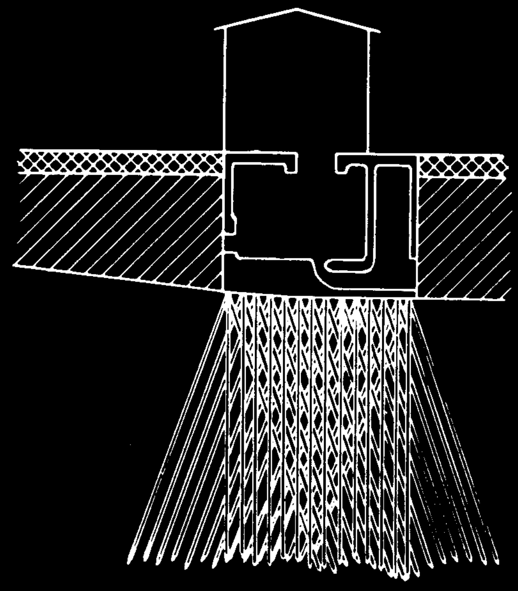


Pile Supported Structures in Lake Deposits

A Water Resources Technical Publication

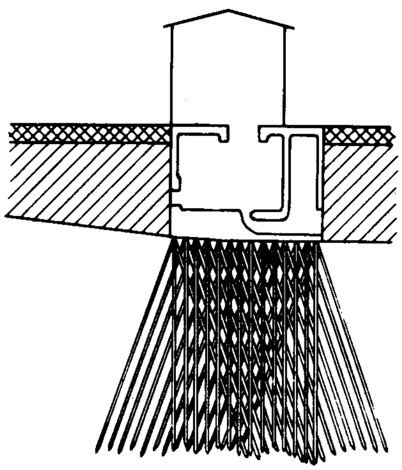
RESEARCH REPORT NO. 11

United States Department of the

INTERIOR

Bureau of Reclamation

Research Report No. 11 • A WATER RESOURCES TECHNICAL PUBLICATION



Pile Supported Structures in Lake Deposits

UNITED STATES DEPARTMENT OF THE INTERIOR
STEWART L. UDALL, *Secretary*

BUREAU OF RECLAMATION
FLOYD E. DOMINY, *Commissioner*

In its assigned function as the Nation's principal natural resource agency, the Department of the Interior bears a special obligation to assure that our expendable resources are conserved, that renewable resources are managed to produce optimum yields, and that all resources contribute their full measure to the progress, prosperity, and security of America, now and in the future.

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PREFACE

This report covers the before, during, and after engineering experience in construction of pile-supported structures on soft lake sediments predominated by lean clay. The construction of heavy structures on soft, somewhat sensitive, clay foundations presents difficult problems to soils engineers and structural designers. In this case, very thorough investigations were performed prior to construction and accurate readings were taken on instruments placed in the foundation. One of the greatest needs in soils engineering research, as expressed in the ASCE "Report on 10-Year Research Needs in Soil Mechanics and Foundation Engineering", is the broad attack on complex problems of major projects involving (1) the best analytical procedures, (2) the most advanced testing procedures, and (3) the direct measurement of performance of structures. The latter requirement is usually neglected because of its cost and other difficulties.

For these reasons, the completeness of the treatment of the foundation problems discussed herein, with the details presented on several novel test methods and full-scale field observations, makes this report a valuable source of information to structural and soil mechanics engineers faced with similar foundation problems. It may also be of educational value to the engineering departments of universities, colleges, and technical schools. Several schools have expressed an interest in the pioneering efforts of the Bureau of Reclamation in the various fields of engineering research pertinent to the design of multipurpose water resource projects, particularly when full-scale field tests are involved.

The research activities and related studies involved two pumping plants which were required on the Willard Canal, which crosses a part of the Great Salt Lake Basin near Ogden, Utah. The generally unstable characteristics of the foundation soils in this area were previously known from the investigations for the design of Willard Dam, another Bureau of Reclamation structure in the vicinity. Geologic investigations and soils engineering explorations and tests at the proposed plant sites confirmed that pile supports would be required. Adequate bearing would require driving timber friction piles into the firmer strata which exist at depths below the plant grades.

The laboratory and field soil tests, model pile studies, and field pile tests dictated the design for depth and spacing of the timber piles for construction. The validity and value of this foundation testing program are confirmed by the results of the settlement observations. Several pairs of benchmarks were set in the base of each pumping plant for recording settlement. Subsidence test readings were taken at regular intervals during construction and for about a year after completion of the pumping plants. The appreciable load changes affected by backfilling the extensive excavation for the plant foundations were apparent in the settlement observations. The backfilling operation accelerated settlement and encompassed the entire pile foundation. The effect of this load change was a matter of concern but was temporary.

The Weber Basin project of the Bureau of Reclamation in Utah consists of numerous dams, miles of pipelines and canals, and several pumping plants. The two pumping plants, Willard Pumping Plants No. 1 and 2, are major features of the project. Water is diverted from the Weber River, during periods of high flow, through the Willard Canal into a reservoir formed by the 14½-mile-long Willard Dam, which isolates part of Willard Bay from the Great Salt Lake. During irrigation seasons and low river flow periods, the Willard Pumping Plants pump water from the reservoir into the "two-way" canal system. To serve their important function in the Weber Basin reclamation system, these pumping plants were of necessity located on the flat shores of the bay area which had no promise of a "near surface" foundation. The extensive soil investigations and pile tests covered by this report were necessary to assure a stable foundation.

This publication includes the data on the soil tests and field pile tests which were furnished in the laboratory reports listed here as the source documents.

Source documents for Research Report No. 11 include the following Bureau of Reclamation Soils Engineering Laboratory reports and papers:

Report No. EM–623, "Laboratory Studies of Foundation and Embankment Materials—Willard Pumping Plants No. 1 and 2—Weber Basin Project, Utah," May 1, 1961, by C. A. Lowitz.

Report No. EM–622, "Report of Pile Testing Program for Willard Pumping Plants No. 1 and 2—Weber Basin Project, Utah," May 3, 1961, by H. J. Gibbs and H. C. Pettibone.

Report No. EM–696, "Settlement at Willard Pumping Plants No. 1 and 2—Weber Basin Project, Utah," September 15, 1964, by J. Merriman.

"Construction Load Effects on Settlement of a Soft Clay Foundation," by H. J. Gibbs and J. Merriman, *Proceedings,* Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, Canada, September 15, 1965, Vol. II, pp. 247–251.

"Effects of Driving Displacement Piles in Lean Clay," by W. G. Holtz and C. A. Lowitz, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 91, September 1965, pp. 1–13.

Included in this publication are an abstract and list of descriptors, or keywords, and "identifiers". The abstract was prepared as part of the Bureau of Reclamation's program of indexing and retrieving the literature of water resources development. The descriptors were selected from the *Thesaurus of Descriptors,* which is the Bureau's standard for listings of keywords.

Other recent issues in the Water Resources Technical Publications group are listed on the inside back cover of this report.

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SECTION I.—INTRODUCTION

General

Willard Pumping Plants No. 1 and 2 on the Bureau of Reclamation's Weber Basin project near Ogden, Utah, required extensive investigations of deep, soft lake sediments. The plants are between the project's major feature, the 14.5-mile-long Willard Dam, and the Weber River, as shown on figure 1. The Willard Dam encloses a portion of Willard Bay on the Great Salt Lake. The dam stores fresh water, which is pumped by the Willard pumping plants into an irrigation distribution system near the city of Ogden. These plants also serve to refill the reservoir behind Willard Dam by pumping surplus flows from the Weber River, thus performing a dual function. The pumping plant sites were necessarily in the broad expanses on the flat shores of the bay area, which had no promise of an adequate near-surface foundation and required piles. The site for Pumping Plant No. 1 may be visualized by the area shown in figure 2. The principal problem of the exploration program was to obtain sites for the structures which would have firm material within a reasonable depth.

The foundation soils in this area of the Salt Lake Basin are generally low-density lean clays with numerous silt and sand lenses. Many of the lenses are quite thin, but some have thicknesses to 20 feet and may be of considerable lateral extent. The soils are generally soft and of low density, often having very low penetration resistance values between depths of approximately 30 to 60 feet. Below about 60 feet, the soils generally become firmer with depth. The sediments, at reasonable depth for the base of pile supports, were less stable in the general area chosen for the location of Plant No. 1. The selection of this site, therefore, required the more extensive exploration and testing.

The depths of the plant foundations, as set by other design considerations, were 35 feet and 30.5 feet below ground surface for Plant No. 1 and Plant No. 2, respectively. At these depths the plant foundations were near the top of an approximate 20-foot-thick unstable lean clay strata.

The Soil Testing Program

The general characteristics of the foundation soils were known from investigations conducted for the design of Willard Dam. As the lake sediments constituting the foundation soils are several hundred feet deep, a pile foundation was anticipated. It was desired to locate the sites where soils of the greatest firmness existed. This was largely accomplished by exploration with the field penetration test using the split-tube sampler.[1] After the sites had been tentatively selected they were explored by test holes in which in-place vane shear tests [1] were made and thin-wall drive samples were taken. Laboratory tests were made on these samples to determine the characteristics of the soils below plant grades. Tests were also made to determine the characteristics of the soils above grade to analyze the stability of the excavated slopes. The water table in the area is high, about 4 feet below the ground surface.

Forty split-tube penetration test holes and six 3-inch-diameter and five 5-inch-diameter drive sample holes were drilled at the two plant sites and other locations in the area to bottom depths of 40 to 200 feet. The predominating fine-grained soils were generally classified as lean clay (CL) with properties as shown in table 1.

Regular triaxial shear and consolidation tests were also performed in the laboratory on several representative samples. The data given in table 1 are for soils close to or below plant grade elevations. A typical drill hole log and penetration resistance data are shown in figure 3. Typical consolidation test results for a range of initial in-place void ratio conditions are shown in figure 4. The softer samples from the upper levels had natural void ratios as high as 1.40 to 1.50 and consolidated large amounts with compression indices in the order of 0.45 in load ranges of 10 to 100 psi. At lower depths, near 70 feet, natural void ratios were in the order of 0.95 to 1.10 and the samples were less compressible, with compression indices in the order of 0.20 for the same load-range.

[1] *Earth Manual,* First Edition, 1960.

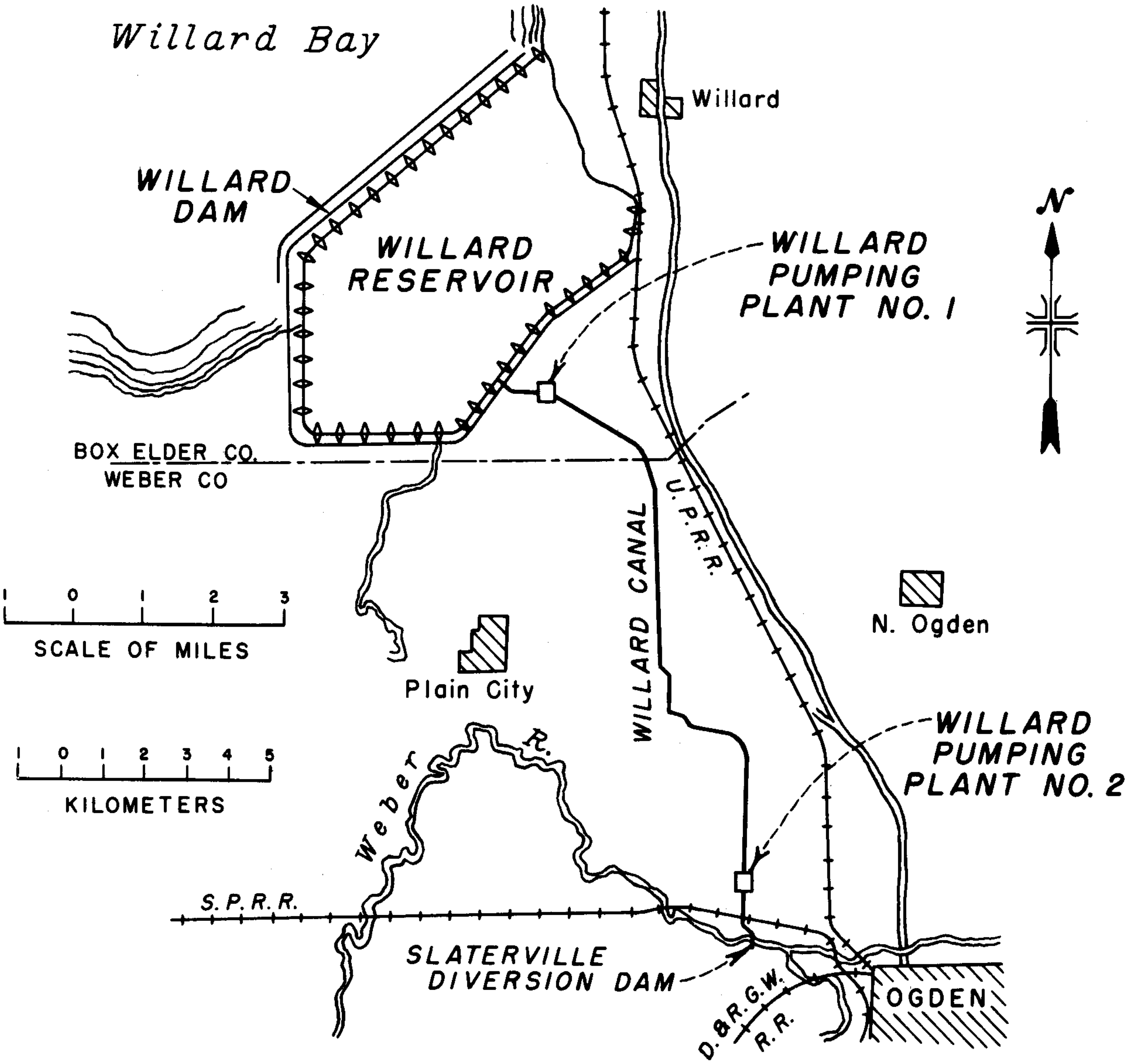

Figure 1.—Location map.

