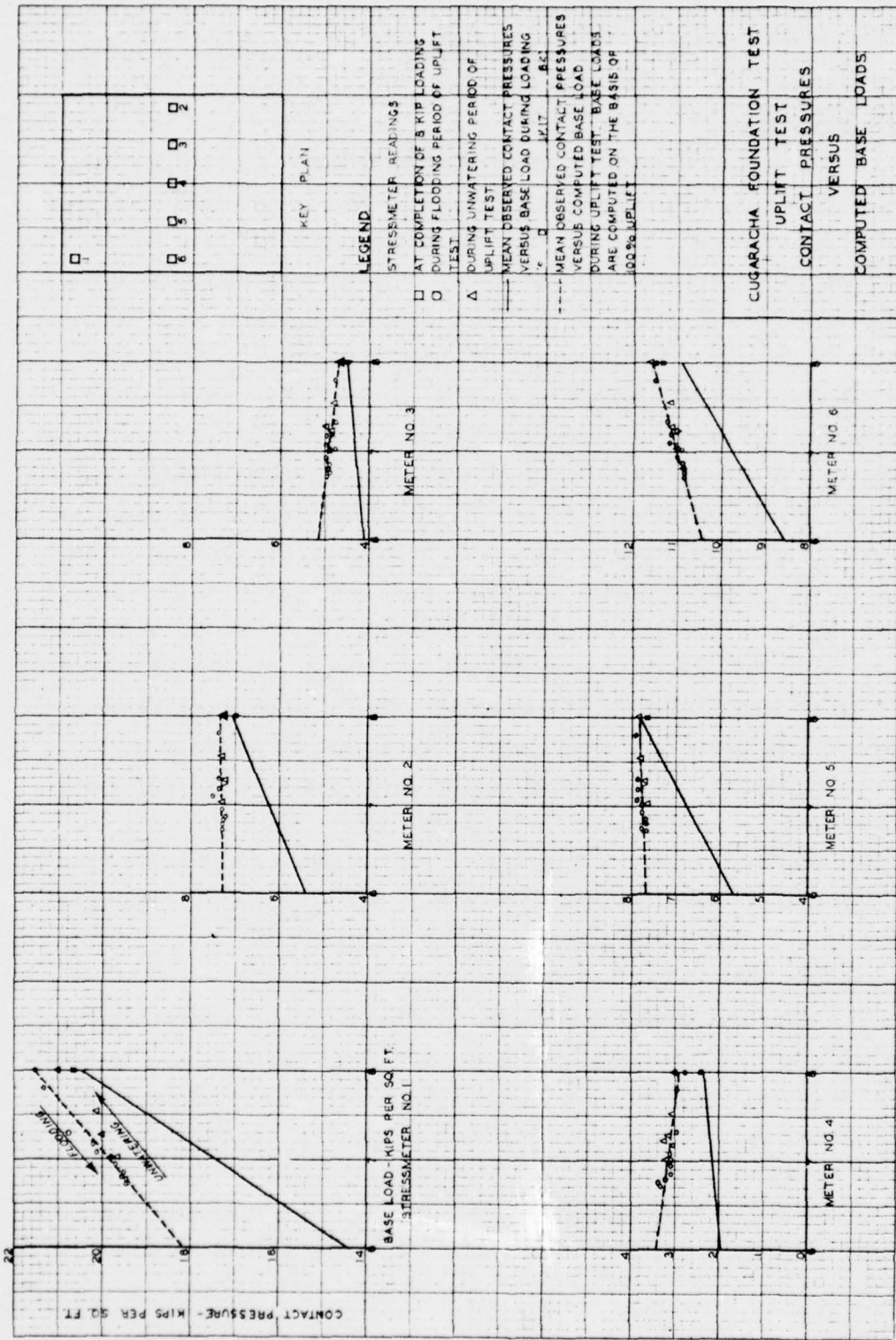


Fig. 37. Contact pressures versus computed base loads

## Base Pressures

### Carlson stress meters

88. Six Carlson stress meters were installed in the concrete base to measure the pressure at the shale contact (see photo 5). The construction of the Carlson stress meter is shown in fig. 31. It consisted of two plates approximately 7 in. in diameter separated by a thin mercury film which served to indicate very small plate deflections. The deflection was magnified by means of a small internal diaphragm to which was connected the moving support of two electrical resistance coils. The coils were wound about porcelain spools so that as one coil was stretched, the other separated by plate movement. This feature counteracted the effects which would otherwise be introduced by temperature variation at the stress meter. Pressure measurements are obtained by measuring the resistance to a small flow of current through the coils by means of a Wheatstone bridge.

### Test results

89. Very consistent results were obtained using the stress meters as illustrated by the curves in fig. 31. Particular attention is directed to meter 1 which gave consistent results to 32 ksf, more than three times its rated capacity of 10 ksf. Above 32 ksf, divergence from the straight-line variation with applied load is noted. It is believed that this is an effect produced by edge restraint of the small diaphragm when deflected by high loads since edge resistance will reduce the diaphragm deflection to a marked extent when the deflection is greater than one-half the thickness of the plate. As shown in fig. 31, the pressures on cells 3 and 4 became practically identical as the pressure increased above 8 ksf. This feature was attributed to a squeeze of the plastic layer of the foundation material which crosses the basal contact close to these cells. In figs. 38-40 are shown the curves of contact pressure versus time for the six cells. The contact pressure distribution for the various loadings is shown in fig 31.

90. It was recognized that the analysis of the pressure distribution beneath the structure would be difficult with only 6 cell

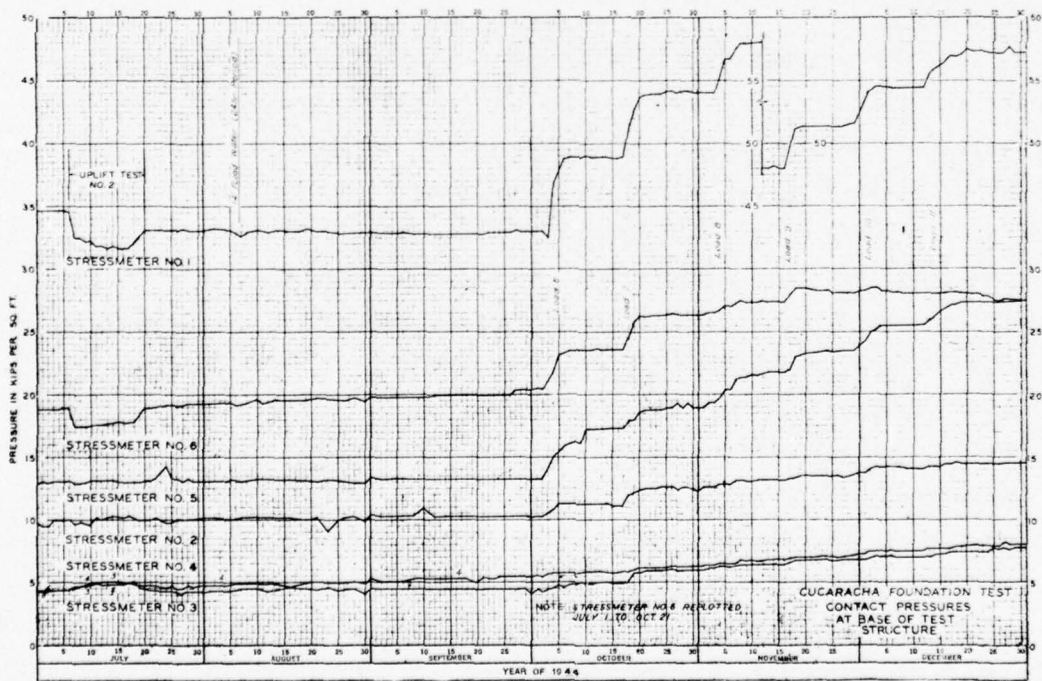
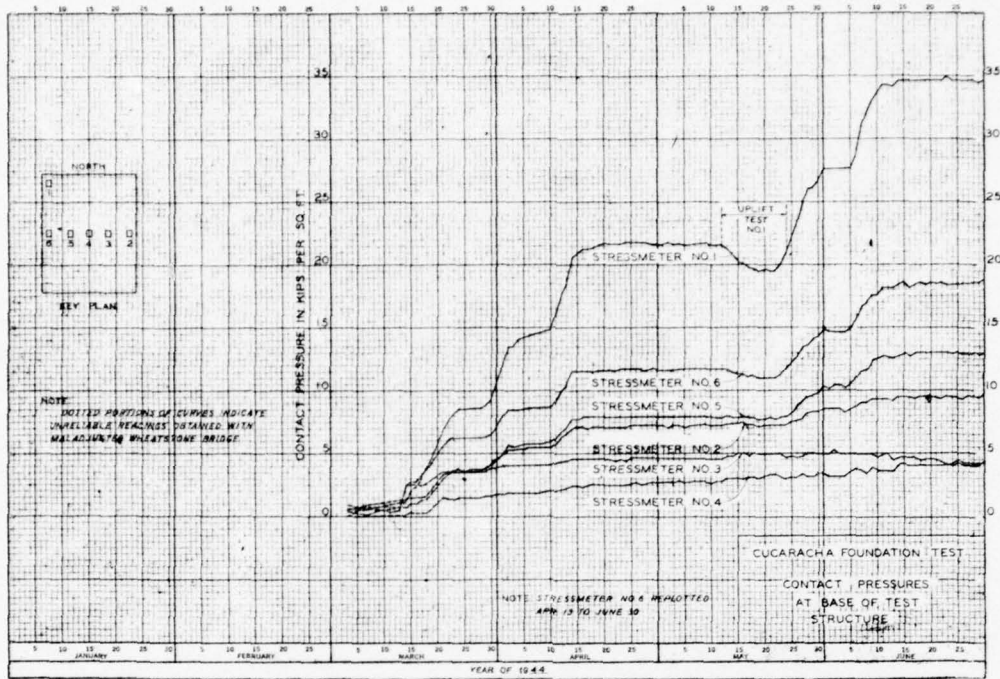