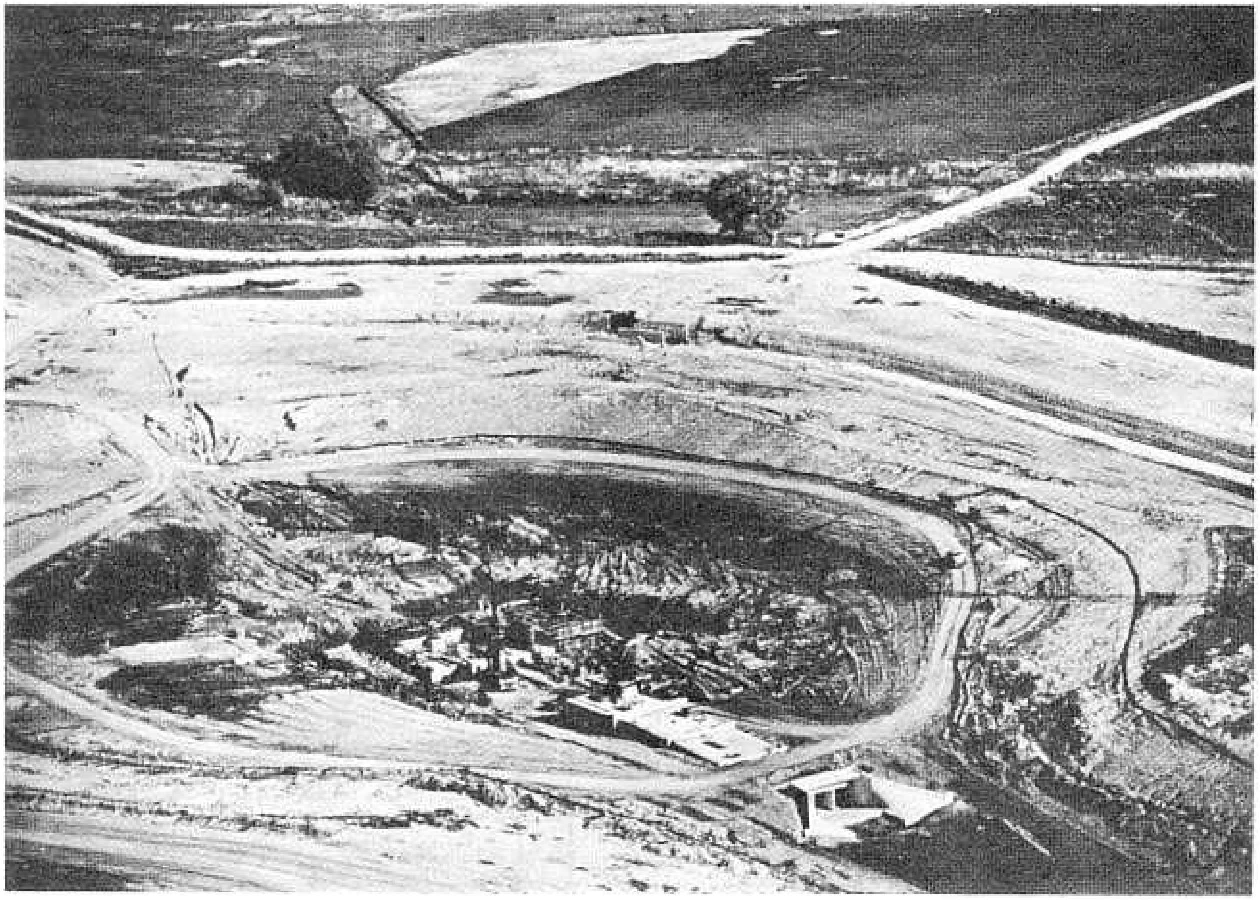

***Figure 2.*—Excavation and foundation construction, Willard Pumping Plant No. 1.**

TABLE 1.—*Soil properties*

Property	No. of tests	Maximum	Minimum	Average
Liquid limit	15	52	33	41
Plasticity index, I_P	15	29	16	23
Liquidity index, I_L	15	1	.5	.9
Unit weight, γ_l in pounds per cubic foot	18	100	67	84
Water content, in percentage	18	54	25	37
Compressive strength, q_U, in psi	22	17	2	7
Axial strain, ϵ_a, at q_U, in percentage	22	21	3	8
Triaxial cohesion, in psi	3	3	2	3
Standard penetration resistance, in blows per foot:				
30–60 feet	83	22	0	3
60–100 feet	45	29	0	11
Maximum vane shear stress, in psi:				
Natural	10	7	4	6
Remolded	35	4		2

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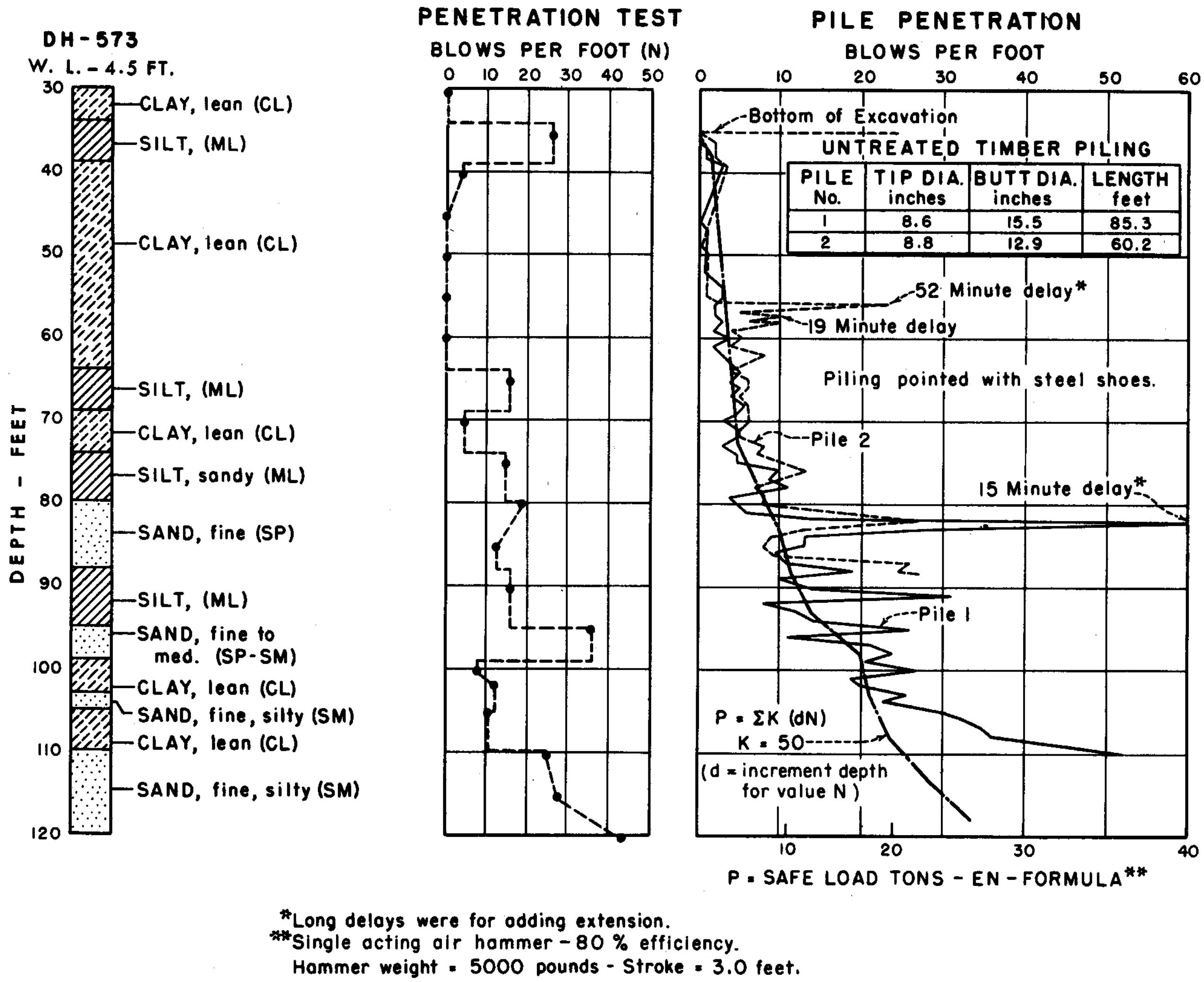


***Figure 3.*—Typical drill hole log, penetration test data, and pile driving data.**

The explorations and investigations performed at the plant sites confirmed that pile foundations would be required and that adequate bearing could be obtained by means of timber displacement piles driven into the firmer strata that existed at depth below plant grade.

When designing structures on saturated silts and lean clays of low density, there is concern that the soils may be sensitive to remolding if the relationship between plastic properties and densities becomes critical. In the case of the Willard pumping plants, the condition appeared to be borderline in this respect, with some of the samples indicating sensitivity and some close to a sensitive condition, as shown in figure 5. The numerous in-place penetration tests showed many low strength areas of lean clay, particularly between depths of 30 to 60 feet. Some tests indicated no measurable strength by this method (table 1). Vane shear tests also were made during exploration at the sites. The natural and remolded maximum strength values are given in table 1. The vane tests showed that the strength loss of the natural fine soil, when remolded along the failure plane, averaged 62 percent.

Because of the strength characteristics indicated by these field tests and the laboratory strength tests, it was decided to conduct a group of special laboratory tests, pile loading tests, and vane shear tests adjacent to the test piles to assist in judging whether driving displacement piles in the fine-grained soils might lower their strength and increase their compressibility.

Special Laboratory Tests

The special laboratory tests were made to (a) evaluate the changes in compressibility from driving dis-

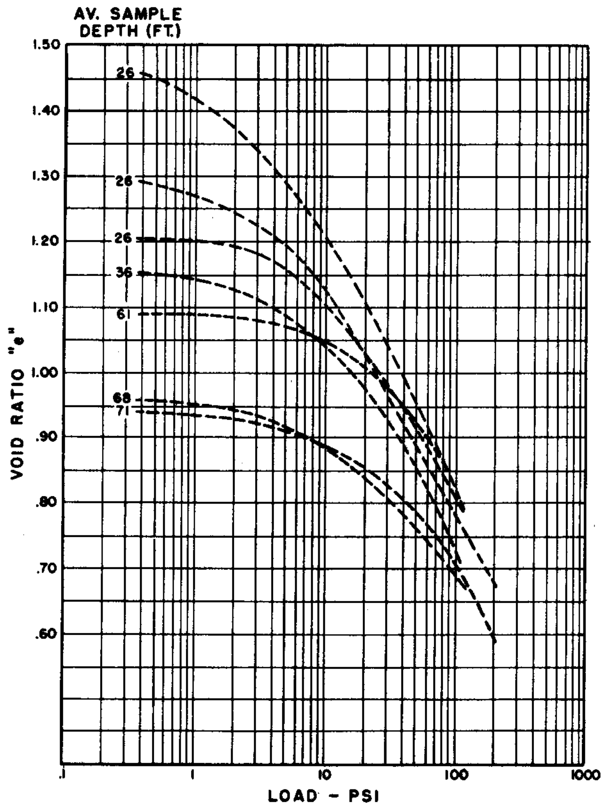
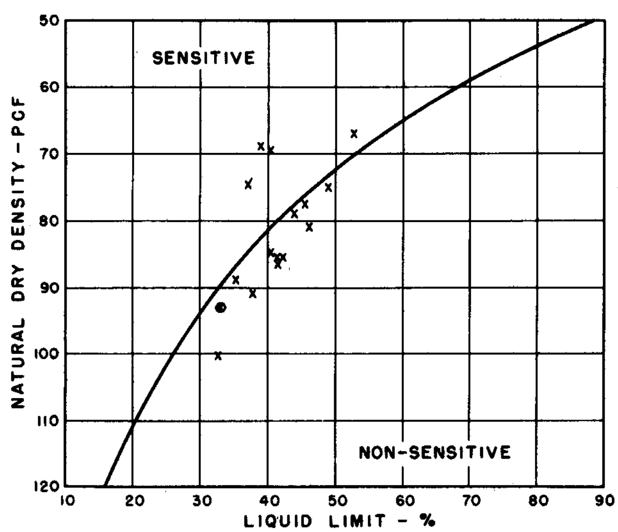


Figure 4.—Typical consolidation test results.



NOTES

Limit line shown is computed for an average specific gravity of 2.73 on the basis of complete saturation for moisture contents equivalent to liquid limits and for corresponding dry density values ($I_L = 1$).

Values shown are from Plant 1 (x) and Plant 2 (●) samples for which both density and liquid limit determinations were made.

Figure 5.—Typical qualitative estimate of soil sensitivity.

placement piles, and (b) to study the strength gain or loss of pile bearing with time.

Compressibility—To study the possible compressibility changes in the laboratory, two series of consolidation tests were conducted on thin-wall (1/8-inch) drive samples, 5 inches in diameter, taken in representative lean clay soils. Each series involved four test specimens that were cut from the samples and tested as follows:

1. Specimen A, which was similar to a triaxial shear specimen, was cut 3 1/4 inches in diameter by 9 inches long and was sealed in a rubber membrane that was clamped to perforated end plates, and then placed in a three-dimensional consolidation apparatus. All-around chamber pressures of 0.35, 12.5, 25, 50 and 100 psi were applied. Drainage was permitted.

2. Specimen B was the same as Specimen A except that, after trimming, the specimen was placed tightly in a metal cylinder and four 1/2-inch-diameter by 8-inch-long wooden dowels (model piles) were pushed into the sample. These dowels provided approximately the same amount of displacement area to soil area as anticipated for piles to be placed in the prototype foundation. The remaining test procedure was exactly the same as for Specimen A.

3. Specimen C was cut 4 1/2 inches in diameter by 1 1/4 inches deep to fit the standard Bureau of Reclamation one-dimensional consolidometer and a normal consolidation test, using five loadings of 0.35, 12.5, 25, 50 and 100 psi, was performed. Each loading was applied for 24 hours. The saturated specimen was covered with water throughout the test period and drainage was permitted as consolidation took place.

4. Specimen D was made from the same sample but was completely remolded to the approximate field density and water content to form a 4 1/4-inch-diameter by 1 1/4-inch-deep specimen for the standard one-dimensional consolidometer. The remaining test procedure was exactly the same as for Specimen C.

The results of the special Series I and II tests are given in figure 6. Data obtained from the Series I tests are shown by the upper group of curves and the data obtained from the Series II tests are shown by the lower group of curves. There were small differences in the initial conditions of the four specimens of each series: the densities of the Series I specimens varied from 70.4 to 73.2 pcf and for the Series II specimens from 81.5 to 84.1 pcf. In the case of the B specimens of both series the driving of the dowels caused a measured increase in density of 0.8 pcf. When the consolidation data of the A specimens are compared with the B specimens (Curves A and B of figure 6), and consideration is given to the initial densities, it appears that the load consolidation characteristics of the soils were reasonably similar and thus it was concluded that characteristics were not significantly changed by the soil displacement of the dowels. Considering, in the same manner, C and D specimens that were tested in the standard fixed-ring consolidometer, complete remolding did not appear to cause any alarming changes in the compression characteristics. Table 2 provides a summary of computed compression indices in load ranges between 10 and 100 psi.

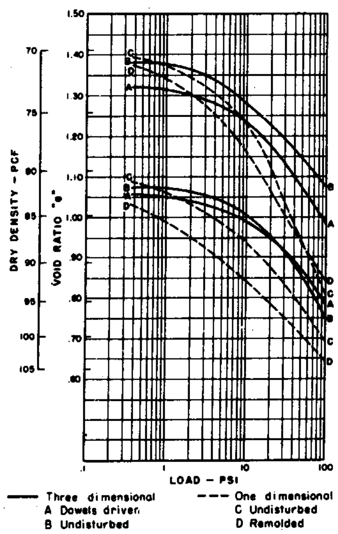


Figure 6.—**Results of special laboratory consolidation tests.**

TABLE 2.—*Computed compression indices*

Specimen	C_c (10–100 psi)	Identification
(A) SERIES I (LOW DENSITY)		
A.	0.25	3–D undisturbed.
B.	0.20	3–D dowels driven.
C.	0.42	1–D undisturbed.
D.	0.32	1–D remolded.
(B) SERIES II (MEDIUM DENSITY)		
A.	0.20	3–D undisturbed.
B.	0.25	3–D dowels driven.
C.	0.25	1–D undisturbed.
D.	0.20	1–D remolded.

Strength—In an effort to gain some preliminary information on the driving resistance and load-carrying capacity of piles driven into the soft, fine soils and to study time effects on load-bearing capacities, special laboratory tests were conducted on soil samples in which dowels (model piles) were pushed into the samples and the effects of time on driving resistance were measured. It was thought that remolding as a result of placing displacement piles might significantly reduce the soil strength and hence the driving resistance and load-bearing capacity. If the bearing capacity was affected by remolding, it was desired to know how much bearing capacity might be regained with time.

Five undisturbed drive tube samples (5 inches in diameter by 2 feet long) were selected to represent the fine-grained soils at depths of approximately 20, 30, 45, 65, and 75 feet. Four ¼-inch-diameter by 15-inch-long wooden dowels were pushed into each sample. The samples were not removed from the ⅛-inch-thick steel casing. The dowels were evenly spaced on the surface of each sample at approximately 1 inch from the edge and 8 dowel diameters apart, to minimize disturbance of the soil between dowels.

The dowels were individually pushed by a compression testing machine at a constant rate of penetration of ½ inch per minute to a total depth of 9 inches, and the load required to maintain the constant penetration rate was measured. Dowels 1, 2, 3, and 4 of each sample were then pushed from the 9-inch to a 10-inch depth at 1-day, and 1-week, 2-week, and 4-week intervals, respectively. Figure 7 is a summary plot showing for the five samples the initial maximum force required to provide the constant rate of penetration from 0 to 9 inches, the set strength or maximum force required to provide a constant rate of penetration from 9 to 10 inches after 24 hours, and the normal driving force for the 9- to 10-inch depth after the set strength had been broken.

The conclusion was reached that there would be a substantial gain in the resistance of prototype piles to penetration under load within a relatively short time after driving. Figure 8 shows the detailed record of driving resistance versus length of drive for the initial drive of 9 inches, and subsequent 1-inch drives at 1, 7, 14, and 28 days. There was approximately a 100-percent gain in resistance to driving after the 1-day set. In fact, in a few extra tests where loadings were resumed after 1 hour, the resistances increased almost the same amount. Further resistances developed at the later dates were only minor. This conclusion was borne out by later full-scale field tests.

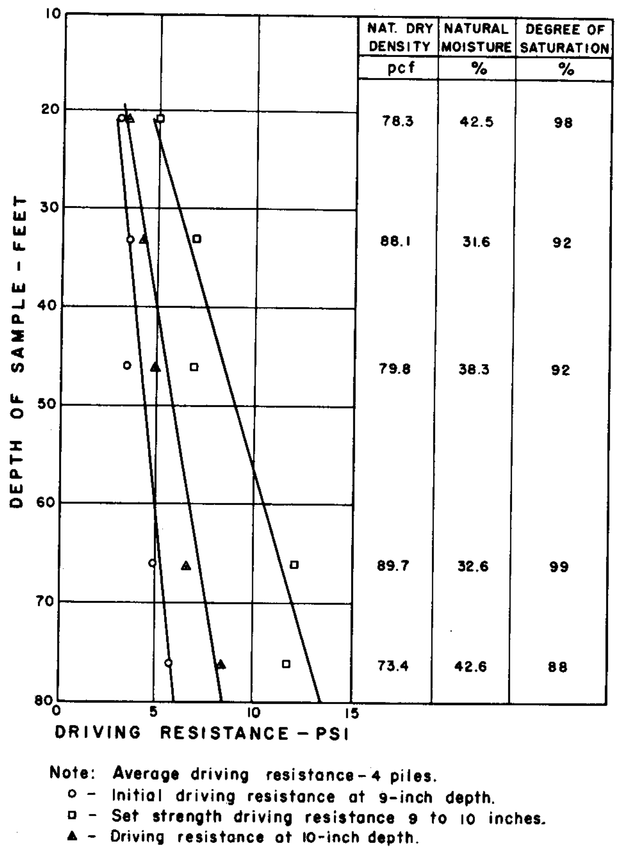


Figure 7.—**Driving resistance of model piles—Summary.**

Field Pile Testing Program

After preliminary foundation designs were made and the final sites selected on the basis of the field explorations and laboratory tests, a test pile driving and loading program was conducted at the two plant sites. Six timber test piles were driven inside of open 5-foot-diameter casings previously placed to structure foundation grades.

The pile-loading tests were conducted in conformance with ASTM Test Designation D1143–57T. Loadings were applied for 24 hours except for piles 1 and 2 in which the 60-ton loads were maintained 5 days. The maximum loadings varied from two to five times the initial contemplated design loads.

A complete record of this pile testing program is described in section III of this report. The driving records for piles 1 and 2 are shown on figure 3 with the log and penetration data of an adjacent test hole. Piles 1 and 2 had tip diameters of 8.6 and 8.8 inches and butt diameters of 15.5 and 12.9 inches, respectively. The driving resistance of these piles varied from very low to low in approximately the first 30 feet of embedment, with resistance increasing beyond this depth as firmer layers were encountered. Total resistance increased with depth for all test piles. In the case of piles 1 and 2, the final resistance, in terms of the EN formula bearing capacity, was 35 and 22 tons, respectively, reflecting the amount of embedment. All test piles showed large increases in driving resistance when delays during driving occurred, as did the model piles in the laboratory. No test piles showed any indication of failure under the applied loadings. The 100-ton loadings for 24 hours produced downward movements of 0.4 and 0.3 inch, respectively, for piles 1 and 2.

The heavy dash-dot line on the pile-penetration portion of figure 3 was initially plotted for the purpose of estimating the pile bearing capacities. This line was computed from the formula

$$P=\Sigma\ K\ (dN) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which K=a factor, depending on the soil properties, d=depth increment for value N, expressed in feet, and N=number of blows per foot for standard penetration test. This method of estimating pile bearing capacities has been previously examined.

As a further check for possible strength changes in the fine soils, 10 vane shear tests were conducted at Site 1 before and after driving test piles 1 and 2 at locations adjacent to the piles. This series of vane shear tests, in which a 3-inch-diameter by 6-inch-high vane was used, included two tests made prior to pile driving at a location midway between the locations scheduled for driving piles 1 and 2, at depths of 45 and 60 feet. Within 24 and 48 hours after driving piles 1 and 2, respectively, four vane tests also were made at a distance of approximately 3 feet from each of the two piles and at the same two depths. Later, after the load tests were completed (about 6 weeks after driving), four additional vane tests were conducted at a distance of about 1½ feet from each of the two piles and at the same depths.

Natural stress-strain values were obtained and several remolded values, taken after at least one full 360° rotation, were secured. (Rod and instrument friction values were determined in accordance with standard Bureau of Reclamation procedure and these values were deducted from the total torque applied during the stressing of the soil for determining the shear stresses applied to the soil.) Figure 9 is a summary of the vane shear test data.

There was no significant loss of strength in soil adjacent to the piles after driving. The number of tests showing slight increases in strength was approximately equal to the number showing slight decrease, and these differences could well be within the range of natural

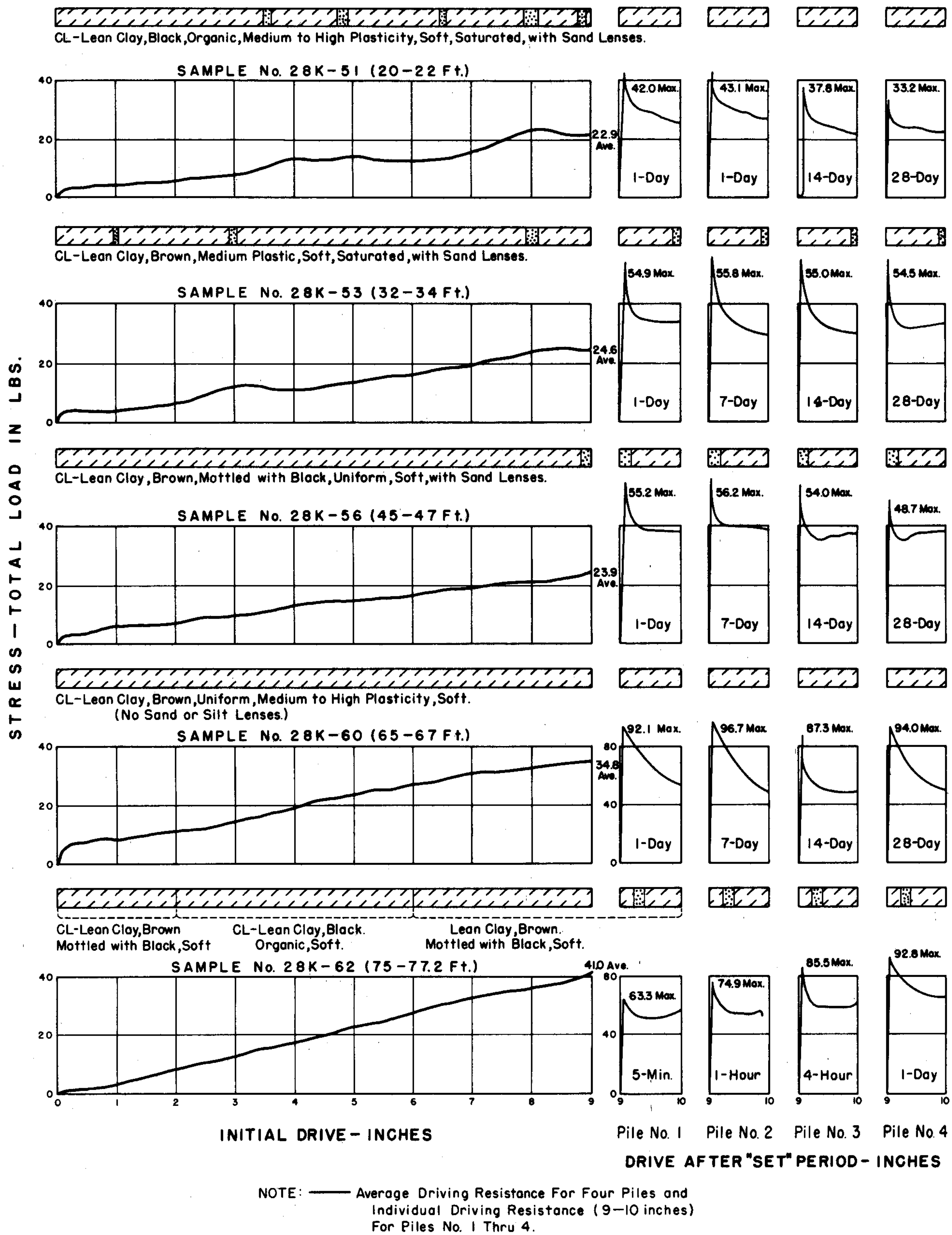
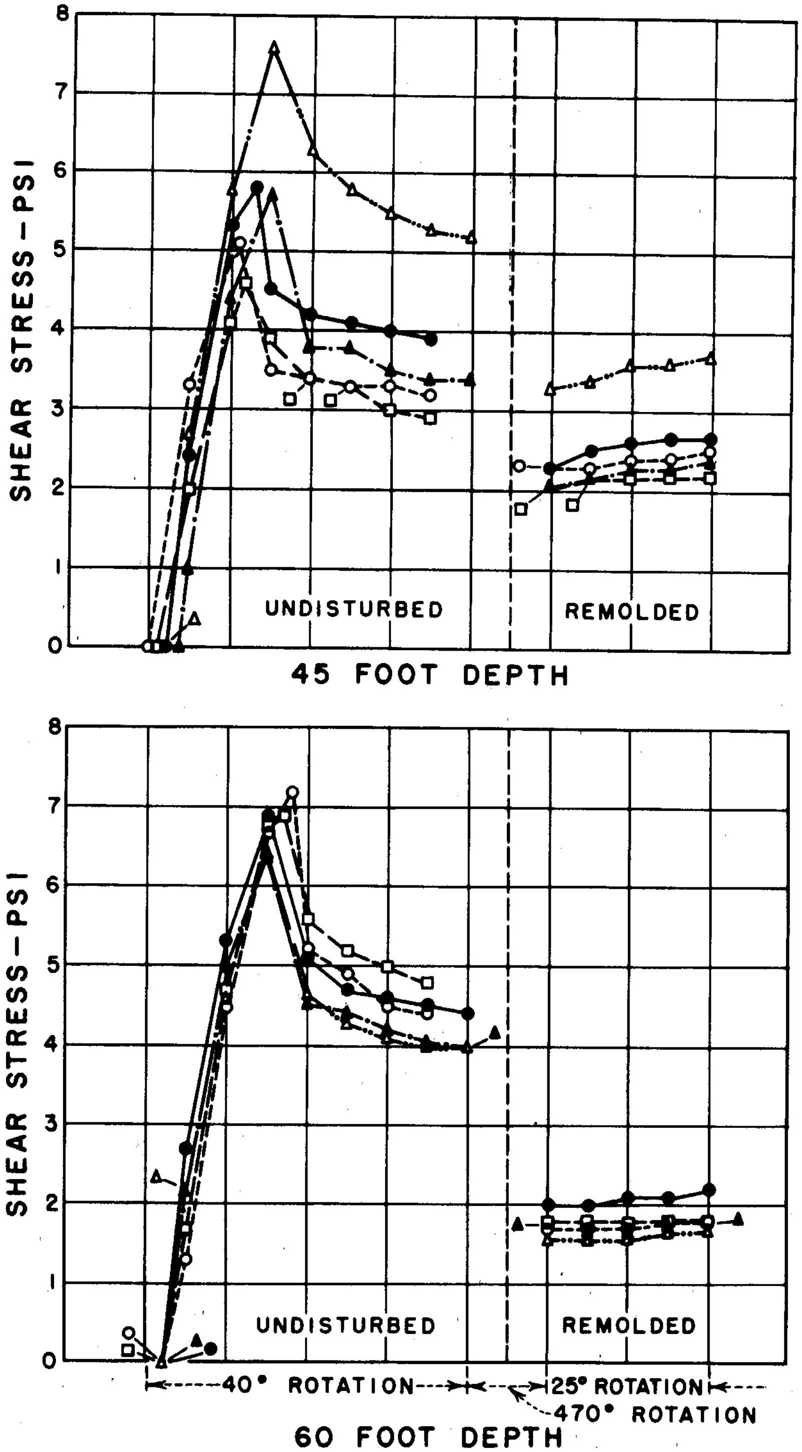


Figure 8.—**Driving record of model piles.**



Vane test before Piles 1 and 2 were driven (120) (DH-573-VT)
Vane test 24 hours after driving Pile 1 (36) (DH-573-1-VT)
Vane test 48 hours after driving Pile 2 (36) (DH-573-2-VT)
Vane test after testing Pile 1 (18) (DH-573-3-VT)
Vane test after testing Pile 2 (18) (DH-573-4-VT)
(120) Distance from piles to vane shear test in inches.