Fig. 38. Contact pressures at base of test structure during 1944

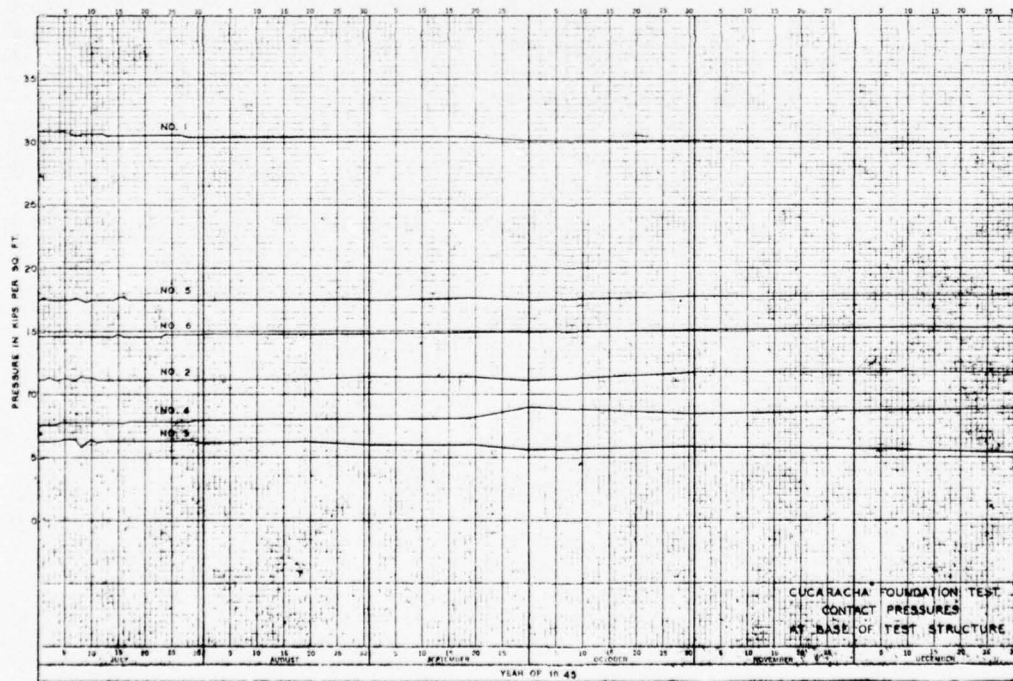
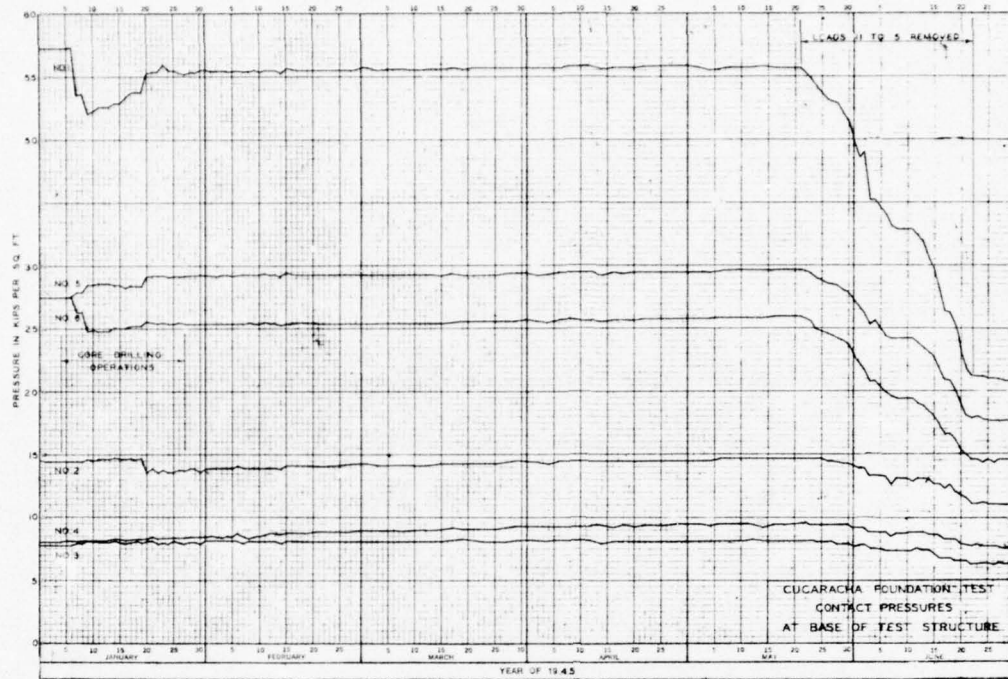


Fig. 39. Contact pressures at base of test structure during 1945

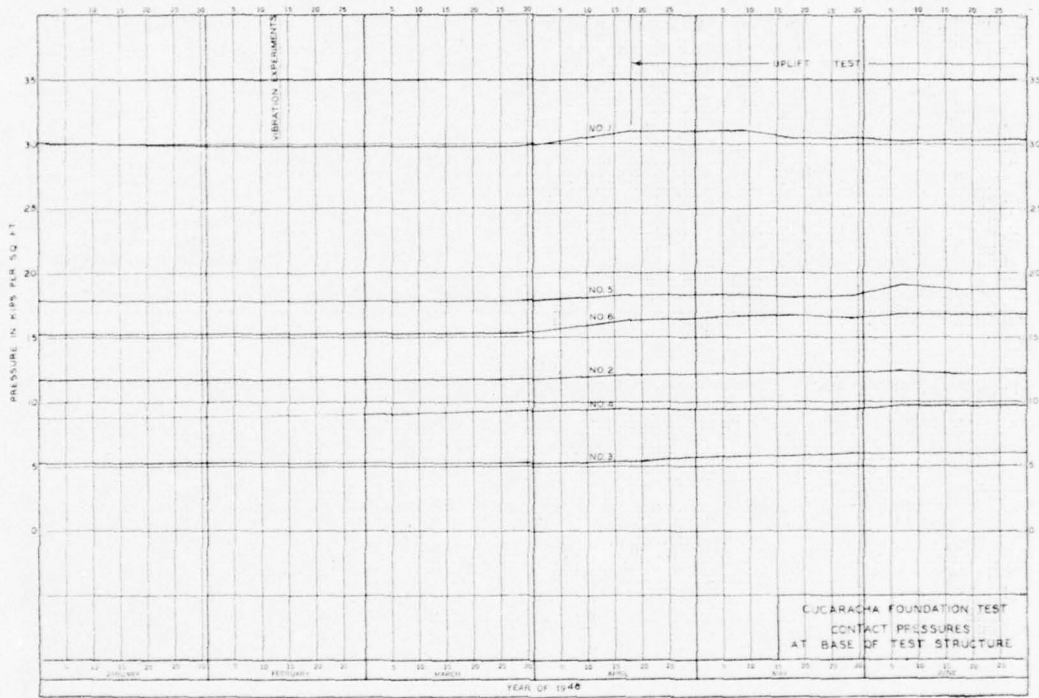


Fig. 40. Contact pressures at base of test structure, January-June 1946

locations. The six cells had been available from Third Locks Project equipment and were located in the structure to determine the transverse pressure distribution only. In a foundation material such as the Cucaracha, adequate information on pressure distribution could only have been obtained by a large number of cells distributed over the whole area. Pressure distribution across the transverse axis of the structure is indicated in fig. 31. It shows that edge pressures were considerably in excess of center pressures and the highest pressures were located at the corner.

#### Subsurface Settlement

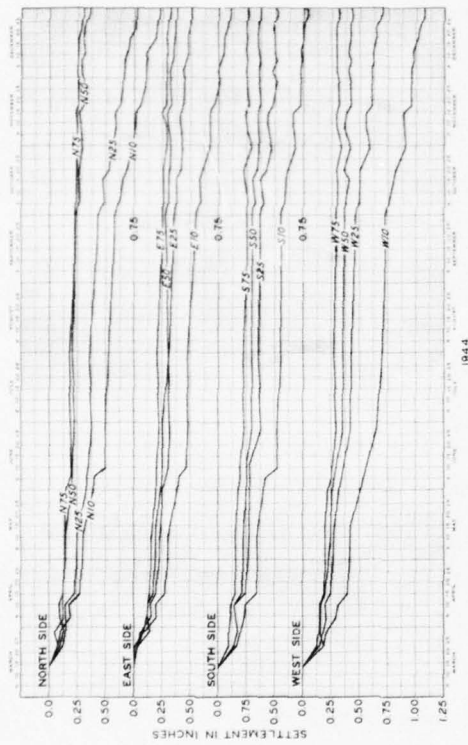
91. Measurement of the movement of subsurface strata was obtained by using steel rods grouted into the bottom of holes 10, 25, 50, and 75 ft deep. The locations of the holes are shown in fig. 30. Results of the measurements are shown as curves of settlement versus time for the various rods in fig. 41. These results are significant in that they indicate that, at a depth about equal to 3 times the width of the structure, the substrata settlement is so small as to be not measurable.

#### Deflection of Structure Base

92. A large crack developed in the base of the structure late in October 1944. Records of its movements were obtained beginning on November 3, 1944, and are indicated in fig. 42. There was no evidence that its occurrence influenced the tests since all data continued to conform to previous observations.

#### Settlement of Circumjacent Shale Surface

93. Settlement of the circumjacent shale surface was very small. Curves of settlement versus time are shown in fig. 43. It is significant that the settlement of the shale surface was less at the north end,



NOTE LETTER AND NUMERAL DESIGNATIONS ON CURVES IDENTIFY SETTLEMENT POINTS. SETTLEMENT POINTS DESIGNATED BY LETTERS WERE LOCATED AT DEPTHS OF 10, 25, 50 AND 75 FEET BELOW FOUNDATION GRADE. NEAR POINTS DESIGNATED BY LETTERS AND NUMERALS N, E, S, AND W IDENTIFY THE STRUCTURE SIDE, AND 10, 25, 50, AND 75 THE DEPTH BELOW FOUNDATION GRADE.

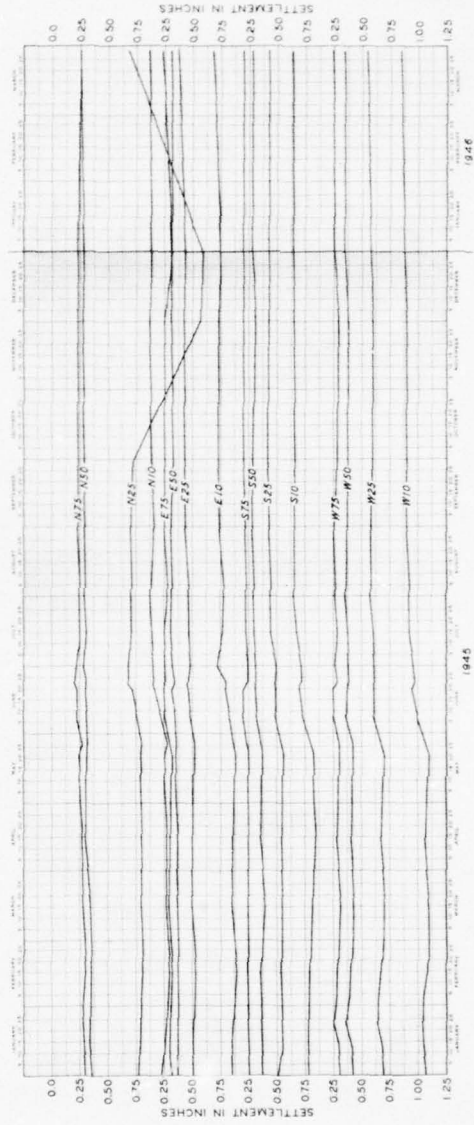


Fig. 41. Settlement of subsurface points, test structure foundation, 1944-1946



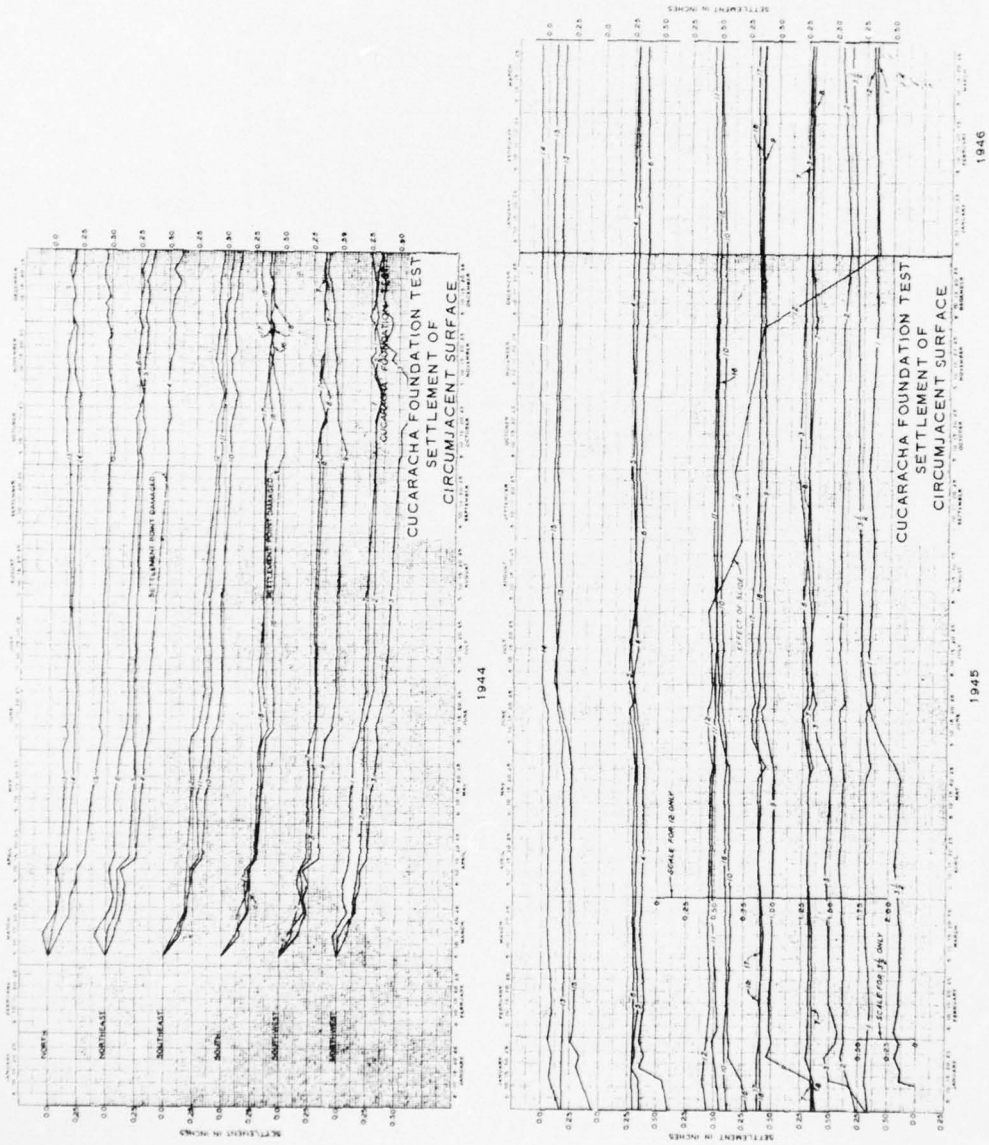


Fig. 43. Settlement of ground surface adjacent to test structure, 1944-1946

where settlement of the structure was greater, than at the south end. This feature can be attributed to the weaker character of the shale at the north end: it compresses more under load but is less capable of transferring stress to areas outside of the loaded area.