Figure 9.—Typical vane shear test results.

soil variations. It was therefore concluded that the pile driving did not significantly affect the natural strength of the soil. The remolding cycles of the vane test caused an average strength loss of 66 percent. Thus, the strength loss of the natural soil on vane remolding was only slightly greater after pile driving than the 62 percent average strength loss before pile driving, as indicated by the original investigation holes previously noted.

The Construction Program

During construction, 311 piles were driven for Plant No. 1 and 312 piles for Plant No. 2. The design required both vertical piles and piles on a 1:3 batter; the batter piles were required to resist horizontal static, operating, and earthquake loadings. The piles were spaced on approximately 3-foot 8-inch centers. The piles averaged 7½-inch tip diameter and 15-inch butt diameter. The average pile embedment below lowest foundation grade was 70 feet at Plant No. 1 and 65 feet at Plant No. 2. At Plant No. 1, the estimated maximum total design load at the most critical construction period, which included no hydrostatic uplift, was 30 tons per pile. The estimated maximum total design load during plant operation is 22.5 tons per pile and includes all static, operational, and earthquake loadings. Similar estimated maximum loadings at Plant No. 2 are 23 and 27.5 tons per pile, respectively.

Several pairs of benchmarks, for recording settlements during and after construction, were provided at both plants. The settlements measured from the start of construction to about 1 year after construction have been recorded. The pairs consisted of benchmarks placed by screwing spiral auger points in soil immediately below the structure bases and extending rods from these points up through the structure bases and floors, and second adjacent benchmarks placed in the concrete of the structure floors.

The difference in elevation changes between the benchmark pairs would indicate the relative movements of the subsoils caused by disturbance if they were significant. The differences in the movements between the paired benchmarks were negligible at both plants. For example, at 13 months after completion of construction, Benchmark 1 in the floor at the center of Plant No. 1 had settled 0.263 foot since the base concrete was placed. Benchmark 1A in the subsoil immediately below the floor at the center of the plant and adjacent to Benchmark 1 had settled 0.268 foot, and the difference between the two is not increasing. These data confirmed the conclusions previously reached on the basis of laboratory and field tests that the driving of the piles would not produce shear and compressibility changes that would cause settlements detrimental to the structures.

SECTION II.—LABORATORY SOIL TESTING PROGRAM

Plan and Objectives

This section presents the results of laboratory studies performed on samples of the subsoils in the foundation areas of the proposed Willard pumping plants.

The subsoil investigations were initiated in 1957, when an extensive drilling program was requested near the proposed Pumping Plant No. 1 site. The program included penetration-resistance-type drill holes and 3-inch undisturbed drive samples for detailed laboratory study. As the investigation program progressed, it became evident, from the study of the drill logs and laboratory test data, that foundation conditions were poor because the soft clays encountered had virtually no penetration resistance. They were very nearly saturated and of extremely low density. Therefore, further investigations were continued near the proposed site in an attempt to locate a firm sandy layer which would support the proposed pumping plant on end-bearing piles. These explorations did locate sand and gravel deposits, but they were relatively thin and too small in overall extent to be considered sufficient for supporting the proposed pumping plant.

Because of the unfavorable foundation conditions in the area of the initially proposed site for Pumping Plant No. 1, a new location was sought. An alternate location in another area was considered to have more promise of producing a sand layer at reasonable depth for the support of end-bearing piles. The investigation program for this location involved detailed penetration resistance testing to determine the extent and thickness of the sand deposit. When a sand layer of sufficient extent and considered adequate for constructing the pumping plant had been located, the investigation program was continued by obtaining undisturbed soil samples for detailed laboratory testing.

The subsoil conditions for Pumping Plant No. 2, located at Station 50+00 on the Willard Canal, were also investigated by field penetration resistance tests and undisturbed sampling for laboratory testing. Relatively firm clays underlie this site.

The testing program for Pumping Plant No. 1 produced test data which were utilized to determine (1) the stability of the cut slope required for construction of the pumping plant, and the slope of the plant fill section extending into the intake channel, (2) the settlement which may be anticipated when the construction has been completed, and (3) the bearing capacity of the soil—soft clay layers and underlying sand stratum—with respect to pile foundation design.

The data concerning logs, penetration resistance tests, pile driving records, pile load tests, and vane shear tests related to six test piles are reported in section III of this report.

Discussion of Tests

General—A review of the laboratory test data and geologic explorations performed at both locations investigated for Pumping Plant No. 1 shows the sites to be quite similar except for a firm sand layer of adequate extent at a lesser depth at the selected location. The upper approximate 30 feet of soil is a medium-firm sandy clay. This, at both locations, is underlain by a soft clay of low density. The water table was generally between 3 and 5 feet below the natural ground surface. The design conditions for the pumping plant dictate that it be founded at approximate elevation 4190. Therefore, the undisturbed sampling program and laboratory testing were concerned mainly with soils taken from below this elevation, in the soft clay layer. A summary of identification tests shows these materials to be predominantly fine-grained silts and clays of medium plasticity. The in-place densities for these soils range from approximately 70 to 90 pcf (dry weight), and the natural moistures range from approximately 25 to 53 percent.

Stability studies—Undisturbed samples of the foundation materials were selected for detailed laboratory testing in triaxial shear and one-dimensional consolidation. All triaxial shear tests were performed by the consolidated undrained test procedure. Each of the samples tested was taken from below the water table. Therefore, they were virtually saturated. To enable a reasonable testing program, the procedure was modified to permit draining of this pore water from the

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specimens during the consolidation periods. The specimen and plates were then closed, and the specimens were tested to failure, maintaining the constant applied lateral pressures required for the consolidation and measuring pore pressures throughout the tests.

These samples were selected to represent the soft materials which were to be encountered in the foundation area on the basis of the visual description of the materials and the penetration resistance. All materials tested in triaxial shear represented the soft clay layers, with little or no penetration resistance, taken from below the pumping plant foundation grade. The coefficient of internal friction for these materials ranged from approximately 0.2 for the clayey soils, with a cohesion of approximately 3 psi, to approximately 0.6, with a cohesion of 0, for the more silty materials. The most representative materials had a median tan ϕ of 0.35 and cohesion of 1 psi. This is considered to be representative of the strength of the average soft foundation soils.

Vane shear studies—Vane shear tests performed in the field at the initially proposed pumping plant location are summarized on figure 10. These represent the materials found at Drill Holes 517, 518, 519, and 520. The total vane shear strength is plotted with respect to the depth of soil strata at each location. Drill Holes 519 and 520 more closely represent the soft clay strata beneath the proposed pumping plant site. The average undisturbed strength shown here, below the approximate pumping plant grade, is approximately 4.5 psi. This value was used for the stability studies. The results of other vane tests, performed in conjunction with the pile load tests, are reported in section III. The average remolded strength from the vane shear tests was 2 psi for the above holes.

Unconfined compression tests—Unconfined compression tests were performed on test specimens at their natural moisture and density conditions. The test specimens represented subsoils ranging in depth from approximately 5 to 75 feet. The natural dry densities for the soils tested ranged between 65 to 107 pcf, with a median density ranging between 85 and 90 pounds. The moisture content determined ranged between 20 and 56 percent, with the median moisture content ranging between 30 and 35 percent. The unconfined compressive strength determined for these materials ranged from approximately 2 to 17 psi, with the median of approximately 6 psi.

The extremes of the moistures, natural densities, and unconfined compressive strengths may be attributed partly to disturbance of the samples in the sampling and preparation for testing. The percent axial strain

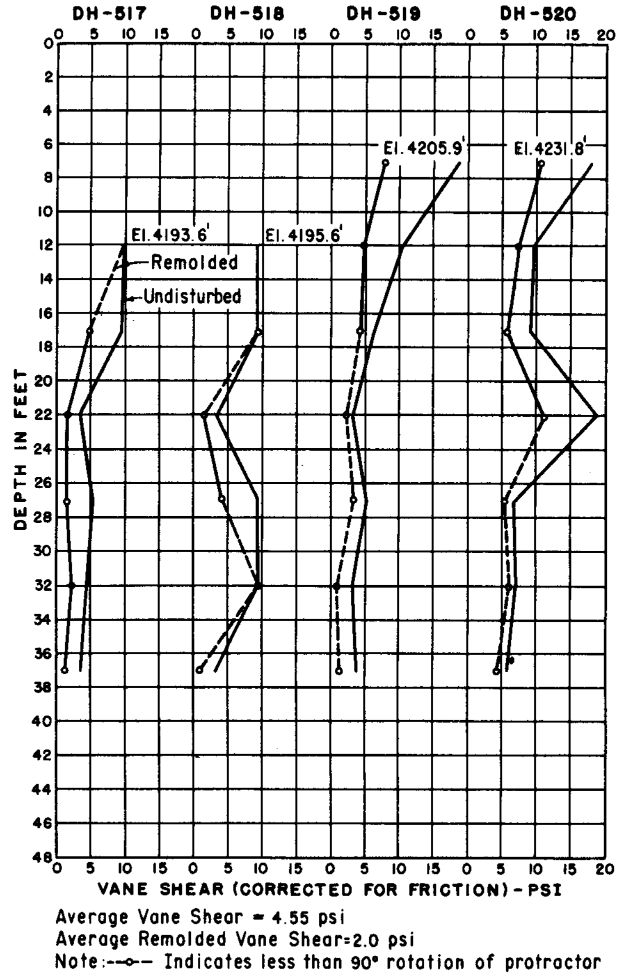

***Figure 10.*—Vane shear tests—Initial site for Plant No. 1.**

during testing varied considerably from sample to sample. Some of the more plastic, soft clay materials showed as much as 20 percent axial strain at the computed failure point. The median percent strain, as computed at failure points, was approximately 6 percent. An empirical evaluation of these data indicated that the ultimate bearing capacity of the soft clay layer would be approximately 1,700 psf.

Slope analysis for Pumping Plant No. 1—The results of these studies and vane shear test values, which were available from field tests, were used for the slope stability analyses. Cut slopes of 4 :1, 6 :1, 8 :1, and 10 :1 were investigated. The stability analysis considered probable base, toe, or slope failures. The toe failures proved to be the most critical. A minimum stability factor was determined using a cohesion of 1 psi and values of tan ϕ ranging from 0.20 to 0.56 for 4 :1, 6 :1, 8 :1, and 10 :1 slopes. The results are summarized on figure 11. Based on the results of this

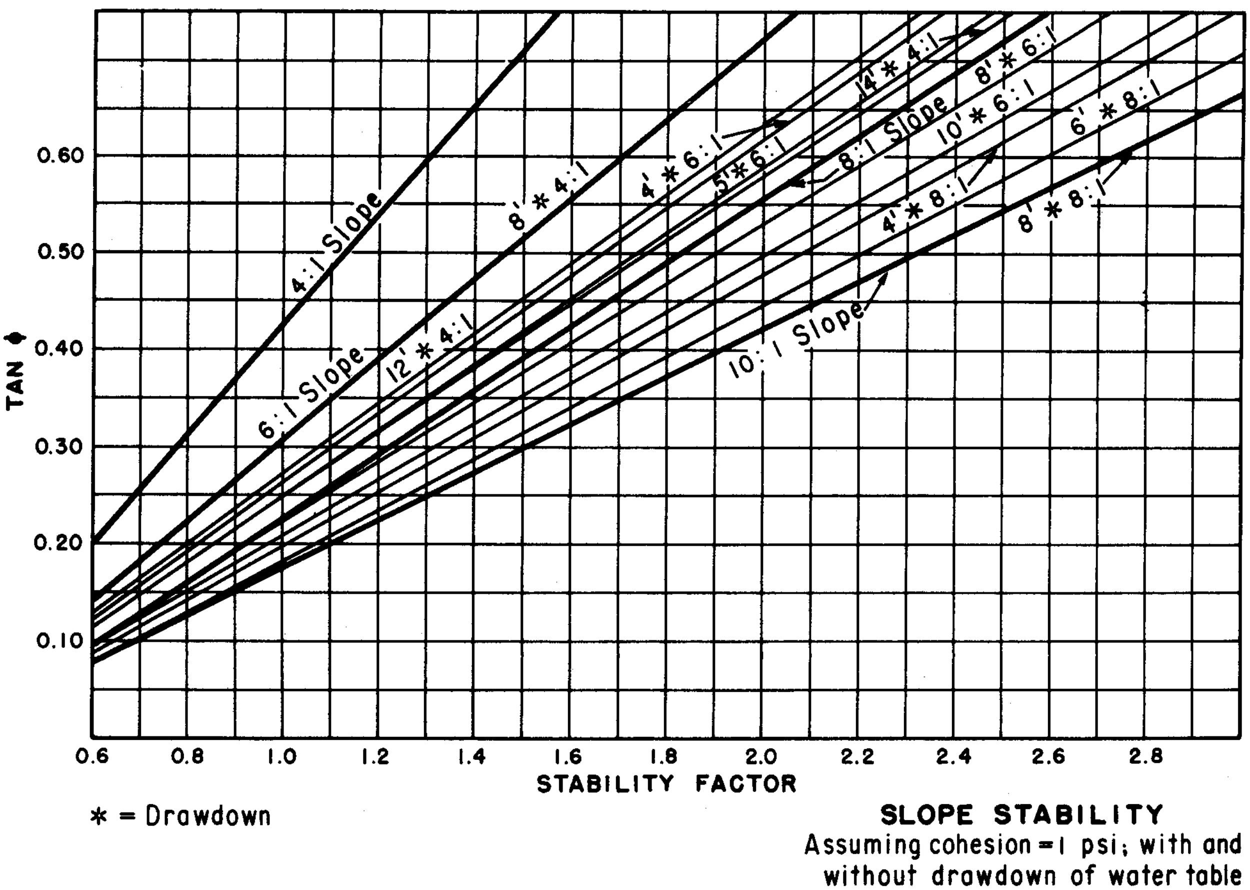

Figure 11.—**Slope stability test results.**

study, using the average shear strength values, cohesion of 1 psi and tan ϕ of 0.35, and a required stability factor of 1.5, a channel cut slope of 9 :1 was indicated.

The effect of drawdown (lowering the saturation line with respect to the cut slope by well points) was also investigated. It is shown on figure 11 that a 6:1 cut slope with 8-foot drawdown (normal to the slope) would have a 1.42 stability factor, as compared to 1.10 stability factor for the saturated condition producing full uplift. Also, if an effective drawdown of 14 feet could be assured, the study indicated a stability factor of 1.30 would be anticipated for a 4:1 slope. In all cases, the assumed drawdown conditions would need to be maintained prior to and during construction to insure the predicted stability. However, the water table could be permitted to rise with the backfill if it were desired to reduce the effective load and the resulting settlement in the underlying soft clay layer.

It was found that the general conditions of stability, discussed above for the plant cut slope, would also apply to the compacted fill section between the pumping plant and the intake structure; however, as the fill would eventually become saturated, a slope of 9:1 would be required to insure stability.

The analysis, using vane shear data obtained from the soft clay layers, was based on the critical height theory. By applying the average vane shear strengths, critical heights (Hc) of 47.7, 48.8, 49.6, and 50.5 feet were determined for cut slopes of 4:1, 6:1, 8:1, and 10:1. This compares favorably with the stability determinations made by the slip circle method.

It was once proposed to backfill the excavated area around the pumping plant, and into the intake channel, over the concrete intake pipe with a free-draining gravel, thereby reducing the volume of fill required for stability. The studies performed for this condition revealed that the flat cut slopes, as recommended above, would be stable, but the additional weight of gravel at steeper slopes would create new failure conditions in the shallower toe circle-type failure. It was concluded that the use of gravel backfill would not add to the stability; it would merely change the location of the failure circle.

Slope analysis for Pumping Plant No. 2—Field explorations in the vicinity of Pumping Plant No. 2 showed that cut slopes would be excavated in a fairly well-graded, free-draining gravel with a high water table. Dewatering of the slopes would be essential; therefore, no stability problems were anticipated. The slopes could be cut slightly flatter than the angle of repose for the gravel (about 35°) or not steeper than 2:1.

Settlement analyses—One-dimensional consolidation tests—A total of seven subsoil foundation samples was tested from the initial site location to determine their consolidation characteristics. These test specimens were selected to represent the soft clay layers.

The consolidation data from the 36-foot-deep sample shown on figure 4 were selected as representative of the average for the soft clays. These data were applied to the settlement studies.

The settlement in feet at plant grade due to fill about plant, as shown in figure 12, was computed for the average subsoil condition in the vicinity of Pumping Plant No. 1, assuming a soft clay layer between elevations 4190 and 4155. The settlement within the 35-foot layer of soft clay was computed for the maximum range of loads that could develop due to dewatering the area and compacting the fill in the excavation adjacent to the pumping plant. The maximum anticipated settlement would be caused by retaining the water table below plant grade elevation 4190 and compacting the fill to the finished grade, 4225. The load increase, over the buoyant weight of soil removed, would be approximately 2,500 psf at elevation 4190. Figure 12 shows a settlement of 1.5 feet might occur due to the fill above plant grade. Settlement of this type would be minor at Plant No. 2 because the clay foundation is firmer than at Plant No. 1, and the final fill loadings are not appreciably increased over the natural loadings.

Consideration was given to the settlement of the soils below pile tip grades at Pumping Plants Nos. 1 and 2. The following analyses are based on pile tip locations at elevation 4130 at Plant No. 1, and elevation 4179 at Plant No. 2, and an added total load (plant load + soil load) at the extremities of the pile tips of 400 psf at Plant No. 1 and 200 psf at Plant No. 2.

For Pumping Plant No. 1, the clay layer of approxi-

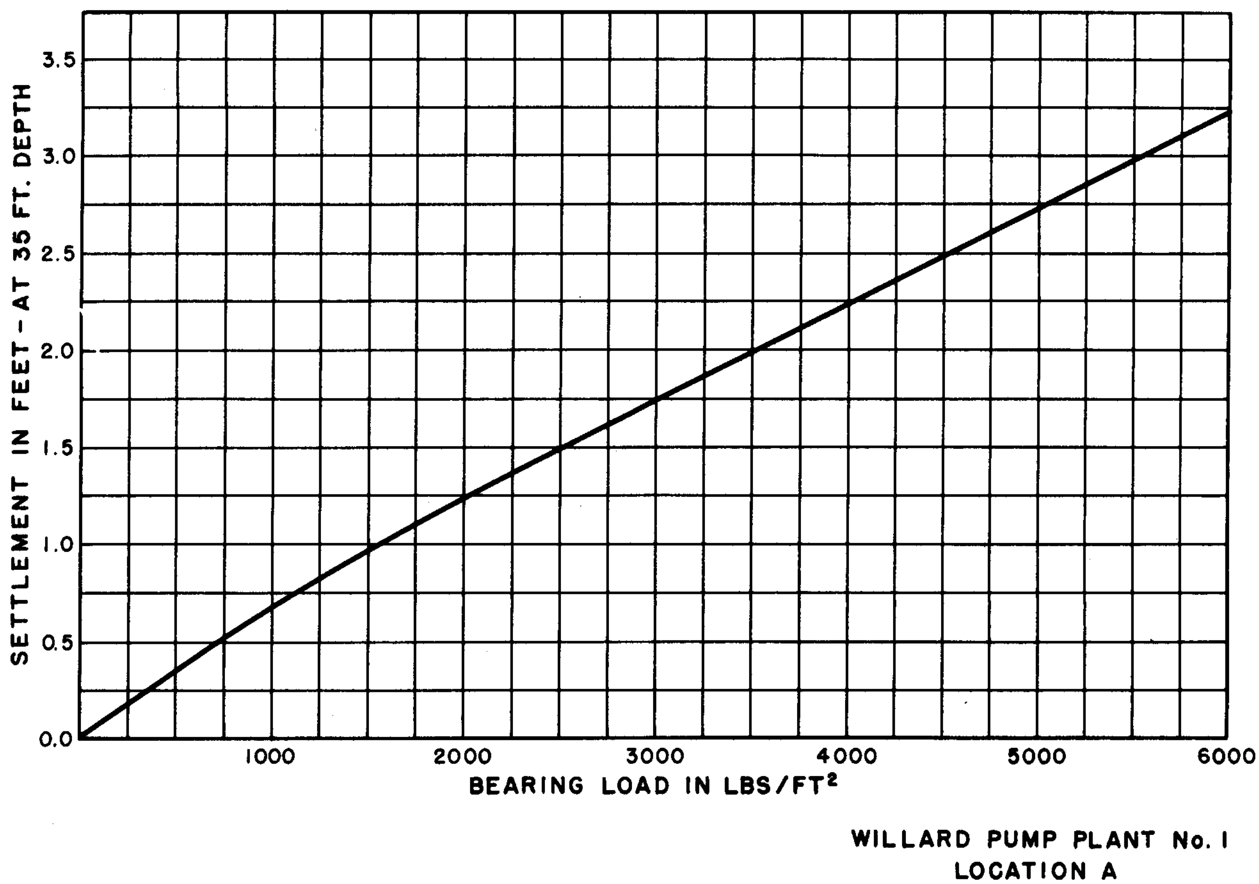

Figure 12.—**Typical bearing load test results.**

mately 10-foot thickness will have maximum settlement of approximately 0.07 foot. For Plant No. 2 (DH–565), the 15-foot layer of soft clay below elevation 4179 was considered compressible. The maximum settlement of 0.02 foot was computed for these conditions.

Model pile studies—The model pile studies were performed to determine the behavior of the soft clayey stratum and the magnitude of strength loss due to reworking and disturbance of pile driving. Five undisturbed drive-tube samples of very similar soils were selected for this testing. The natural moisture and density were determined for each sample prior to the testing. Wooden dowels of ¼-inch diameter and approximate 15-inch length were used to represent the piles. Four of these were driven into each of the undisturbed samples, being equally spaced in the soil to minimize the effect of disturbance from one pile to the other. The five samples were selected to represent depths of 20, 32, 45, 65, and 75 feet. The piles were driven at a constant rate of strain of approximately ½ inch per minute to a total initial depth of 9 inches. A stress-strain curve was plotted to show the increased driving resistance with depth. For each of the first four samples, pile 1 was permitted to set for 24 hours and then driven for 1 more inch, depth 9 to 10 inches, to check the amount of strength gained by "setting". Pile 2 was permitted to set for 1 week; pile 3, 2 weeks; pile 4 was permitted to set for 4 weeks, and their respective "set" strengths were measured. The fifth sample, 28K–62, was tested to determine the rate at which the set strength would build up. In this sample, each of the four piles was driven, as above, to the 9-inch depth. The above procedure varied in that pile 1 was allowed to set for 5 minutes; pile 2, 1 hour; pile 3, 4 hours; and pile 4 for 24 hours. Figure 8 shows the results of the driving tests. The average driving resistance of the four piles is shown plotted against the initial depth for each of the five soil samples. Also shown is the resistance of each pile to further driving after varying periods of "set". Figure 7 is a summary of the data derived from pile studies, in which the driving resistance and set strengths are shown with respect to depth of sample. The driving resistance shown here is based on the circumferential area of the pile. This plot shows that the driving resistance for a piling will increase, independent of length, with the depth of soil strata. The plot also shows the gain of strength with time.

Effect of piles on soil remolding—Additional studies were performed to determine the relative consolidation characteristics of the natural soil with respect to the same soil in which the model piles were placed. This was done by cutting two test Specimens A and B (3- by 9-inch) from an undisturbed drive sample for testing in the three-dimensional consolidation apparatus. Specimen B was partially remolded by driving four ½-inch wooden dowels into the soil. This would give about the same pile area to soil area ratio as planned for the plant foundations. Specimens C and D were tested in the one-dimensional consolidation machine. Specimen C was an undisturbed specimen cut from the same sample as Specimens A and B, while the material for Specimen D was obtained from the same sample and completely remolded and placed at approximately the same moisture content and density as Specimen C.

Table 3 shows that the initial soil density for each of Specimens B was less than the density for Specimens A of the same sample. The difference in the initial density for Specimens A and B is 2.5 pcf for Sample No. 28K–323 and 1.7 pcf for Sample No. 28K–327. In each case, Specimen A was taken from the bottom of the sample. The difference in the densities of Specimens A and B is due to unavoidable compression of the soft soil while taking the drive samples. This is also indicated by the lesser density of Specimens C, which were taken from the top of each drive sample.

Also, the data in table 3 show that driving the model piles increased the soil density in each case 0.8 pcf.

When the consolidation characteristics of Specimens A are compared with B, giving consideration to initial densities, it is evident that the consolidation-load relation, as shown by the shape of the consolidation curves in figure 6, is very similar and indicates that the consolidation characteristics were not affected significantly by driving the model piles. Considering Specimens C and D in the same manner, complete remolding caused only a small amount of additional consolidation in the low-load range as compared with undisturbed conditions (0.75 percent at the approximate over-burden load).

Test Procedures

All standard laboratory tests were performed in accordance with the procedures described in the Bureau of Reclamation's *Earth Manual,* First Edition, 1960. The special test procedures are discussed in the text.

Table 3.—*Summary of one-dimensional consolidation test results*

Sample identification				Initial specimen data			Percent consolidation						Maximum load and saturated			
Laboratory sample No.	Location	Depth (feet)	Specific gravity	Dry density (pcf)	Moisture content (percent)	Degree of saturation (percent)	12.5 psi	25 psi	50 psi	100 psi	100 psi, saturated	Rebound (percent of maximum consolidation)	Dry density (pcf)	Moisture content (percent)	Degree of saturation (percent)	Percolation rate (ft/yr)
ONE-DIMENSIONAL CONSOLIDATION																
28K–323	Spec. C.....	Natural......	2.71	70.4	46.2	89	8.3	13.5	19.5	25.2	25.4	29	94.3	26.6	91	....
	D.....	Remolded....	.....	71.4	46.0	89	11.0	15.2	19.3	23.7	23.9	27	92.8	29.0	95	....
28K–327	Spec. C.....	Natural......	2.73	81.5	39.9	99	8.3	11.3	15.0	19.4	19.5	38	101.2	28.1	100	....
	D.....	Remolded....	.....	84.1	36.6	97	9.6	12.5	15.7	19.1	19.2	29	104.1	24.2	100	....
THREE-DIMENSIONAL CONSOLIDATION																
28K–323	Spec. A.....	Natural......	2.71	73.2	48.8	100	4.0	6.5	9.7	14.6	.....	....	85.3	37.6	100	....
	B.....	With piles....	.....	71.5	50.5	100	6.5	9.2	11.0	13.2	.....	....	80.3	40.2	100	....
		Before piles...	.....	70.7	51.8	.....	.....	.....	.....	.....	.....	....	.....	.....	.....	....
28K–327	Spec. A.....	Natural......	2.73	83.1	38.3	100	3.9	6.1	9.2	13.5	.....	....	95.8	28.6	100	....
	B.....	With piles....	.....	82.2	39.8	100	3.7	6.3	10.5	15.4	.....	....	95.8	30.0	100	....
		Before piles...	.....	81.4	39.8	.....	.....	.....	.....	.....	.....	....	.....	.....	.....	....

SECTION III.—PILE TESTING PROGRAM

Synopsis

The results of the laboratory tests described under section II indicated that Willard Pumping Plants Nos. 1 and 2 should be constructed on foundations requiring piles. Six untreated timber piles were driven and tested to determine their bearing capacity in various strata under the proposed pumping plants. Piles 1 to 4 were located at Pumping Plant No. 1, and piles 5 and 6 were at Pumping Plant No. 2.