PART VI: RECOMMENDATIONS OF CONSULTANTS

94. A conference was held in Washington, D. C., on February 27 and 28, 1945, with Dr. A. Casagrande, and Messrs. L. F. Harza, Joel D. Justin, and W. H. McAlpine, consultants, and Messrs. T. A. Middlebrooks, E. B. Burwell, and G. D. Smith of the Office, Chief of Engineers. As a result of this conference, it was decided that safe upper lock chambers could be built at the upper end of the existing Miraflores Locks to provide triple-flight locks and an anchorage harbor at the summit level, provided certain limitations in the design and construction as described below would be followed:

- a. It was recommended that the foundation loading on the Cucaracha material for the normal operating conditions should not exceed about 6 tsf. This load could be exceeded during the construction period or when a lock chamber was unwatered for repairs.
- b. The fact that the maximum foundation loading during construction would be considerably greater than the loads which would be effective during normal operation was not considered objectionable. These heavier loads would actually be beneficial because they would result in a major portion of the consolidation occurring during the construction period.
- c. In order to further expedite consolidation during construction, it was recommended that sand drains be installed in the foundation with suitable outlets. For this purpose, all exploratory holes would be used by backfilling with clean sand.
- d. There was concurrence with the tentative design that no dependence should be placed on the sliding resistance of the Cucaracha formation. In general, the lengths of the monoliths should not exceed 35 feet, except where greater length might be needed for the gate blocks.
- e. Excavation should be carried deep enough to remove any layers of weak carbonaceous shale or any gouge material in the fault zone revealed by more extensive borings to be thick enough or near enough to the theoretical foundation line to weaken the supporting value of the foundation.

PART VII: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

95. The results of the excellent tests in Pedro Miguel Test Pit No. 3 have made a major contribution to the knowledge of the behavior under loading of those phases of the Cucaracha formation that were tested. Most remarkable of all the data are the 8- and 16-in.-diam plate bearing tests. Under rapid loading, the shale acted as though it were elastic, homogeneous, and isotropic. When such an ideal material is loaded on its surface, the settlement can be expressed by the following equation:

$$S = C \frac{p}{E} A^{1/2} (1 - u^2) \quad (1)$$

where

- S = settlement of loaded area
- C = coefficient (a function of shape and stiffness of the loaded area)
- p = unit pressure
- E = modulus of elasticity of the material
- A = loaded area
- u = Poisson's ratio of the material

Equation 1 can be expressed in logarithms as

$$\log p = \log \frac{S}{A^{1/2}} + \log \frac{E}{C(1 - u^2)}$$

This is the equation of a straight line; in addition, the slope equals unity which makes it a 45-deg line. The results of the 8- and 16-in.-diam plate bearing tests when plotted on log-log paper show that the line is very close to a 45-deg line. The results of the 40-in.-diam plate bearing tests show a line inclined slightly greater than 45 deg. The conclusion is that the shale tested acted elastically when loaded rapidly and only small volumes were involved. The modulus of elasticity

can therefore be determined from these tests. The shape of the time-settlement curves suggests that the Terzaghi consolidation theory or Biot consolidation theory might be used to determine settlement under constant loading over a period of time. However, creep or secondary compression enters the problem, and this settlement could be equal to the settlement due to consolidation over a long period of time. The question then arises as to whether or not the 8-, 16-, and 40-in.-diam bearing tests can be used for predicting the settlement of large structures. For this reason, the large-scale test was performed. Conclusions on the large-scale test are given in the following paragraphs.

96. It is unfortunate that the large-scale test was performed in an area where the layer of soft black carbonaceous clay shale crossed the site diagonally, dipping below the northwest corner. This fact increases the difficulty of analysis. If the  $S/A^{1/2}$  ratio is plotted against the unit load  $p$  on log-log scale for the uniform loads up to 8 ksf the inclination of the line is much greater than 45 deg. This would indicate that the shale is not acting as an ideal elastic material as would be expected because of the dipping carbonaceous shale layer. However, there are other factors that could influence the inclination of the log-log plot. One of these is that each increment of loading was applied over a considerable period of time and therefore the settlement recorded at the end of the loading may include settlement due to consolidation and creep.

97. The total settlement for a long period of time for a loaded rigid rectangular area on a semi-infinite saturated porous material can be easily calculated if certain conditions are fulfilled. The conditions are that the material has mechanical properties that can be considered as an idealization of the properties of actual shales and that the load is applied instantly. Under these conditions and if the total settlement is used, the log-log plot mentioned in paragraph 95 above would be a straight line and the effective modulus of elasticity  $\bar{E}$  and effective Poisson's ratio  $\bar{u}$  could be determined. Since the shale is not an ideal material, the  $\bar{E}$  and  $\bar{u}$  would be the average for the materials under the loaded area. As the loads in the bearing

test were applied over considerable time, and rest periods in most cases allowed only partial consolidation settlement to take place, analysis would be difficult but could be made using analytical methods developed since the test was completed in 1946.

#### Recommendations

98. It is recommended that the results of the Cucaracha Foundation Test and the tests in Pedro Miguel Pit No. 3 not be used for the design of large structures built on Cucaracha clay shale until a thorough investigation of the test results is made using the sophisticated methods of analysis available at the present time. This recommendation is based on indications that the actual settlements of large structures on Cucaracha clay shale over a long period of time could be materially different than those expected in 1946.

LITERATURE CITED

1. Flach, A. C., Jr., "Report on the Foundation Materials of the New Miraflores and Pedro Miguel Locks," Nov 1942, Special Engineering Division, The Panama Canal, Canal Zone.
2. Woodring, W. P., "Geology and Paleontology of Canal Zone and Adjoining Parts of Panama; Description of Tertiary Mollusk (Gastropods: eulimidae, marginellidae to helminthologyptidae)," Professional Paper 306-D, Chapter D, 1970, U. S. Geological Survey, Department of the Interior, Washington, D. C.
3. Thompson, T. F., "Exploratory Borings at Base of Cucaracha Foundation Test Structure," Memorandum to Supervising Engineer, 9 Feb 1945, Special Engineering Division, The Panama Canal, Canal Zone.
4. Vincent, C. F., "Report of Tests in Pedro Miguel Test Pit No. 3," Nov 1942, Special Engineering Division, The Panama Canal, Canal Zone.
5. Thompson, T. F., "Origin, Nature, and Engineering Significance of the Slickensides in the Cucaracha Clay Shales," Interoceanic Canal Studies Memorandum 245, 25 Apr 1947, The Panama Canal, Canal Zone.
6. Tracy, R. L., "The Cucaracha Foundation Test Summary Report," Jul 1946, Special Engineering Division, The Panama Canal, Canal Zone.

Table 1  
Settlement Data for Concrete Blocks for Shear Tests

Block No.	Normal Stress tsf	Settlement Under Normal Stress, in.
1	7.6	--
2	7.4	0.145
3	4.0	0.186
4	7.4	0.218
5	4.0	0.128
6	7.4	0.163

Table 2  
Field Bearing Test Results

Diameter of Plate in.	Bay No.	No. of Cycles	Ultimate Bearing Strength Value tsf	Loading Rate*
40	6	5	**	
16	7	4	30	Quick
8 (test 1)	8	5	38	Quick
8 (test 2)	6	5	10+	Slow
8 (test 3)	5 to 6	6	15	Slow

\* See paragraphs 68-70.

\*\* Shale did not fail.

+ Test result not considered valid (see paragraph 72).

Table 3  
Modulus of Elasticity E from Field Bearing Tests

<u>Diameter of Plate in.</u>	<u>Deformation 1st Cycle in.</u>	<u>Deformation 3rd Cycle in.</u>	<u>E for 1st Cycle psi</u>	<u>E for 3rd Cycle psi</u>
40	0.234	0.134	14,000	24,400
16	0.122	0.051	10,700	25,700
8 (test 1)	0.056	0.028	11,700	23,400
8 (test 2)	0.140	0.048	4,670	13,600
8 (test 3)	0.053	0.027	12,300	24,200

Table 4  
Laboratory Bearing Test Results

<u>Sample</u>	<u>Type of Test</u>	<u>Ultimate Bearing Capacity tsf</u>	<u>Modulus of Elasticity psi</u>
Shaft Sample No. 1	Slow	25.5	23,000
Shaft Sample No. 2A	Slow	28.0	14,000
Shaft Sample No. 2B	Quick	30.5	12,500
Drift No. 1, Sample No. 1	Slow	60.6	45,000
Sample No. 2	Quick	87.3	81,000
Drift No. 2, Sample P	Quick	110.0	54,000
Average of slow tests		38.0	27,000
Average of quick tests		75.9	49,000

Table 5  
Settlement of Structure at Various Loads

Average Base Load ksf	Settlement, in.		Total*	Immediate	Intermediate
	Immediate	Intermediate		Settlement Percent of Total	Settlement Percent of Total
8	1.082	0.223	1.305	82.9	17.1
12	1.442	0.370	1.812	79.8	20.2
20	2.120	0.790	2.910**	72.8	27.2

\* Settlement data recorded at end of rest period following load placement.

\*\* Settlement equal to 0.159 in. induced by drilling operations during final rest periods has been deducted.

Table 6  
Settlement Versus Rebound for Average Base Load  
Increment, 12 to 20 ksf

Type of Settlement	Settlement in.	Rebound in.	Rebound in Percent of Settlement
Immediate	0.678	0.294	43.3
Intermediate	<u>0.420</u>	<u>0.135</u>	32.2
Total	1.098	0.429	39.1

Note: The immediate settlement was 61.8 percent of the total, and the immediate rebound was 68.5 percent of the total. The intermediate settlement was 38.2 percent of the total, and the intermediate rebound was 31.5 percent of the total.

Table 7  
Contact Pressures During Uplift Test 2 (ksf)

Condition	Pressure, ksf, at Cited Meter					
	1	2	3	4	5	6
Before flooding	34.50	9.95	4.33	4.32	13.07	18.78
After flooding	32.45	9.65	4.55	4.65	12.90	17.42
At maximum rebound	31.57	10.27	4.87	4.82	12.97	17.68
After unwatering	33.08	9.95	4.33	4.75	13.07	18.96

---

Note: Average unit base load was 12.00 ksf.