Piles 5 and 6 were tested to the specified maximum load of 40 tons using ASTM Test Designation D1143–57T, "Tentative Method of Test for Load-Settlement Relationship for Individual Piles under Vertical Axial Loads." Figure 13 shows a typical detail of the method used for vertical loading of the test piles. The tests on piles 1 through 4 were also conducted according to this ASTM procedure, but the maximum loads were altered as follows:

a. Test piles 1 and 2 were first subjected to vertical loading of 40 tons as provided by the specifications; then the piles were unloaded and zero load maintained for 24 hours. They were then subjected to a constant load of 60 tons for 120 hours. After unloading and again maintaining zero load for 24 hours, the vertical load was increased on each pile to 100 tons or failure, whichever occurred first. Since the load of 100 tons was attained without failure of either pile, it was maintained for 24 hours and then unloaded, maintaining the zero load for 24 hours.

b. Test piles 3 and 4 were first loaded to the specified 40 tons load; then in a second test each pile was loaded to 60 tons. In addition, pile 3 was tested to 80 tons.

c. The vertical pull test on pile 1 was increased to 20 tons instead of the 10 tons specified; then in a second test the pile was pulled to the maximum capacity of the equipment, which was 30 tons. Piles 2 and 5 were tested by vertical pull to a maximum load of 10 tons, as originally specified.

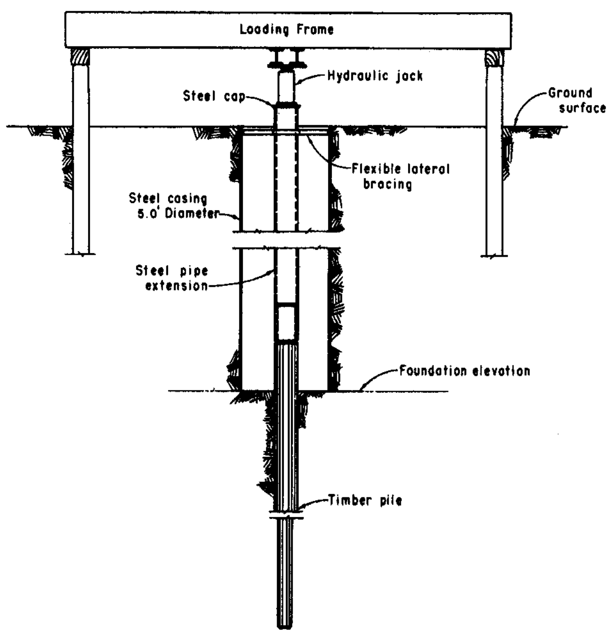

***Figure 13.*—Section through test pile under vertical loading.**

Test Sites

The tests were at the sites of Willard Pumping Plants Nos. 1 and 2 (figure 1).

The subsoil at Plant No. 1 consisted of interstratified layers of fine sand, silt, and lean clay. The drill logs in figures 14 and 15 indicate that the penetration resistance values vary from zero to about 45 blows per foot at Plant No. 1, with a very soft layer extending from 30 to 35 feet of depth below the foundation excavation. At Plant No. 2, the first 28 feet of the present subsoil consisted of well-graded sand and gravel with a maximum size of 5 inches (figure 16). The proposed foundation elevation of Plant No. 2 will be below this gravel. The material below the gravel is a lean clay with a penetration resistance value of between five and 30 blows per foot, with the penetration resistance test values increasing generally with depth.

Since the softer clay soils underlying Plant No. 1 presented the more difficult foundation problems, it was decided to drive and test four piles at this location. Two piles were tested at Plant No. 2. The locations of individual piles with respect to the drill holes at each site are given in figures 17 and 18.

The proposed foundation elevations of Plants Nos. 1 and 2 were 35 and 30.5 feet, respectively, below the existing ground level. Five-foot-diameter steel casings were installed at the location of each test pile so that

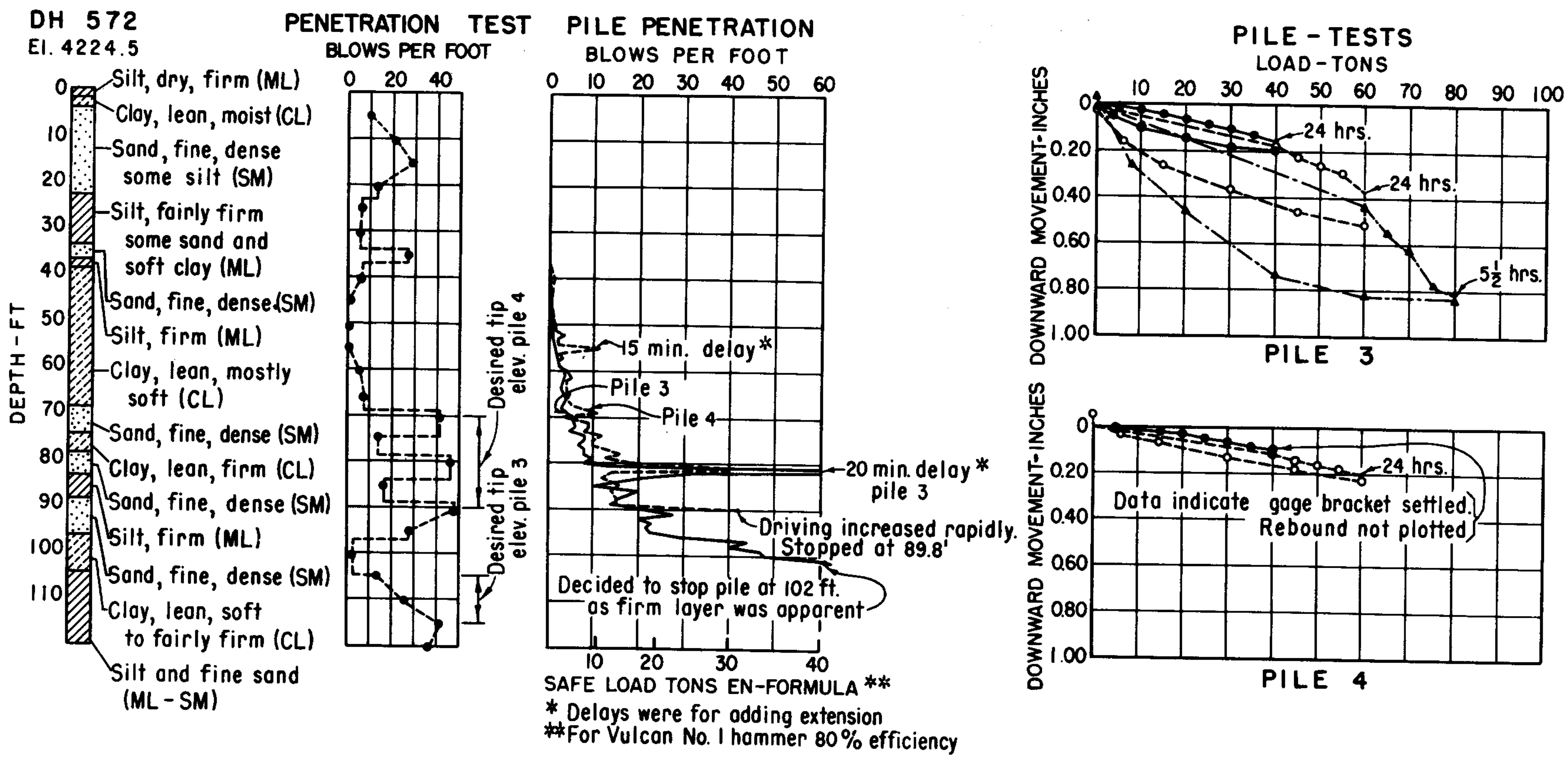

Figure 14.—**Penetration resistance, pile driving record, and pile load tests for piles 3 and 4, Pumping Plant No. 1.**

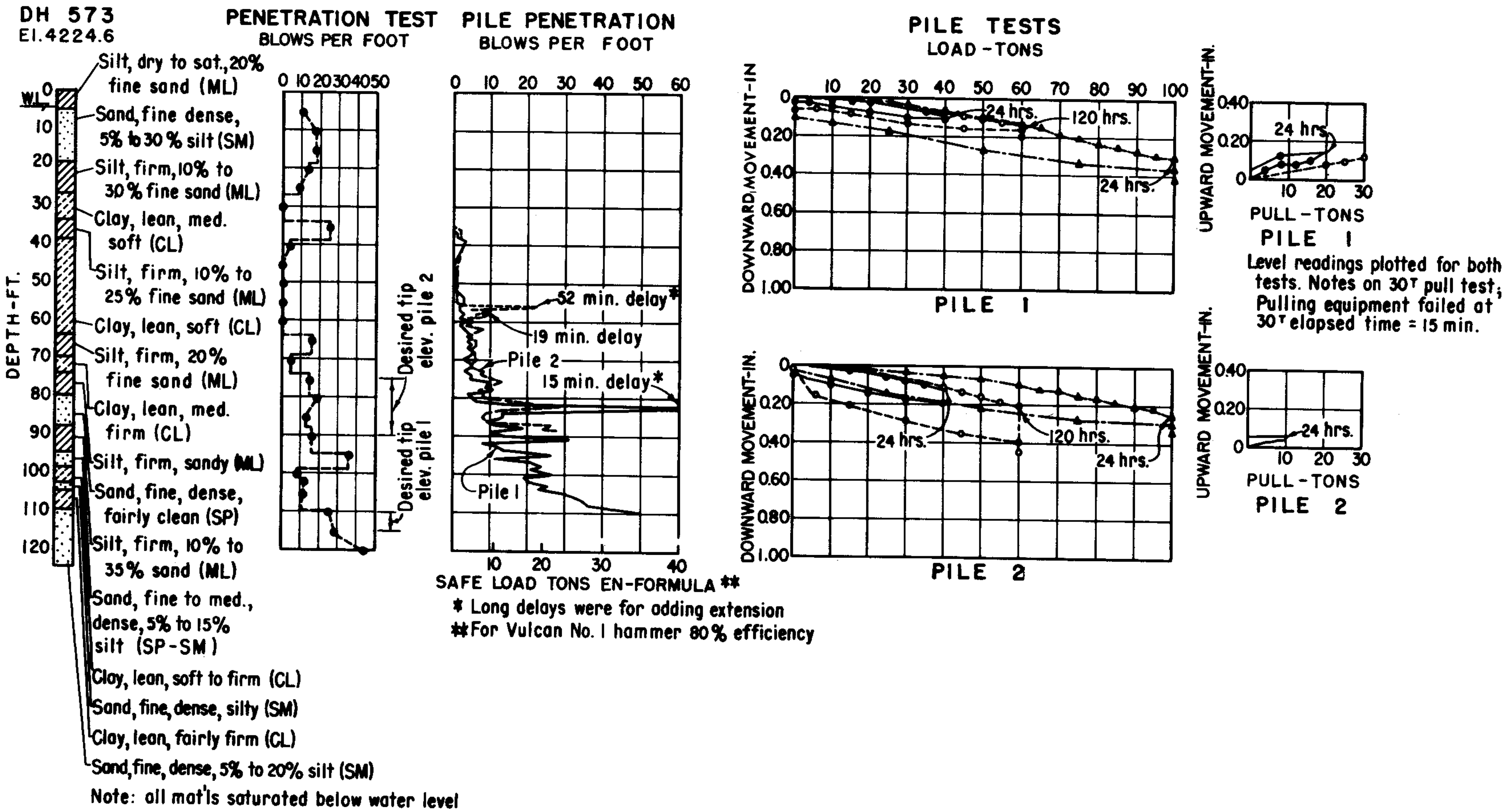

Figure 15.—**Penetration resistance, pile driving record, and pile load tests for piles 1 and2, Pumping Plant No. 1.**

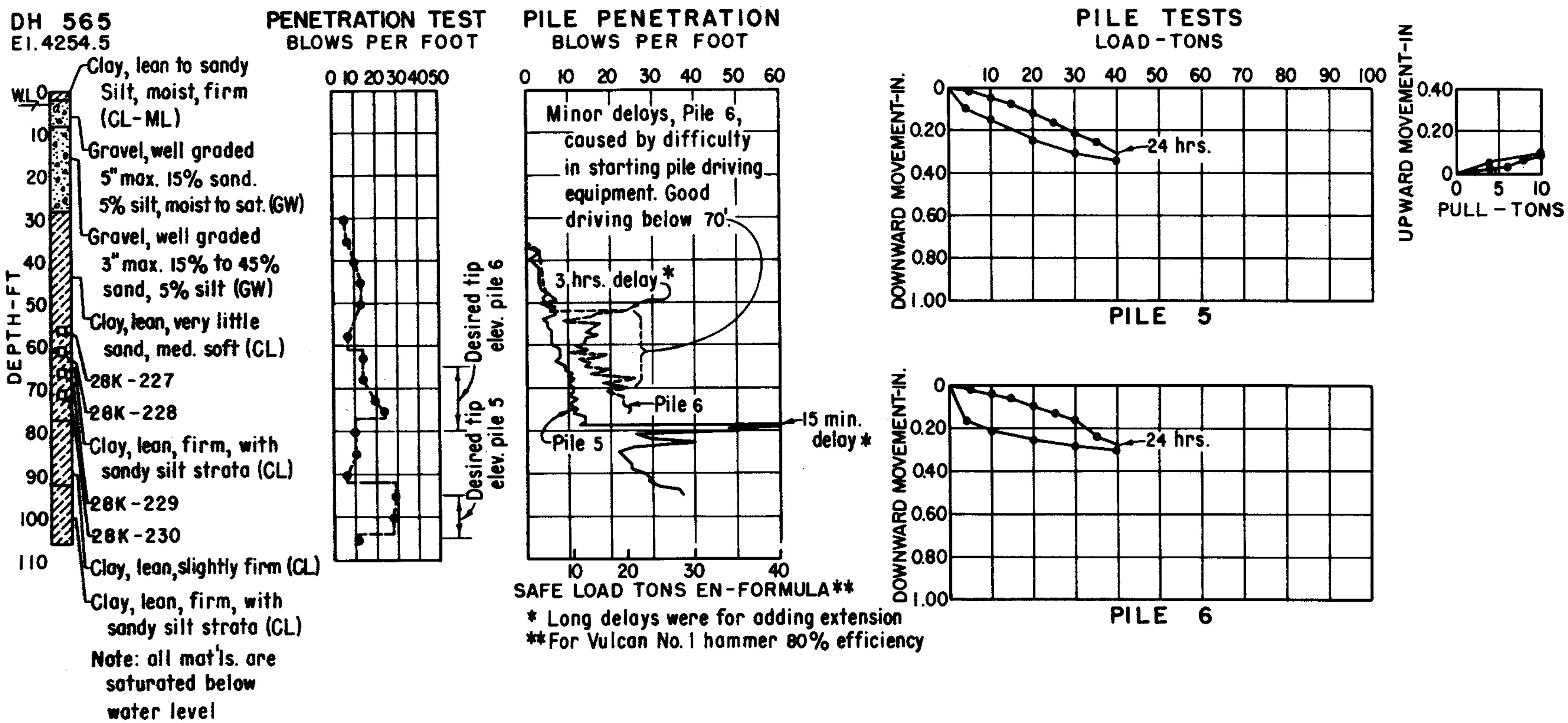

Figure 16.—Penetration resistance, pile driving record, and pile load tests for piles 5 and 6, Pumping Plant No. 2.

the piling could be tested at the proposed foundation elevations and under conditions that represent the excavation without friction of the present overburden affecting the piles.

The contractor used a crane with an orange-peel-type bucket (figure 19) to remove the soil from inside the casings as the casings were placed. At Plant No. 1, the casings advanced under their own weight as the soil inside was excavated. At a depth of about 25 to 30 feet, the outside skin friction increased to the point where the casings stopped moving under their own weight, and it was even difficult to drive them. This problem was solved by not backfilling the area which slumped around the outside of the casings and by jetting around the casings with compressed air.

The contractor found it extremely difficult to install the casings through the 28 feet of gravel at Plant No. 2. The difficulties were increased because the gravel was cemented. The casings were driven by dropping a 1,100-pound breaker ball on a reinforced beam laid

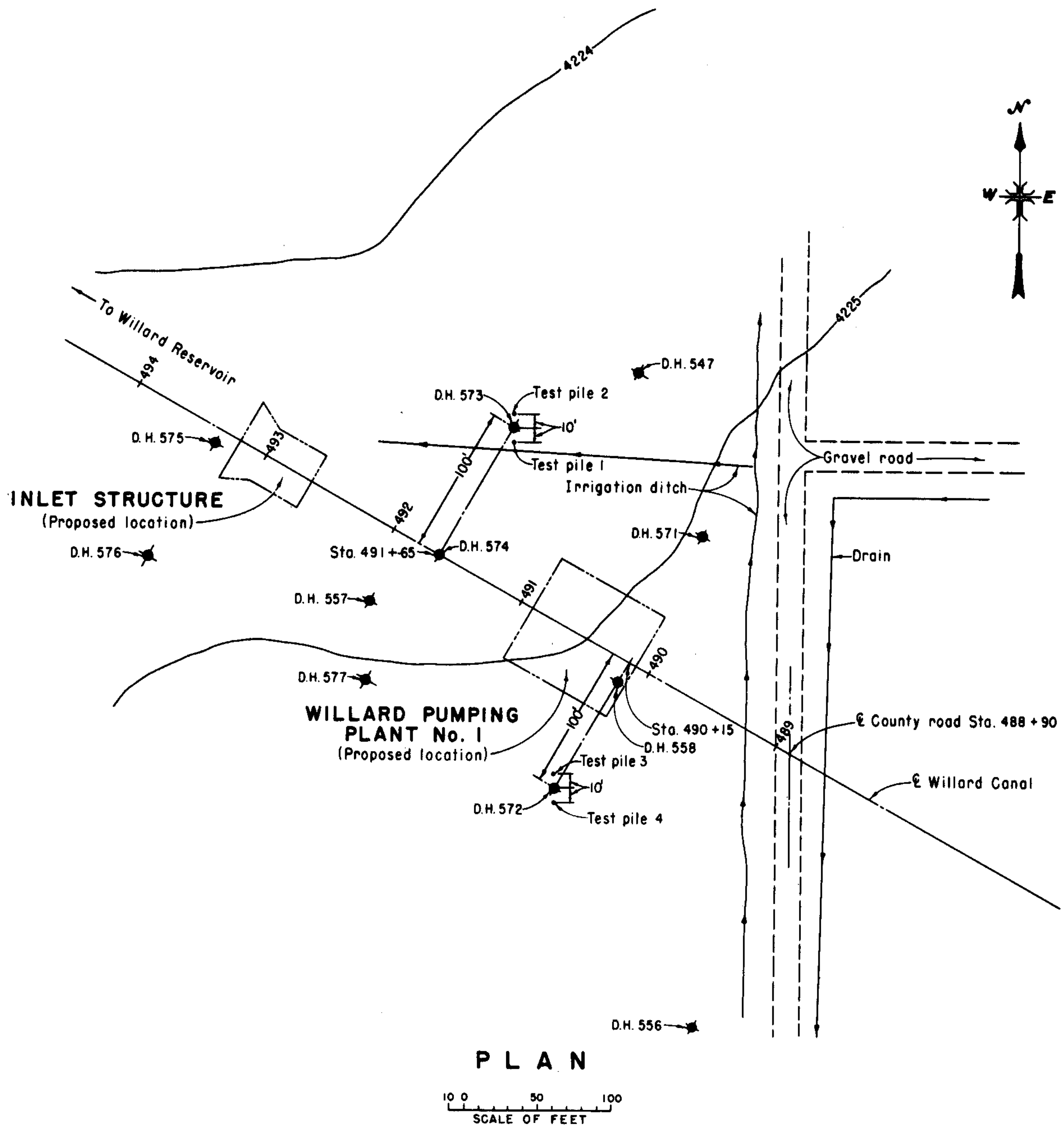

Figure 17.—**Foundation exploration and pile testing plan for Plant No. 1.**

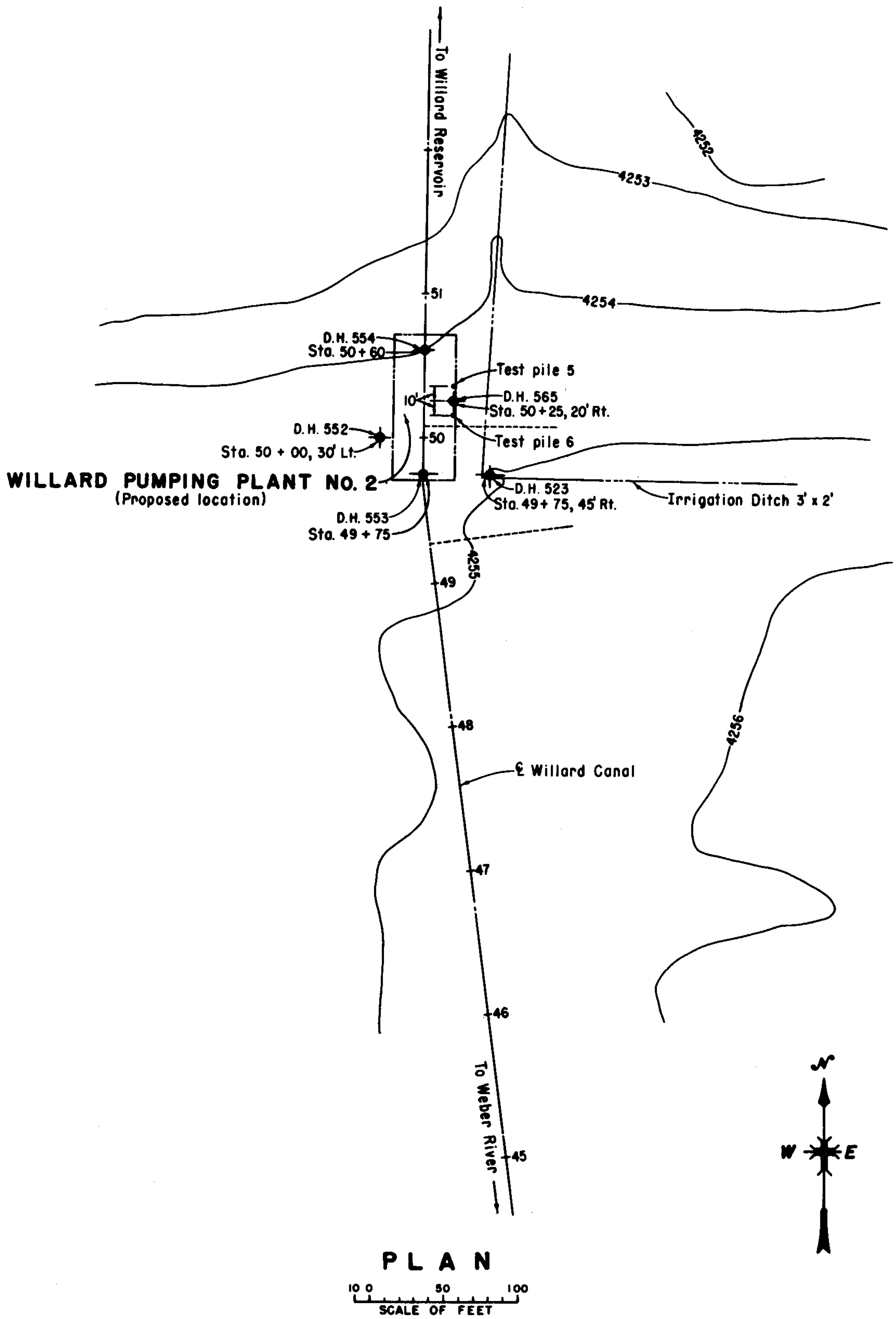

Figure 18.—**Foundation exploration and pile testing plan for Plant No. 2.**

Figure 19.—**Installation of 5-foot-diameter casings.**

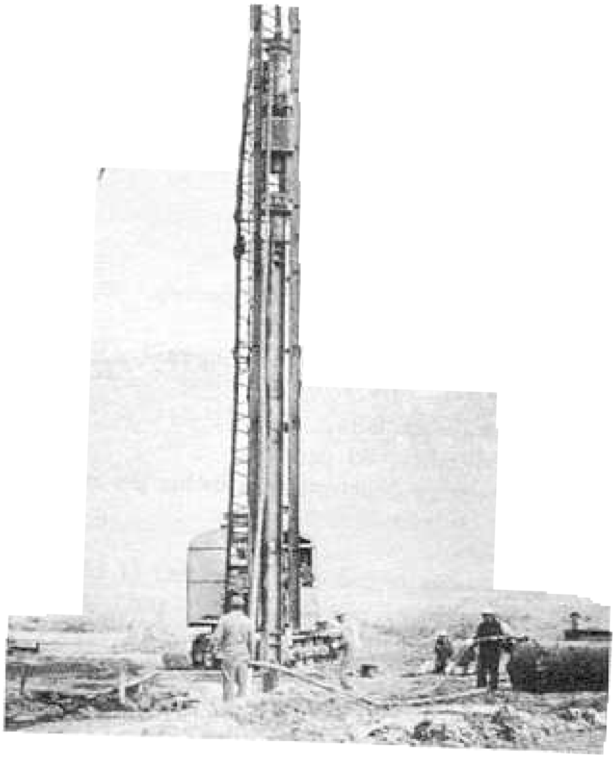

Figure 20.—**Driving test pile 4.**

across the top of the casing. The breaker ball and beam may be seen at left center in figure 19.

Equipment and Procedures

Driving piles—The contractor's pile-driving rig consisted of a crane with a 55-foot boom and extended hanging leads 70 feet long (figure 20). The pilings were driven with a single-acting air hammer (figure 21), equipped with a standard base.

The piles were equipped with steel driving shoes, and the head of each pile was fitted with a steel band to prevent damage during driving.

The contractor elected to use a follower extension to drive the head of the piles down to elevations of 20 to 25 feet below the existing ground elevation. The extension shown in figure 22 was fabricated from a 22-foot length of 12-inch-wide flange beam, 65 pounds per foot. The same extension or a similar one was used to apply the bearing and pull loads to the test piling.

The bracing shown in figure 23 was used to guide the piles at the start of the drive. When the heads of the piles were close to the ground surface, the extension was added, and driving was continued to the specified driving resistance.

The design driving resistance of 40,000 pounds was

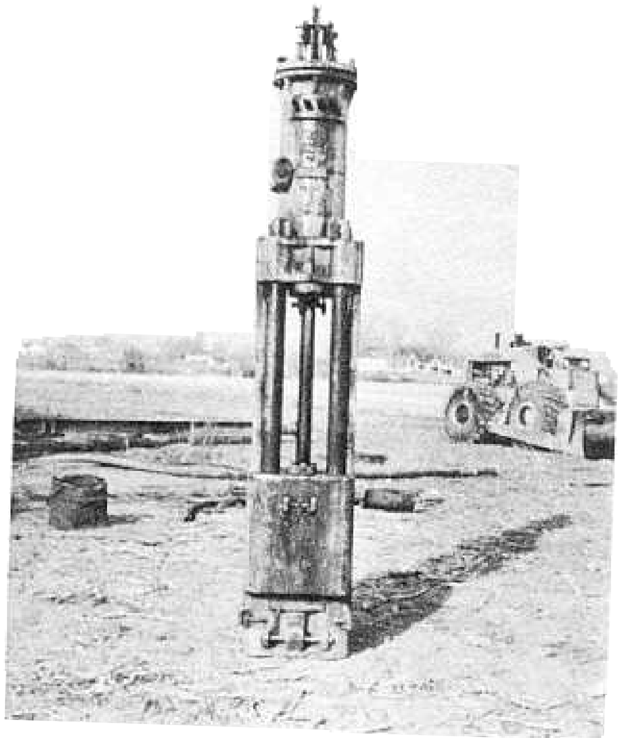

Figure 21.—**Pile driving hammer.**

computed by the formula given in the specifications to be 24 blows per foot with the Vulcan No. 1 hammer. Excessive driving resistance of 80,000 pounds was computed to be 60 blows per foot by the same formula. The specified formula is as follows:

$$P=\frac{2(WH)E}{S+0.1}$$

where:

P=bearing resistance in pounds, 40,000 or 80,000;

W=weight in pounds of striking part of hammer, 5,000;

H=stroke or fall in feet, 3;

E=efficiency, 80 percent;

S=average penetration in inches per blow for at least 10 to 20 blows.

A continuous record of the number of blows required to drive each pile was obtained and is shown in figures 14, 15, and 16.

Bearing tests—Heavy construction equipment was used as the reaction load for all the vertical bearing tests. Two tractors with dozer blades and a crane supplied the necessary weight for the 100-ton tests on piles 1 and 2 (figure 24). As an added precaution, 1¼-inch reinforcing bars were placed over the reaction beam and welded to the 5-foot-diameter steel casings for the tests on piles 1 through 4 (figure 25). The reaction load for piles 3 through 6 was supplied by two tractors with dozer blades, as shown in figure 26.