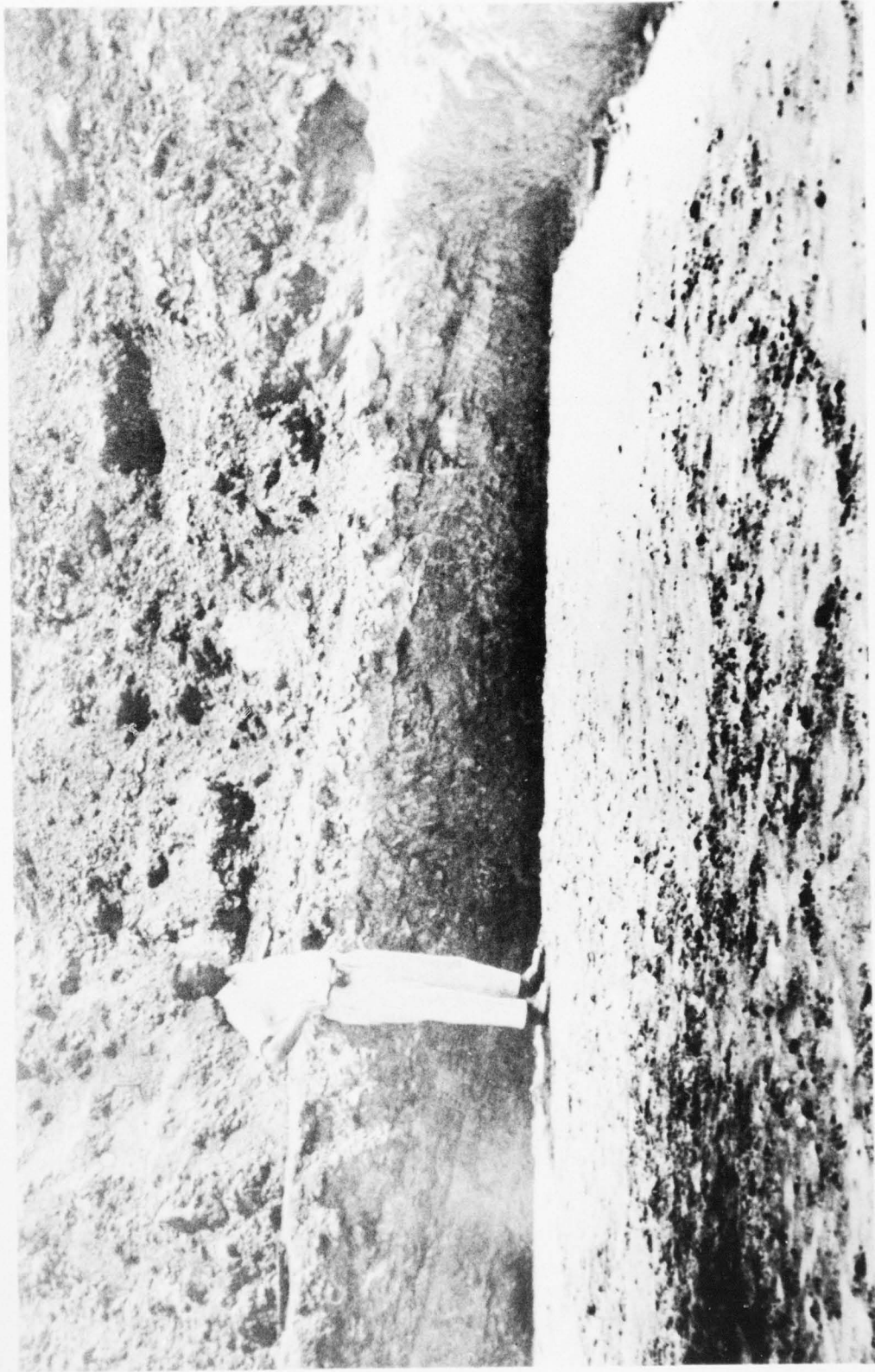


Photo 1. Interbedded black carbonaceous shale and typical green-gray Cucaracha clay shale.  
Southwest corner of excavation

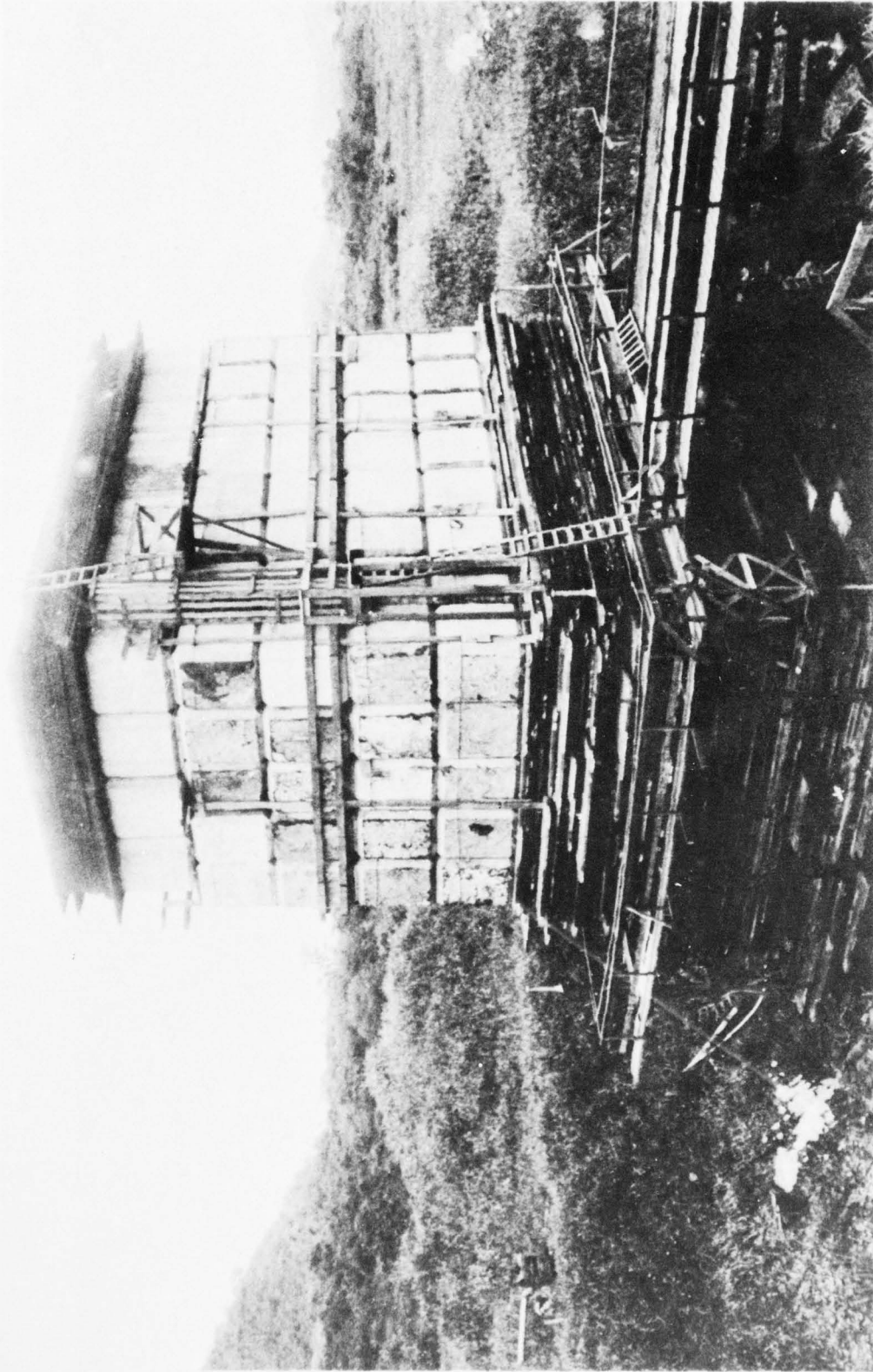


Photo 2. Test structure under maximum loading. Viewed from northwest corner

AD-A032 943

ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISS F/G 8/13  
FIELD TESTS OF THE CUCARACHA FORMATION, PANAMA CANAL 1942-1946.(U)  
MAY 74 C K SMITH, R J LUTTON

UNCLASSIFIED

WES-MP-S-74-16

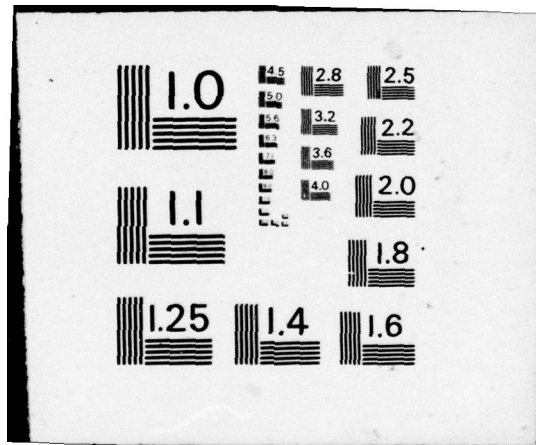
NL

2 of 2  
ADA032943



END

DATE  
FILMED  
1 - 77



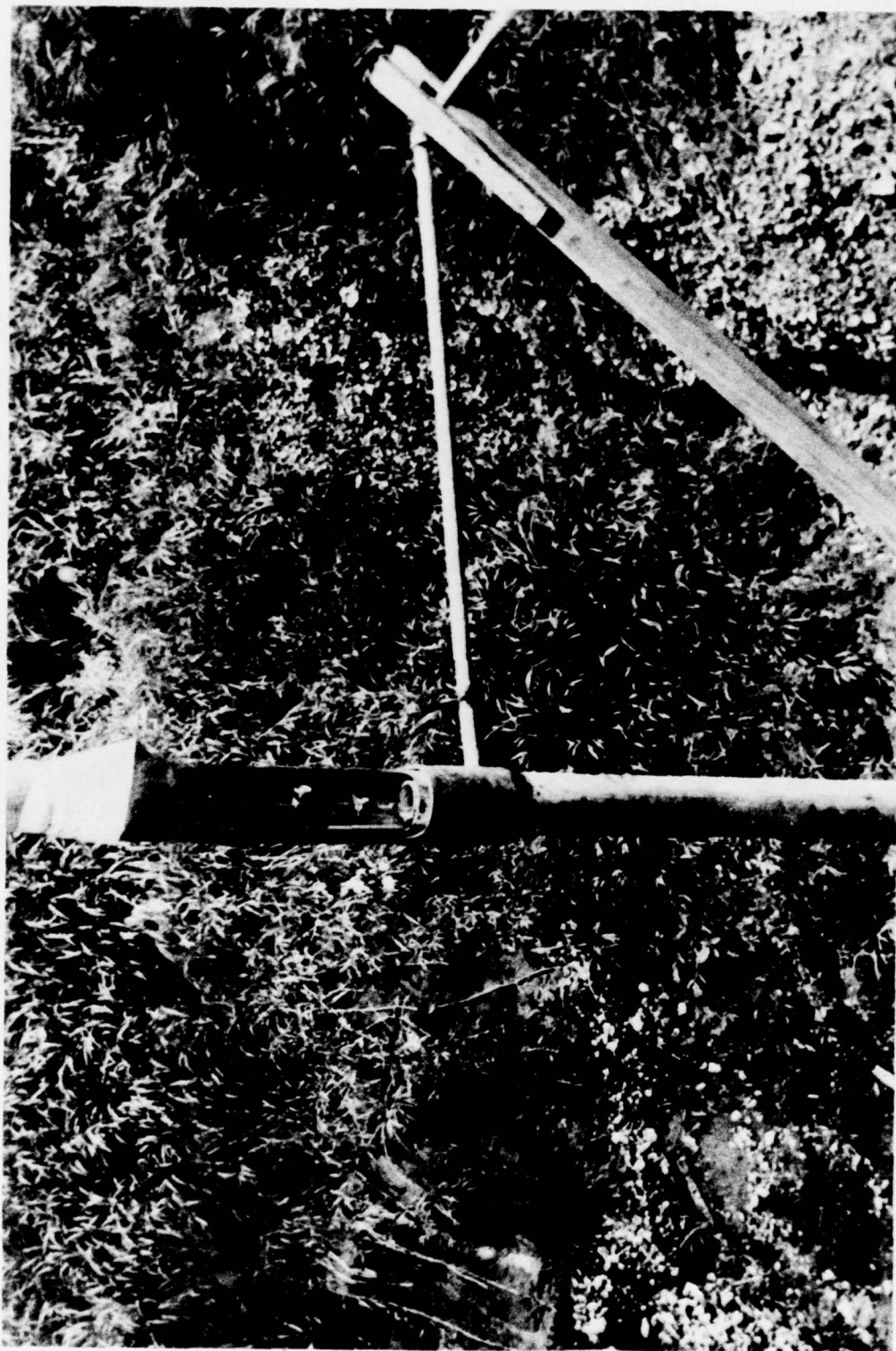


Photo 3. Hook gage assembly for measuring settlement at corners of test structure

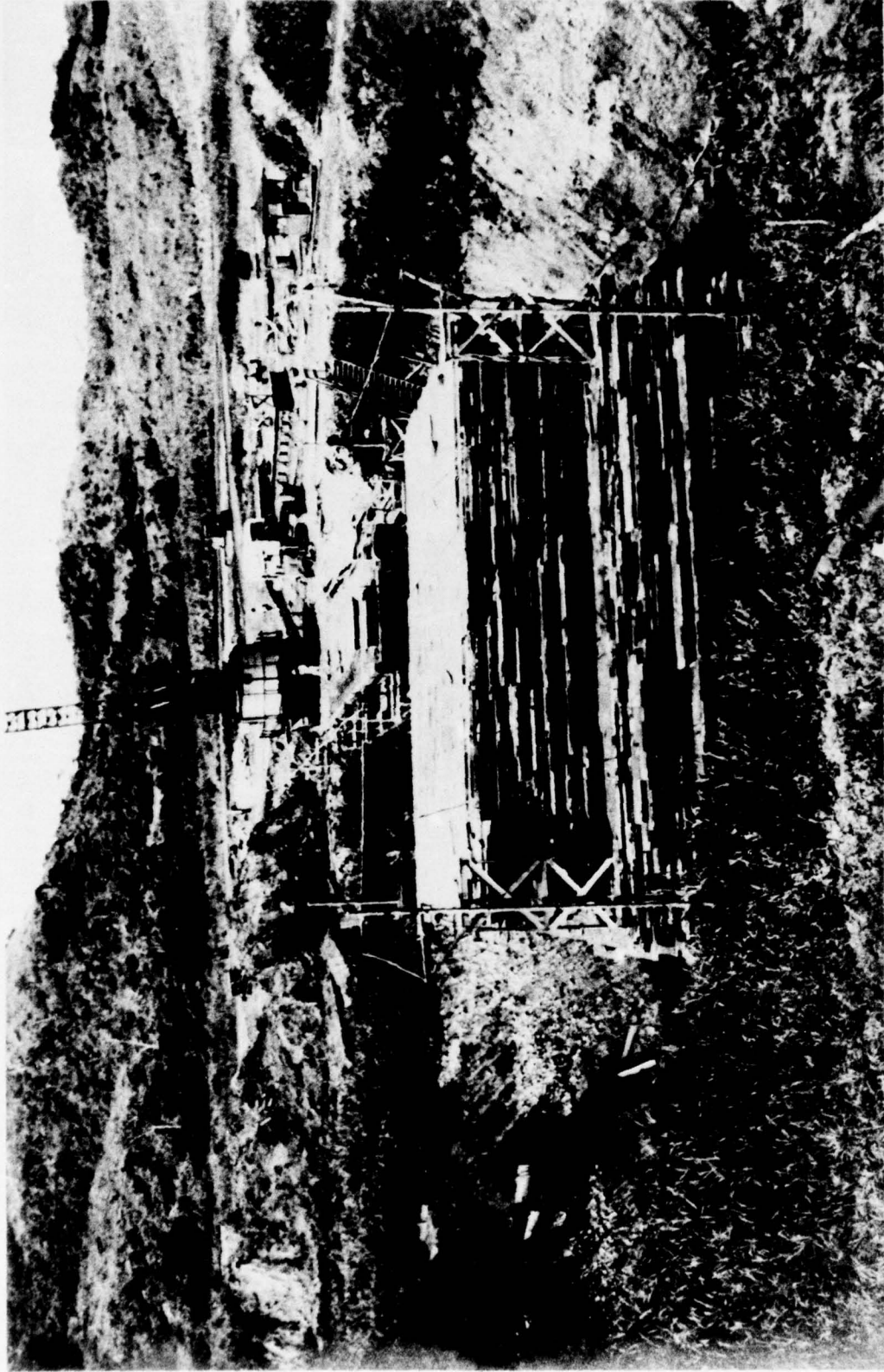


Photo 4. Water line from bench mark in center background to pipe standards for settlement measurements at corners (looking westerly during rest period under load 5)



Photo 5. Carlson stress meters on foundation along east-west axis, prior to concrete placement

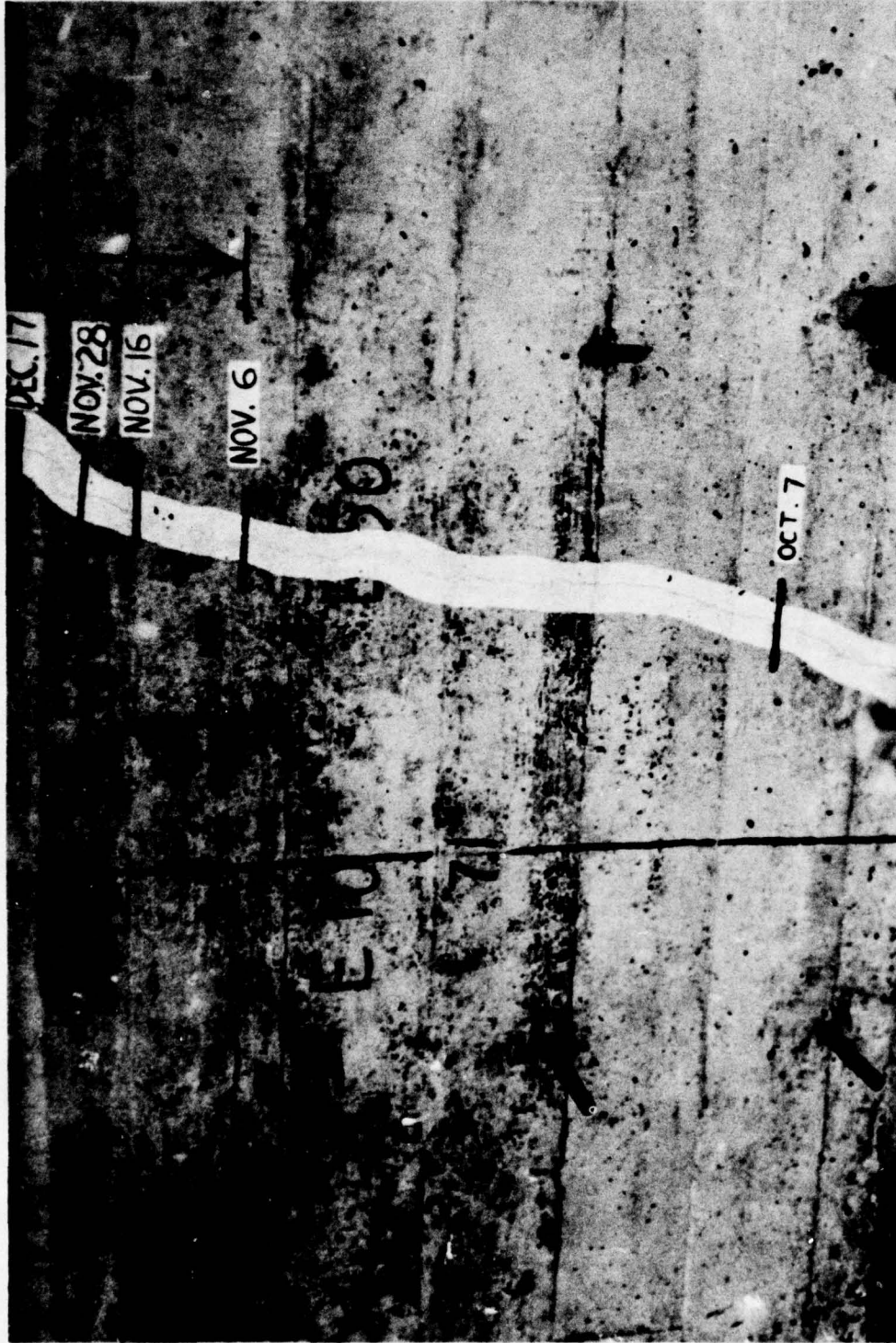


Photo 6. Crack on east side showing rate of development



Photo 7. Crack on west side showing rate of development

APPENDIX A: ORIGINAL GEOLOGICAL LOGS OF BORINGS IN THE  
VICINITY OF THE TEST STRUCTURE AND TEST PIT NO. 3

GEOLOGICAL LOG DRILL HOLE FM-77

63.2/0

(PC-2) Line P-5; Station 94460; Az. 25°54'; Offset 485'R GR. ELEV: 69.6  
*Line P-5 - Sta. 71122 (off 460)*

RECOVERY: 69.0%

69.6 OVERBURDEN, brown to dark plastic SILT, containing some  
(0.0) carbonaceous matter.

47.1 TOP OF WEATHERED ROCK  
(22.5) CUCARACHA SHALE, soft, iron-stained ground and broken  
fragments of typical Cucaracha soft, weak, slickensided  
shale.  
RECOVERY: 4.0'

37.6 TOP CUCARACHA SHALE, soft, green, slickensided shale, core much  
(32.0) broken at frequent intervals due to joints and slickensides,  
weak, incompetent foundation rock.  
RECOVERY: 26.0'

-0.4 FINAL DEPTH  
(70.0)

26

47

Classified by E. L. Spain, 5-29-40  
Typed by M. Curtis, 6-3-40  
Checked by: *AB* 6-3-40

Core Boxes Emptied  
For Reuse Aug. 46

cc-Miraflores Field Office ✓  
Soils, Lab. —

GEOLOGICAL LOG DRILL HOLE FM-296

(PPD)

Line P-8-C; Station 73450; Offset 170'R

RECOVERY: ~~90%~~ 92.9%  
GROUND ELEV: 65.2'

- 65.2      OVERBURDEN (Dredge fill to about 13.5)  
(0.0)      CLAY AND SILT, very soft, weak, buff to brown dredge fill. Silty and buff to 9.9, plastic, dark brown 9.9 to about 13.5'.
- 51.7  
(13.5)      CLAY AND PEBBLES, stiff brown clay with numerous hard water worn pebbles up to 2" across.
- 46.7  
(18.5)      TOP OF WEATHERED ROCK  
SANDSTONE, soft to medium hard, shaly, greenish, broken, weathered.  
REC: 0.6'.
- 46.0  
(19.2)      TOP OF SOUND ROCK  
SANDSTONE, shaly, medium hard, broken, iron stained along joints, green, Cucaracha formation.  
REC: 1'.
- 45.0  
(20.2)      SHALE, medium hard, soapy, somewhat slickensided and jointed, dense, typical red phase of Cucaracha, with a little local green mottling. Strength average or a little less.  
REC: 21'.
- 23.0  
(42.2)      SHALE, medium hard, somewhat jointed, slickensided, soapy, typical greenish gray phase of Cucaracha. A little red mottling in upper 10'. Mainly fairly dense, with especially dense zone around 87 to 89'. Strength about average or a little less.  
REC: 44.5'.
- 25.6  
(90.8)      SHALE, medium hard, carbonaceous, dark gray, thin-bedded, brittle, core somewhat broken down, and dip of bedding not determinable.  
REC: 4'.
- 30.8  
(96.0)      SHALE, medium hard, massively bedded, dense, soapy, somewhat jointed, slickensided, typical greenish gray phase of Cucaracha. Strength about average (for typical soapy Cucaracha) or less.  
REC: 22.5'.
- 54.1  
(119.3)      FINAL DEPTH 93.6  
8.5  
100.5  
Note: All Cucaracha, mainly soapy, and, as a whole, weaker than average.

Classified by: A. E. Sandberg, 5-20-42  
Typed by: E. L. Patton, 5-2  
Checked by: J.B. - 5-21-42

Core Boxes Emptied  
For Reuse Aug 46

cc - Geology Section (2)  
Miraflores Field Office  
Soils Lab.  
Area Geologist

GEOLOGICAL LOG DRILL HOLE NO. P1-305

(PPP)

Line P-8-C; Station 70400; Offset 140'L

RECOVERY: ~~91.0%~~ 84.9%  
Ground Elev: 65.9'

65.9 OVERBURDEN

(0.0) CLAY, locally silty to sandy, brown, fairly soft. Scattered roots. May be old flood plain deposit. Not as weak as dredge fill seen farther southeast. Stiff below about 8'. Looks like old surface, with roots and plant remains. Shrinkage 2%. Shrinks somewhat on drying.

48.4

(17.5) BOULDERS, hard, fresh, partly rounded, basalt. Fragments up to 7" across recovered. Clay and silt or sand matrix, if any, washed out in drilling.

39.4 TOP OF WEATHERED ROCK

(26.5) SHALE, weathered, soft, crumbly, iron-stained, brown.  
REC: 3'.

34.4 TOP OF SOUND ROCK (For Cucaracha slope factor)

(31.5) SHALE, medium hard, thinly laminated, with abundant plant remains. Bedding inclined. Black.  
REC: 2.8'.

30.0

(35.9) SHALE, medium hard, massively bedded, slightly soapy, slickensided; checks and crumbles on drying. Greenish gray. Silty in lower part, with scattered limy concretions. Bedding obscure, inclined about 15°.   
REC: 34'

-8.7

(74.6) SANDSTONE, hard, few joints, blue gray, medium grained. Very limy in upper foot, with fossil zones around 75.5' and 83'. Few calcite veins and joints.  
REC: 10'.

-13.1

(84.0) SHALE, carbonaceous in upper foot, then medium hard, soapy, much slickensided, jointed, typical green Cucaracha, with local red streaks.  
REC: 10'

1 of 2

GEOLOGICAL LOG DRILL HOLE PM-305

-31.0  
(96.9)  
26.5  
70.4

FINAL DEPTH

Note: Six inch core. Fair Cucaracha to 74.6'; very good,  
74.6' to 84'; then weak, 84' to final depth.

Classified by: A. E. Sandberg, 1-26-42.

Typed by: E. L. Patton, 1-26-42.

Checked by: *W. H. Lewis* 1-26-42

cc - Geology Section (2)  
Miraflores Field Office  
Soils Lab.  
Area Geologist, Pacific Area.

GEOLOGICAL LOG DRILL HOLE PM-399

(PVV)  
Line P-8-C; Station 72400; Offset 150'L

RECOVERY: 80% 73.2%  
GROUND ELEV: 66.2'

- 66.2  
(0.0) OVERBURDEN  
SILT AND SAND, few small pebbles, brown, loose. May be coarse phase of dredge fill, but not weak as usual.
- 60.7  
(5.5) CLAY, stiff, mottled red and gray, dense, massive, locally containing weathered rock material and becoming crumbly, as from 15.8 to 18.9. Plant matter in upper 2'.
- 47.3  
(18.9) BOULDERS, recovery, for distance from 18.9 to about 22' consists of a few basalt pebbles up to 2½" across.
- 44.2  
(22.0) TOP OF MUCH WEATHERED ROCK (Better class as overburden for slopes)  
SHALE, soft, plastic to crumbly, buff, weak, typical much weathered Cucaracha.  
REC: 4.6'
- 40.6  
(25.6) TOP OF WEATHERED ROCK (for slopes)  
SHALE, soft to medium hard, crumbly, broken, soapy, buff in upper to greenish gray in lower part. Somewhat weathered.  
REC: 4'
- 38.2  
(28.0) TOP OF SOUND ROCK (for slopes) (B.C.E.)  
SHALE, medium hard, massively bedded, somewhat jointed, slickensided, soapy, dense, typical greenish gray Cucaracha, of about average strength. Slightly carbonaceous zone around 59.6 to 61'. Locally very dense. Below 65', core a little stronger than average. Low recovery mainly in upper part. Slightly silty in lower part with a few limy concretions.  
REC: 47.0'

-29.1  
(95.3)  
22.0  
73.3

FINAL DEPTH

Note: All Cucaracha.

Classified by: A. E. Sandberg, 5-6-42  
Typed by: E. L. Patton, 5-7-42  
Checked by: TB 5-7-42

cc - Geology Section (2)

RES Emptied  
448 46

GEOLOGICAL LOG DRILL HOLE PM-4,99

Line P-8-C; Station 72408; Offset 9' Right

RECOVERY: 92%  
GROUND ELEV: 60.0'

Fishtailed through overburden and weathered rock zones

60.0 (0.0)		<u>TOPSOIL</u> - fine sand and silt, soft, brownish-gray.
57.0 (3.0)		<u>MUCK</u> - very soft, plastic, water-saturated, dark gray.
46.0 (14.0)		<u>ALLUVIUM</u> - silty clay with sandy streaks, brown.
42.6 (17.4)		Gravel and sand. Rocks up to 6" in diameter. Light gray to yellow. Caves when drilled into (loosely compacted).
40.0 (20.0)		<u>TOP OF WEATHERED ROCK</u> <u>SHALE</u> - weathered to the consistency of a stiff clay, closely jointed. Color: yellow.
34.9 (25.1)		<u>TOP OF SOUND ROCK</u> <u>SHALE</u> - carbonaceous, soft with crushed seams up to 6" thick, few basaltic rocks present have caved into hole from gravel bed in the overburden. Broken pieces of harder shale intermingled. Upper portions are of a drab brownish-gray hue. Grades to somewhat harder, less disturbed, greenish-gray shale. Recovery: 5.5 feet.
27.5 (32.5)		<u>GOUGE</u> - soft crushed green-gray Cucaracha shale, plastic, contains small pieces of firm rock. Recovery: 0.5 feet
26.0 (34.0)		<u>SHALE</u> - medium hard, dense, soapy, green-gray, highly slickensided. Material in core box probably not representative of in place rock composition, as this interval was dropped during drilling and was run over three times. Recovery: 3 feet.
20.0 (40.0)		<u>SHALE</u> - gray-green, slickensided, medium hard (soft where dry blocked in drilling), locally grades to compact, highly bentonitic, non-slickensided shale of a soapy texture, gouge seams occasionally recovered up to 1' in thickness and a 1' thick highly slickensided horizon (sheared) between 54.4 and 55.4 feet depth. This

GEOLOGICAL LOG DRILL HOLE PK-499

interval is perhaps a little inferior to the average Cucaracha clay shales with respect to its strength as a foundation rock. A crushed zone is present between depths of 62.0 and 64.4 feet.  
Recovery: 24.0 feet.

- 4.4  
(64.4) SHALE - medium-hard, silty, carbonaceous, dark-gray, parts on drying along bedding laminations which are inclined at about 12°. Recovery: 2.0 feet.
- 6.4  
(66.4) SANDSTONE - medium hard to hard, jointed, calcareous and contains small broken bits of shell, occasional thin, greenish-gray shale breaks and small limy concretions present. Fine grained. Recovery: 6 feet.
- 12.8  
(72.8) SHALE - soft, crushed and sheared at top to medium hard, silty, with fragments of oyster shells in lower portion. Color, dark gray. Recovery: 6.0 feet.
- 20.0  
(80.0) SHALE - soft, crushed, gray-green, slickensided, with much gouge-like material containing small lumps of harder shale. Gray-green. Recovery - 1.1 feet.
- 22.0  
(82.0) No recovery - dropped core. Drillers log describes the interval as "very soft - probably gouge".
- 27.0  
(87.0) SHALE - medium hard, slickensided, soapy, dense, varies in color from dark to olive green, typical Cucaracha shale (undisturbed) and of approximately average strength for the clay shale members of the formation. Crushed or gouge zones noted at 97-99' depth (core was here dropped and recovered on the second run) Contains a few limy concretionary horizons. Base of successive runs broken by blocking of core barrel. Recovery - 28.0'
- 60.0  
(120.0) SHALE - medium hard, silty, carbonaceous with 1/4th inch seams of coaly plant or woody material. Core samples check and part along bedding laminations. Recovery: 6.0 feet.
- 68.0  
(128.0) SHALE - medium hard, soapy, slickensided with calcite seams along joint planes. Color, gray-green. Grades to next classification. Recovery: 4.0 feet.
- SHALE - silty, medium hard, carbonaceous (disseminated carbonized plant matter). Color, dark gray. A soft, crushed,

GEOLOGICAL LOG DRILL HOLE PM-499

brown clay horizon is present from depth 128.5 to depth 130.0 feet. NOTE: There is 7 feet of core in the box for the 3 feet of drilling between 127 and 130 feet. Recovery: 3 feet.

-71.0  
(131.0)

SANDSTONE - coarse-grained, hard, "gritty", slightly friable, moderately calcareous, cored in pieces of more than 5'. This bed is composed of angular basaltic rock grains. Grades downward into the underlying conglomerate. Color, dark gray. Recovery: 14.5 feet.

-85.8  
(145.8)

CONGLOMERATE - hard, angular to subrounded pieces of basalt of about 1/2 inch average diameter in a shaly to silty matrix which becomes increasingly calcareous with depth and at the base is best classified as a conglomeratic limestone. Color, dark gray becoming light gray in basal portion. Recovery: 2.0 feet.

NOTE: Hole still drilling. Will be continued for about 75 feet more. This hole was logged to depth of 147.8 during drilling operations.

-91.3  
(151.3)

SANDSTONE - hard, medium grained becoming finer grained downward with coaly partings, calcareous, Color, light gray. Better than average rock for the Cucaracha formation. Recovery: 8.7 feet.

-100.0  
(160.0)

SHALE - medium hard to soft, slickensided and locally crushed (gougey) often with greenish-gray or brownish phases, where the lignitic material is present to greater extent. Recovery 19.5 feet. About average or somewhat below average for the Clay shale phases of the formation.

-119.5  
(179.5)

SANDSTONE - (argillaceous) soft to hard depending upon the extent to which the rock has been disturbed by shearing. Scattered hard limy concretions, bits of oyster shell and numerous orbitoidal foraminifera. This is the highest horizon containing *Lepidocyclina* that has been noted to date and is correlative to the zone designated as the "false Culebra" in the logging of the drill holes in the scheme "B" drilling. Sample taken for microfossil identification. Calcite seams along steeply dipping irregular joint planes up to 1/2 inch in thickness. Gouge zone from 181.0 to 182.4 and 187.5 to about 189.0. Is above normal strength for the Cucaracha formation. Recovery: 19.0 feet.

GEOLOGICAL LOG DRILL HOLE PM-499

- 137.0  
(197.0) SHALE (carbonaceous) dense, medium hard, dark gray with much plant material compressed into layers. Oyster shells and other megafossils noted in short sections. Generally fine-grained but locally variably silty or sandy. Becomes highly lignitic at about 207.0 feet depth and a soft crushed seam or lenticular mass is present from 208.0 to 210.8 feet, which is largely brown, crushed clay gouge. Recovery: 14.0 feet.
- 151.0  
(211.0) SHALE - medium hard to soft, dense, soapy, slickensided. Soft gouge horizons between 222.0 and 224.3 and 228.0 and 229.0 feet, otherwise is of approximately average strength for the Cucaracha formation. Recovery 33.0 feet.
- 173.0  
(243.0) SHALE (carbonaceous) generally soapy to silty but with thin lignitic layers notable. Medium hard with soft crushed zones from 244.5 to 246.0 and from 254.0 to 256.0. Bedding dips about 15 degrees. Recovery: 13.0 feet.
- 196.2  
(256.2) SHALE - medium hard, gray-green, typical Cucaracha clay shale, thin sandy streaks. Recovery: 3.3 feet.
- 199.5  
(259.5) SANDSTONE - argillaceous, gray-green, fine-grained with small rounded calcareous concretions, moderately jointed, medium hard. Recovery: 5.0 feet.
- 204.8  
(264.8) SHALE - carbonaceous, with silty streaks, medium hard, contains soft crushed seams and thin lenses. Recovery: 5.6 feet. Above average for the Cucaracha.
- 211.4  
(271.4) Final Depth

NOTE: This hole was drilled to provide fresh core samples for examination of the consultants, Mr. McAlpine and Mr. Justin, and was located within the same stratigraphic horizon as the Pedro Miguel Test area and Pedro Miguel Test Pit No. 3.

The entire section explored is within the lower and middle portions of the Cucaracha formation.

GEOLOGICAL LOG DRILL HOLE NO. PM-500

Letter: None  
 Lino: P-8-C  
 Station: 71\*12

Recovery: 91%  
 Elev: Platform 35.5  
 Offset: 147 left

35.5  
 (0.0) Platform

30.3 Top of Sand Rock - Shale, medium hard, closely jointed, dense, soapy,  
 (5.2) slickensided. Broke to short, irregular shaped pieces during drilling  
 along irregularly spaced, unoriented planes (pieces from 0.1 to 0.2 feet  
 in length). Becomes slightly darker in color from elevation 27.7 to  
 26.0, but is essentially the same rock as above. Thin calcite seams  
 present along slickensided surfaces between elevation 25.5 and 23.1.  
 Core is dry-blocked and crushed to a gouge-like material at the end of  
 successive runs.

Recovery - 6.3 feet      Sample No. 1 - Elevation 26.1 to 25.6

21.5 Shale - Light gray, dense, soapy grading to carbonaceous in lower portion.  
 (14.0) Recovery - 1.5 feet

.0 Shale - Carbonaceous, medium hard to soft. Dense, soapy and similar to  
 (15.5) overlying green clay shale in strength to elevation 18.6, thin coaly  
 seams recovered but majority of material is argillaceous. Dry-blocked  
 and crushed to clay at the end of successive runs (depths of 16.9 feet,  
 18.8 feet and 21.1 feet) - grades downward to next classification.

Recovery - 5.6 feet      Sample No. 2 - Elevation 19.7 to 19.5

" " 3 - "      18.4 to 17.9

" " 4 - "      17.6 to 17.3

14.4 Shale - Dense, soapy, medium hard, slickensided. Cored in pieces to 0.3 feet  
 (21.1) long. Grades in color from brown-green to olive-green. Dry-blocked to  
 clay at the base of successive runs. Cored in increasingly long pieces  
 below elevation 7.5 (one 2.2-foot piece recovered). This is typical of  
 the average phase of the Cucaracha clay shales. Core was dropped and  
 recovered from depths of 43.7 to 46.0 feet. Several thin gouge streaks  
 noted in lower portion of recovery (near depth of 46.0 feet)

Sample No. 5 - Elevation 11.2 to 10.7

" " 6 - "      7.8 to 6.9

" " 7 - "      6.9 to 6.3

" " 8 - "      5.6 to 2.9

" " 9 - "      -1.8 to -3.0

" " 10 - "      -3.5 to -4.9

" " 11 - "      -15.7 to -17.5

" " 12 - "      -17.5 to -19.0

-19.6 Shale - All sandy variety in section, medium hard, dense, a little stronger than  
 (55.2) average. Color green, with scattered darker, carbonaceous specks. Cored  
 in fairly long pieces (one of which was 1.6 feet in length)

Recovery 3.2 feet



GEOLOGICAL LOG DRILL HOLE NO. FM-501

Letter: None  
 Line: P-8-C  
 Station: 71+12

Recovery: 88%  
 Platform Elev.: 35.4  
 Offset: 98' left  
 103'

- 35.4  
 (0.0) Platform
- 31.4  
 (4.0) Top of Sound Rock - Shale, dense, soapy textured, non-slickensided, medium hard, olive green grading to gray green. Recovery - 0.5 feet.
- 30.8  
 (4.6) Shale - Light gray, soft, gougey, (H-3 on hardness scale) plastic. Recovery - 0.6 feet.
- 30.2  
 (5.2) Shale - Carbonaceous (brown), gougey in top 1 foot of recovery, soft (H-2 to H-3). Firm pieces of dark gray to black shale intermixed up to 0.3' in length. Soft from elevation 28.6 to 27.3. Medium hard below to elevation 25.6. Dry-blocked at ends of successive runs and ground to clay-like material. Grades to next classification. Recovery - 4.6 feet.
- 25.6  
 (7.8) Shale - Slightly carbonaceous, highly slickensided, dark greenish gray with scattered specks of bright shiny, carbonaceous material. Medium hard, soapy, dense. Grades to next classification. A 0.2-foot gouge seam present at depth of 12.3 feet with soft green clay and iron-stained angular shale fragments. Drilling fluid was lost at this depth and required setting the casing below the broken material. The fluid showed at the surface at three distinct points, 5, 10 and 20 feet from the hole. Two of the points of issue were along the trace of the break that marks the upslope limit of the small slide on this side of the structure. Recovery 3.2 feet.
- 21.3  
 (14.1) Shale - Dense, soapy, typical green Cucaracha clay shale. Moderately slickensided, medium hard. A 0.5-foot zone of gougey material was recovered from elevation 20.1 to 19.6. Successive core runs show dry-block crushing at lower ends. Core is of a somewhat darker shade of green than average. Recovery 10.3 feet.
- 10.9  
 (24.5) Shale - Crushed, gougey, soft, lighter colored than material above and below, semi-plastic, hardness H-4. Recovery - 1.3 feet.
- 9.6  
 (25.8) Shale - Medium hard, dense, soapy, green, cored in long pieces. This interval is typical of the undisturbed, better phases of the Cucaracha clay-shales. One 3-foot length of core recovered. Dry-blocked and crushed to clay at the ends of the run (elevation 5.4 to 4.4 feet). Recovery - 5.7 feet.
- ..4  
 (31.0) Shale - Medium hard, blocky jointing, parts along slickensides into 0.2-foot lengths or less. Medium hard, iron stained on parting surfaces indicating the presence of water (free). A 0.3-foot crushed seam present at elevation -2.2 to -2.5 feet. Core is crushed by dry-blocking in bottom foot of recovery of successive runs. (An 0.8-foot soft crushed zone is present from elevation -3.6 to -4.4 feet. Recovery - 7.8 feet(poor)

GEOLOGICAL LOG DRILL HOLE NO. FM-501 (Cont'd.)

- 7.8 (43.2) Shale - Slightly silty, medium hard, dense, dark gray, grades downward to a harsher texture, assuming the character of an argillaceous siltstone. Color dark greenish gray because of the presence of finely disseminated carbonaceous matter. Recovery - 4.9 feet.
- 12.9 (48.3) Sandstone - Argillaceous at top, becomes increasingly limy with depth (below elevation -15.5 feet). Medium hard to hard, depending upon calcium carbonate content. Numerous oyster shell fragments between elevation -20.7 and -21.7. Good marker horizon. Recovery - 5.8 feet.
- 21.5 (56.9) Shale - Coaly (highly carbonaceous), medium hard with layers of black, lignitized wood interspersed with more argillaceous matter. Slickensided along bedding laminae. This material is comparable in strength to the average green, slickensided, unfaulted Cucaracha clay-shale. It is stronger than the overlying carbonaceous layer in that the gouge layers are absent here. Recovery - 2.1 feet.
- 23.6 (59.0) Shale - Sandy at top but grading to soapy texture in upper 0.3 feet. Below the harsh horizon is typical dense, fine-grained clay shale ranging from moderately to highly slickensided. Dry-blocked to clay-like material at bottom of successive runs. A thin coaly layer (0.2 feet thick) cored at -31.1 feet elevation. A 0.3-foot gouge seam recovered from -34.2 to -34.5 feet elevation. This interval is probably a little weaker than average for the clay shales of the formation. Apparently some of the material was dropped and re-drilled. From elevation -36.8 to the bottom of the hole the rock is more fractured than average, but the fractures are "healed" by compression so that there is little evidence of weakening other than a closer spacing of slickens. Cores from the fractured and "healed" intervals are characterized by a pitted appearance on the cut surface. Recovery - 22.5 feet.
- 47.0 (82.4) Final Depth

Note: This hole was located at the northeast corner of the Cucaracha Foundation Test Structure. The core is stored in the humid room of the General Materials Laboratory, and consists of three boxes. 16 samples were taken as follows:

<u>Sample No.</u>	<u>Elevation</u>	<u>Sample No.</u>	<u>Elevation</u>
1	28.8 to 28.5	9	-12.1 to -12.6
2	26.9 to 26.2	10	-12.8 to -13.4
3	17.4 to 16.5	11	-21.1 to -21.6
4	14.7 to 13.2	12	-31.7 to -32.1
5	7.0 to 6.5	13	-32.3 to -33.2
6	6.5 to 6.1	14	-44.4 to -45.6
7	-6.9 to -7.4	15	-45.0 to -45.4
8	-7.4 to -8.1	16	-46.0 to -46.5

Classified by T. F. Thompson, 1/20/45  
 Typed by M. G. Evans, 1/20/45

GEOLOGICAL LOG DRILL HOLE NO. PA-502

Letter: None  
 Line: P-8-C  
 Station: 70+58

Recovery: 97%  
 Elev: Platform 36.0  
 Offset: 147 left

36.0

(0.0) Platform

31.0

(5.0)

Top of Sound Rock - Shale, medium hard, light gray, soapy, dense, scattered carbonaceous leaf imprints (well preserved).  
 Recovery - 1.6 feet.

29.4

(6.6)

Shale - Carbonaceous (variably) medium hard to soft, with crushed seams and lenses parallel to bedding. Dry-blocked to clay at bottoms of successive runs. Gougey (mashed) layers from elevation 27.2 to 25.0. Weaker than average Cucaracha shale from elevation 24.4 to 23.5 (soft, finely laminated, highly carbonaceous) intervening short sections are more argillaceous and similar in strength properties to the average green clay shales.  
 Recovery - 6.4 feet.

23.0

(13.0)

~~(29.4)~~

Shale - Medium hard, blocky, core parts on slickensided surfaces, soapy, dense. Typical of the average shale phases of the Cucaracha formation. Variations in shades of green from olive to brown. Dry-blocked to mud at the ends of successive runs. Core parts to average 0.3 pieces in drilling, or smaller. Can be scratched with the fingernail, leaving a shiny cut surface. Gougey crushed seams (not dry-blocked) from 20.4 depth (elevation 15.6) to 21.0 depth (elevation 15.0) and from 26.6 depth (elevation 9.4) to 27.0 depth (elevation 9.0). Iron stains noted at depths of 16.7 and 25.0 (elevations 19.3 and 11.0) indicate circulation of free water. A crushed and "healed" zone from 33.8 feet depth (elevation 2.2) to 35.0 feet (elevation 1.0).  
 Recovery 20.0 feet.

1.0

(35.0)

Shale - Medium hard, green, cored in sections to 2 feet long, moderately slickensided, occasionally with thin calcite seams along slickens, can be readily scratched with the fingernail, of average or slightly above average strength for the Cucaracha clay shales. Dry-blocked and mashed to clay at the ends of successive core runs. Grades to next classification.  
 Recovery - 6.0 feet.

-5.0

(41.0)

Shale - Slightly silty, green-brown, becomes browner and more silty with depth (grades to a material of argillaceous siltstone character), slickensided, with thin calcite seams along slickens. Dry-blocked to clay at ends of successive runs. Pyrite crystals present along partings at elevation -11.8, in slightly grittier than average material.  
 Recovery - 5.8 feet.

GEOLOGICAL LOG DRILL HOLE NO. FM-502 (Cont'd.)

- 12.2 Sandstone - Variably argillaceous to limy, medium hard to hard, scattered  
(48.2) pieces of broken shells present and concretions of limy material are especially notable at elevations -15.8 and -21.0. Becomes carbonaceous below elevation -19.0. Poor recovery from -15.4 to -20.0 (about 1 foot of core for 4.6 feet of drilling). Soft "gougey" layer from elevation -24.0 to -24.7. Oyster shell marker horizon from -24.7 to -25.5. Recovery - 10.9 feet.
- 25.5 Shale - "Coaly" - highly carbonaceous, dense, compact, an impure form of  
(61.5) lignitic coal. Dried pieces are inflammable. A similar layer has been found in other nearby holes at the same stratigraphic horizon. Recovery - 0.9 feet.
- 26.4 Shale - Green, medium hard with soft crushed seams, moderately slickensided,  
(62.4) soapy texture. Soft horizons recovered from elevation -29.2 to -30.2, -31.5 to -36.2. Core dry-blocked and crushed at the ends of successive runs. Core samples part to 0.4-foot pieces or shorter. Fractured and "healed" from elevation -45.4 to -46.3. Recovery - 18.8 feet.
- 47.5 Final Depth  
(83.5)

Note: This hole was located at the southwest corner of the Cucaracha Test Structure. Three boxes of samples are stored in the humid room of the General Materials Laboratory. Samples were taken as follows:

Sample No. 1 - Elevation	29.8 to 29.5	Sample No. 11 - Elevation	6.6 to 6.0
2	28.9 to 28.7	12	1.5 to 1.0
3	27.7 to 27.0	13	-1.3 to -1.8
4	26.2 to 25.6	14	-6.7 to -8.0
5	24.0 to 23.7	15	-8.4 to -8.8
6	23.4 to 23.1	16	-11.8 to -13.5
7	23.0 to 22.3	17	-21.5 to -22.6
8	21.7 to 21.0	18	-22.6 to -23.5
9	16.0 to 15.3	19	-27.5 to -28.0
10	8.6 to 8.2	20	-40.2 to -40.5
		21	-45.5 to -46.2

Classified by T. F. Thompson, 1/23/45

GEOLOGICAL LOG DRILL HOLE NO. PM-503

Letter: None  
 Line: P-8-C  
 Station: 70+58

Recovery: 94%  
 Elev: Platform 36.0  
 Offset: 98 left  
 103'

- 36.0  
 (0.0) Platform
- 31.7  
 (4.3) Top of Sound Rock - Shale, medium hard to soft and crushed. Medium hard in uppermost recovery (green color) - lighter shade below elevation 31.4 and crushed. This hole may have entered the small fault zone that cuts this corner of the excavation at the top of the hole.  
 Recovery - 2.1 feet
- 29.6 Shale - Medium hard, soapy textured, moderately slickensided, core parts along slickensided surfaces to 0.5 foot lengths or less. Blocked and ground to clay at the ends of successive runs. Color green. Soft, crushed "gougey" seams or lenses at elevations 27.3 to 27.2, 13.1 to 13.0, 10.6 to 10.2; otherwise is of about average strength for the green slickensided phase of the Cucaracha clay shales. This bed was found underlying the upper carbonaceous shale horizons in the previously-drilled holes. A fractured and "healed" horizon is present from elevation 12.4 to 11.7 and from 4.1 to 3.6, but the rock is little different at the present from its pre-fractured strength.  
 Recovery - 24.9 feet.
- 3.6  
 (32.4) Shale - Slightly silty, becoming more so with depth (grading downward to an argillaceous siltstone, slightly carbonaceous (disseminated). Color, brownish green, medium hard, slickensided. Dry-blocked to clay as much as 1.5 feet above successive runs.  
 Recovery - 7.3 feet.
- 3.7  
 (39.7) Sandstone - Variably limey to argillaceous, medium to fine-grained, medium hard to hard (hard layers of equal strength to calcareous phases of the Culebra formation). Shaley layers up to 1 foot thick show dry-block mashing. Notable shaley horizons (softer) from -9.0 to -9.3 and -11.6 to -16.0. Oyster shell horizon at -16.0 to -17.1. Calcite crystals (dog-tooth spar) present in joints cutting the fossiliferous layers. Color green to dark gray brown.  
 Recovery - 12.9 feet.
- 17.1  
 (53.1) Shale - Coaly, black, impure lignitic material (will burn when dried out) slickensided parallel to bedding laminae. Is about equal in strength to undisturbed green clay shale.  
 Recovery - 1.7 feet.

GEOLOGICAL LOG DRILL HOLE NO. FM-503 (Cont'd.)

-18.8 (54.8) Shale - Green, medium hard to soft. Cored in short pieces. Crushed zones noted at -19.3 to -20.0, (dry-blocked ?) -20.5 to -21.2, -22.0 to -22.4, -28.8 to -31.4 (dry-blocked at end), -31.4 to -32.4, -34.1 to -34.8, -35.4 to -35.8, -38.3 to -40.3. This interval is apparently more broken in the recovered samples than was noted in other holes possibly because of proximity to forementioned small fault. Inspector notes that core was lost and recovered on redrilling from -31.4 to -34.8, and -34.2 to -40.3 which may partially explain unusually broken condition.  
Recovery - 18.5 feet.

10.3 (33.3) Shale - Dark gray-green, medium hard, dense, soapy to slightly silty, less disturbed than overlying material and appears somewhat stronger. A crush seam (soft) present from -43.3 to -43.9, otherwise good solid core recovered in 1-foot lengths or less.  
Recovery - 6.7 feet.

-47.0 (83.0) Final Depth

Note: This hole was located at the southeast corner of the Cucaracha Foundation Test Structure. Three boxes of samples are stored at the humid room of the General Materials Laboratory. Samples were taken as follows:

Sample No. 1 - Elevation 27.9 to 27.3	Sample No. 9 - Elevation -4.0 to -4.7
2 25.5 to 25.1	10 -17.4 to -17.8
3 24.3 to 23.2	11 -18.4 to -18.9
4 18.2 to 17.3	12 -18.9 to -19.3
5 15.8 to 15.1	13 -31.9 to -32.2
6 9.7 to 7.4	14 -39.0 to -39.7
7 1.0 to 0.0	15 -41.3 to -41.8
8 -2.4 to -3.8	16 -42.7 to -43.2

Classified by T. F. Thompson, 1/26/45  
Typed by M. G. Evans, 1/27/45

GEOLOGICAL LOG DRILL HOLE NO. PM-504

Letter: None  
 Line: P-8-C  
 Station: 71+14.6

Recovery: 97%  
 Elev: Platform 36.0  
 Offset: 140.7 feet left

36.0  
 (0.0)

Platform

31.2  
 (4.8)

Top of Sound Rock - Shale, medium hard, soapy, dense, closely jointed, broken, (closely) to elevation +30.0. All of recovery in fragments less than 0.1 length. Dry-blocked to clay to 0.3 above successive bottoms of runs (el. 29.0, 28.0, 26.0) crushed and gougey sections from elevation 25.7 to 23.7. Grades downward to a darker brownish-green color. Lost mud (circulation fluid) during drilling above elevation 23.0 and return noted at surface at subsurface settlement point N 25 feet and at the collar of Hole PM-501. Lowered casing to elevation 23.0 and shut off leakage. Recovery - 8.8 feet.

21.5  
 (14.5)

Shale - Brown gray, dense, soapy, slightly softer than overlying material, becomes browner in lower portion. Recovery - 1.9 feet.

18.7  
 (17.3)

Shale - Carbonaceous, argillaceous to coaly, medium hard to soft and plastic, finely laminated, grades to an impure lignite in short sections, burned and crushed at ends of successive runs. A 2-inch gouge layer of gray plastic material at elev. 16.7. Recovery - 3.0 feet.

15.7  
 (20.3)

Shale - Brown-green at top (slightly carbonaceous) grading to green (purely argillaceous) downward. Medium hard, blocky parts along slickensided surfaces to pieces of 0.5 length or less, to elevation 11.0, from elevation 11.0 to 6.0, cored in long pieces (up to 3' lengths), this rock is of average or slightly above average strength for the Cucaracha clay-shales. Recovery - 11.1 feet.

6.0  
 (30.0)

Shale - Green, medium hard, core broken and crushed locally due to dropping and redrilling on 2nd run. Badly crushed at the end of recovery because of dry-blocking. This rock in place is probably better than would be thought from the appearance of the recovery. Slickensided, soapy. Color gray green where crushed, green in undisturbed sections. Recovery - 4.8 feet

1.0  
 (35.0)

Final Depth

Note: This hole located near the northwest corner of the test structure, 4.3 feet from the corner of the concrete base, 4.6 north of the north face of the base. No indication of artesian flow of water and no unusual settlement of the structure was observed during its drilling. Loss of drilling fluid above elevation 23.0 required setting casing down to this elevation. Mud loss reappeared at surface around subsurface settlement point 25 N, and at the location of Drill Hole PM-501. Core recovered was placed in storage at the humid room of the General Materials Laboratory.

GEOLOGICAL LOG DRILL HOLE NO. PM-504 (Cont'd.)

Samples were taken and wax-coated as follows:

Sample No. 1 - Elevation 27.9 to 26.9	Sample No. 4 - Elevation 22.9 to 22.5
2                    26.9 to 26.5	5                    17.5 to 17.0
3                    23.6 to 22.9	6                    16.4 to 15.5
	7                    6.4 to 5.4

Classified by T. F. Thompson, 1/27/45  
Typed by M. G. Evans 1/29/45

Unclassified  
Security Classification

**DOCUMENT CONTROL DATA - R & D**

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

<b>1. ORIGINATING ACTIVITY (Corporate author)</b> U. S. Army Engineer Waterways Experiment Station Vicksburg, Miss.		<b>2a. REPORT SECURITY CLASSIFICATION</b> Unclassified	
		<b>2b. GROUP</b>	
<b>3. REPORT TITLE</b> FIELD TESTS OF THE CUCARACHA FORMATION, PANAMA CANAL, 1942-1946			
<b>4. DESCRIPTIVE NOTES (Type of report and inclusive dates)</b> Final report			
<b>5. AUTHOR(S) (First name, middle initial, last name)</b> Carneal K. Smith Richard J. Lutton			
<b>6. REPORT DATE</b> May 1974		<b>7a. TOTAL NO. OF PAGES</b> 119	<b>7b. NO. OF REFS</b> 6
<b>8a. CONTRACT OR GRANT NO.</b>		<b>9a. ORIGINATOR'S REPORT NUMBER(S)</b> Miscellaneous Paper S-74-16	
<b>b. PROJECT NO.</b>			
<b>c.</b>		<b>9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)</b>	
<b>d.</b>			
<b>10. DISTRIBUTION STATEMENT</b> Approved for public release; distribution unlimited.			
<b>11. SUPPLEMENTARY NOTES</b>		<b>12. SPONSORING MILITARY ACTIVITY</b> Office, Chief of Engineers, U. S. Army Washington, D. C.	
<b>13. ABSTRACT</b> This report describes and presents the results of field studies of shales of the Cucaracha formation conducted during 1942-1946 by the Special Engineering Division of The Panama Canal Company in connection with the design of the Third Locks Project. Geology of the area in the vicinity of the then-proposed New Pedro Miguel Lock where the field studies were made, as presented in this report, was based on a review of geological studies available at the time of the tests supplemented by more recent studies and exploration of the Cucaracha formation by geologists of The Panama Canal Company and the U. S. Army Engineer Waterways Experiment Station. The two principal field studies discussed in the report are: (a) <u>Pedro Miguel Test Pit No. 3</u> . The pit was excavated in 1942. Field shear and bearing tests were performed in a drift on Cucaracha clay shale, and laboratory shear and bearing tests were made for comparison, and (b) <u>Cucaracha Foundation Test</u> . This test, which was initiated in early 1944, consisted of a large-scale bearing test of a concrete base 40 by 50 ft in plan and 10 ft thick. The base, which simulated that of a lock wall monolith, was loaded in stages over a period of about 9 months with steel plate and concrete to produce maximum pressures of 12.5 tsf at the toe and 7.5 tsf at the heel. Maximum pressures were maintained for about 5 months and then reduced to an average pressure of 6 tsf, which was maintained for 300 days before concluding the program in April 1946. Measurements were made of structure settlements, settlements adjacent to the structure up to 40 ft away, and pressures immediately beneath the base of the structure. Attempts to measure pressures induced at various depths beneath the structure were not successful. Recommendations made by a board of consultants in February 1945 following review of the test results are included in this report. The data indicate that settlement of large structures on Cucaracha clay shales could be higher than anticipated in 1945. It is recommended that analyses be made of the test data using present-day analytical methods.			

**DD** FORM 1 NOV 65 **1473** REPLACES DD FORM 1473, 1 JAN 64, WHICH IS OBSOLETE FOR ARMY USE.

Unclassified  
Security Classification

14.	KEY WORDS	LINK A		LINK B		LINK C	
		ROLE	WT	ROLE	WT	ROLE	WT
	Cucaracha formation						
	Field tests						
	Geology--Panama Canal						
	Load tests						
	Panama Canal						
	Pedro Miguel Locks						
	Settlement (structural)						
	Shales						
	Shear tests						
	Third Locks Project						

In accordance with ER 70-2-3, paragraph 6c(1)(b), dated 15 February 1973, a facsimile catalog card in Library of Congress format is reproduced below.

Smith, Carneal K

Field tests of the Cucaracha formation, Panama Canal, 1942-1946, by C. K. Smith and R. J. Lutton. Vicksburg, U. S. Army Engineer Waterways Experiment Station, 1974.

1 v. (various pagings) illus. 27 cm. (U. S. Waterways Experiment Station. Miscellaneous paper S-74-16)

Sponsored by Office, Chief of Engineers, U. S. Army.

Includes bibliography.

1. Cucaracha formation. 2. Field tests. 3. Geology--Panama Canal. 4. Load tests. 5. Panama Canal. 6. Pedro Miguel Locks. 7. Settlement (Structural). 8. Shales. 9. Shear tests. 10. Third Locks Project. I. Lutton, Richard J., joint author. II. U. S. Army. Corps of Engineers. (Series: U. S. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper S-74-16)

TA7.W34m no.S-74-16