The bearing tests were performed according to ASTM Test Designation D1143–57T, "Tentative Method of Test for Load-Settlement Relationship for Individual Piles under Vertical Axial Loads". The test loads were applied to the piles with a 100-ton-capacity hydraulic jack in 5-ton increments. All settlement readings were made on a 5½-inch by 5½-inch by ¼-inch steel plate, which was attached to the top end of a ¾-inch pipe. This pipe extended from the ground surface down to a lag screw in the pile head. The tip of the dial indicator gage (4-inch travel, 0.001-inch graduations) rested on the steel plate. An engineer's scale was also attached to this plate and was read with a transit which was referenced to a level rod and target, as shown in figure 26. Both the level rod and the tripod legs of the transit were buried in the ground about 18 inches. A typical installation of the dial gage and engineer's scale is shown in figure 25. The gage in the background measured the movement of the pile head while the tip of the gage rested on the top of the ¾-inch pipe. The gage in the foreground is resting on a bracket attached to the hydraulic jack, which moved

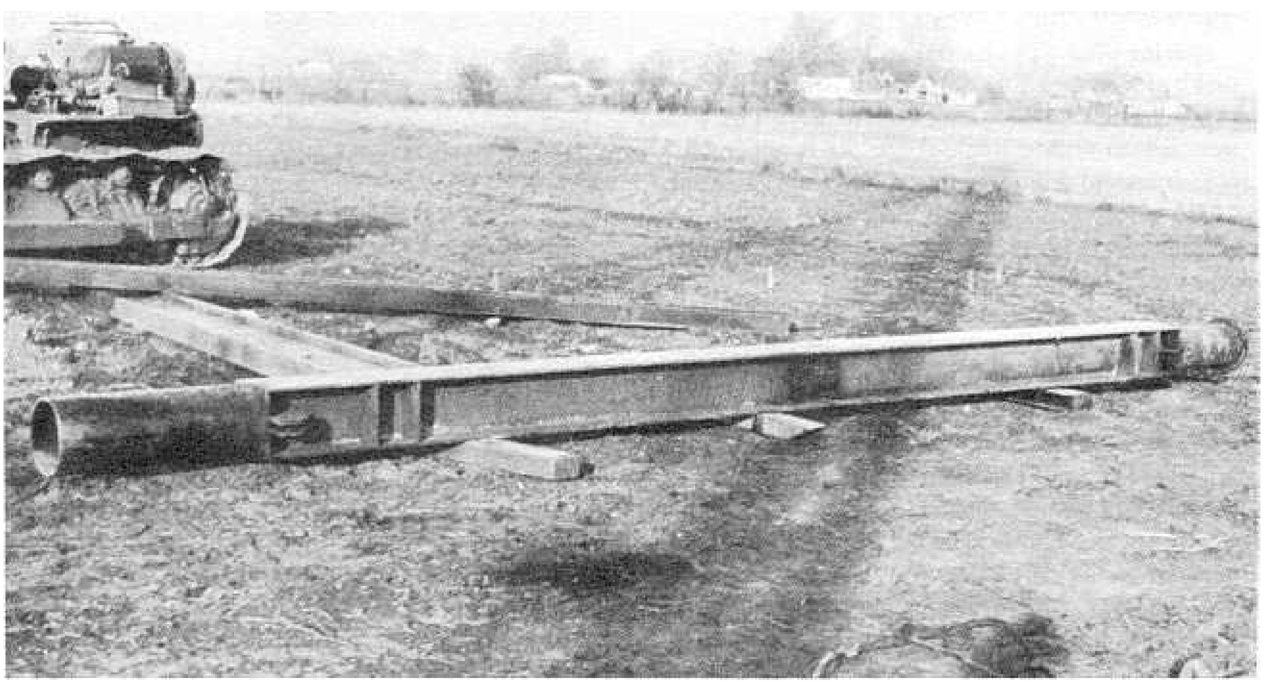

Figure 22.—**Pile driving extension.**

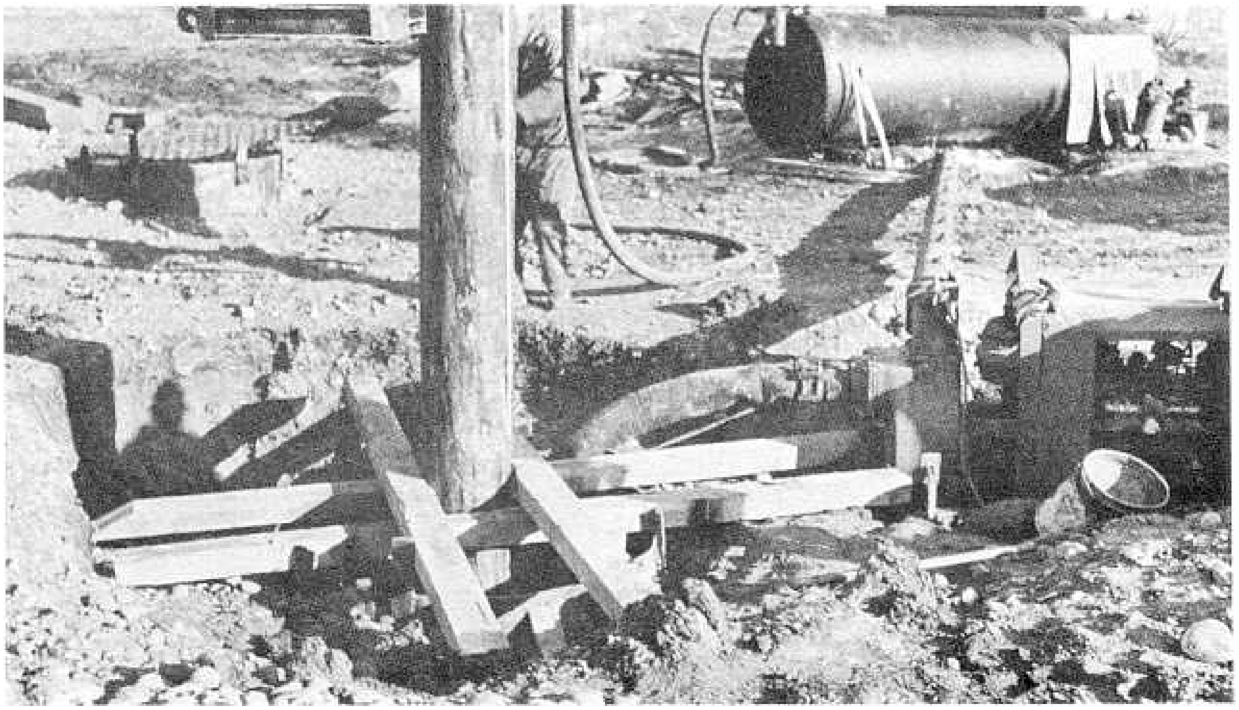

Figure 23.—**Guide bracing for test piles.**

with the extension such that this second gage measured the vertical movement of the upper end of the extension. This measurement was taken on the first few tests, then discontinued, as it included the takeup of slack in the extension.

The dial gages were supported by a reference beam which was independent of the reaction load and the test pile. The reference beam for the tests on piles 4, 5, and 6 was an 18-foot length of 4-inch channel, 7.7 pounds per foot, welded to two steel stakes driven into the ground about 2½ feet. During the testing of pile 4, the entire area within about 10 feet of the casing settled and caused the reference beam to settle also. To correct this, a 3-inch channel, 5.7 pounds per foot, and the 4-inch channel were spliced to form a 34-foot beam. The bearing area of the stakes, to which the beam was attached, was increased by welding 8- by 8-inch steel plates to the lower end of the stakes. This 34-foot reference beam was used for the tests on piles 1, 2, and 3 (figures 25 and 26).

Another instrument which was used on the bearing tests of piles 5 and 6 is the pencil device for recording lateral movement of the pile extension shown in figure 27. The device was useful to detect lateral movement of the reaction beam when the limit of the reaction load was reached and the load shifted slightly. The use of this device was discontinued when the contractor started attaching the reaction beam to the casing with 1¼-inch reinforcing bars.

Pull tests—Figure 28 illustrates the general test set up for the pull tests except that, for the tests on pile 1, two 18-inch **I**-beams, 54.7 pounds per foot, placed side by side were used.

The pull tests were performed according to the procedure given in the specifications. All loading increments and unloading decrements for the 20-ton pull test on pile 1 were double those specified for the 10-ton pull tests. The 30-ton failure pull test on pile 1 was applied in 5-ton increments at 5-minute time intervals. The gage and level readings were made in a manner identical to those taken for the bearing tests.

The pull force was transmitted to the pile by a 12-inch-wide steel band. The band was formed by two semicircular pieces bolted together with 1¼-inch bolts and recessed into the butt of the pile. These were also spiked to the pile with six mine rail spikes. The pulling bars shown in figure 28 were attached to the follower, which in turn was attached to the band on the pile by a similar set of pulling bars.

Vane shear tests—To investigate the possible effect of remolding caused by pile driving, vane tests were performed at Plant No. 1 in the vicinity of piles 1 and 2. These tests were made at depths of 45 and 60 feet before driving piles, and again within 24 hours after

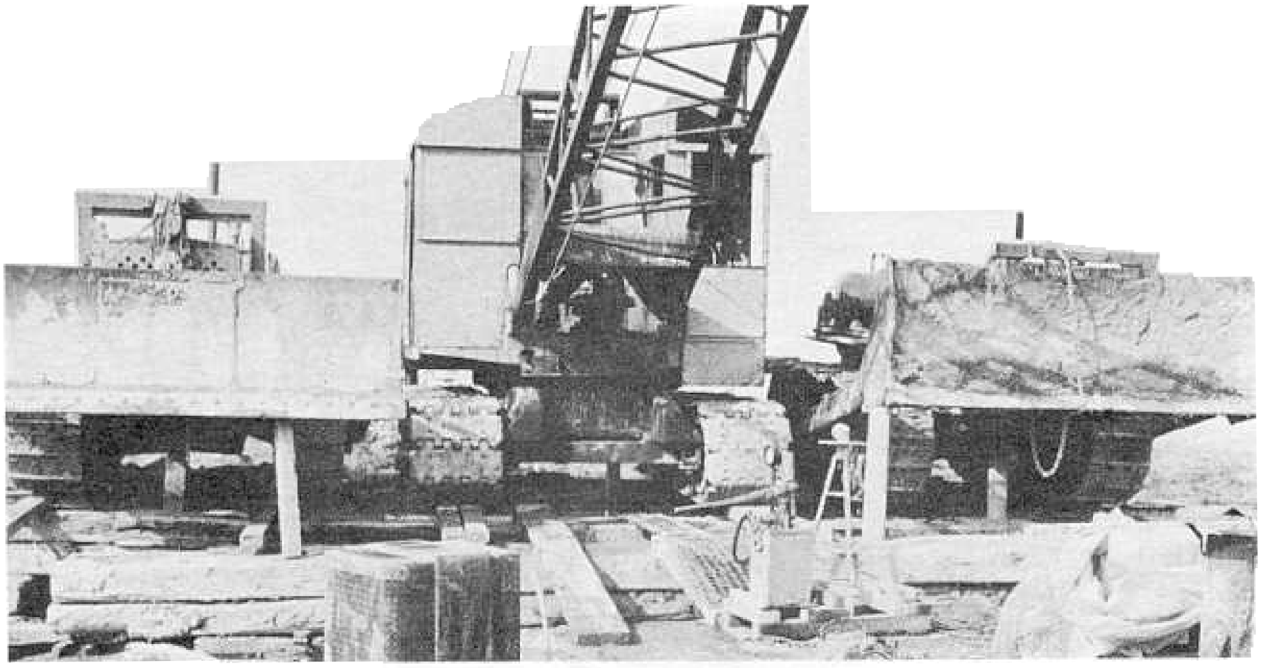

Figure 24.—Equipment for 100-ton reaction load.

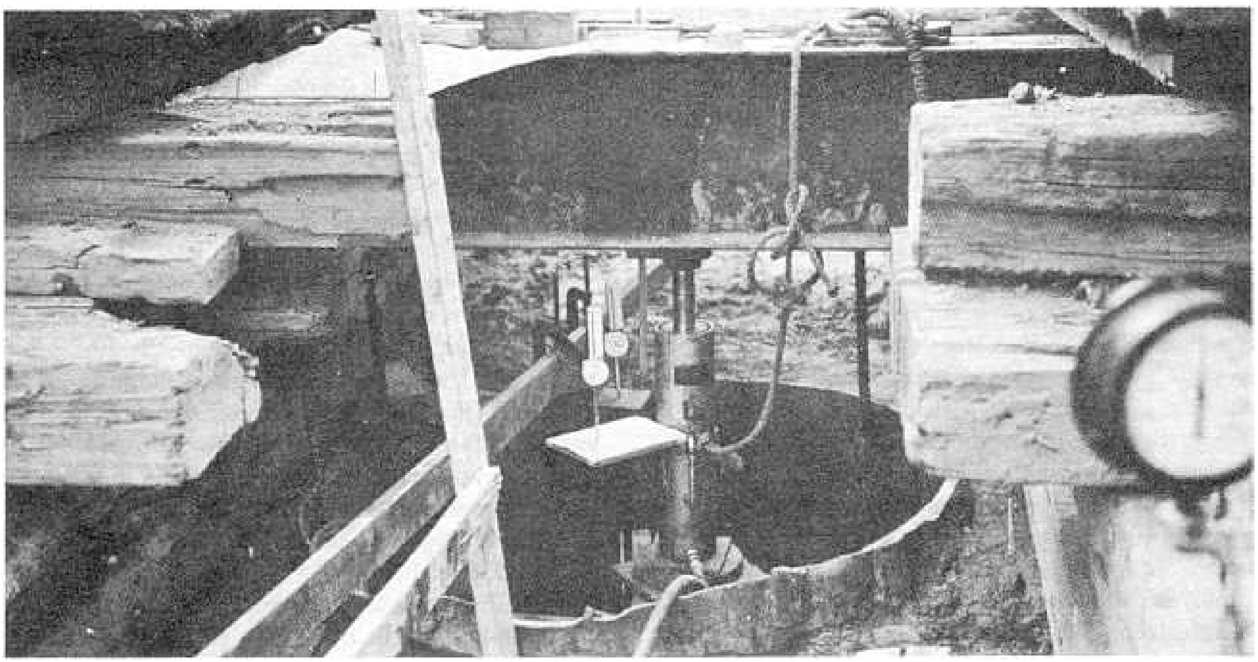

Figure 25.—**Test set-up on pile 3.**

driving piles 1 and 2. The after-driving vane tests were made within 3 feet of each pile and at the same depths. After the testing of piles 1 and 2 was complete, a third series of vane tests was made within 1½ feet of piles 1 and 2 and at the same depths as previous vane tests. These tests were performed about 6 weeks after the piles were driven. All the tests were conducted according to Designation E–20, "In-place Vane Shear Test", First Edition of the Bureau of Reclamation *Earth Manual,* with one exception. The remolded portion of the test was repeated several times with the vane being rotated 360° between each repetition.

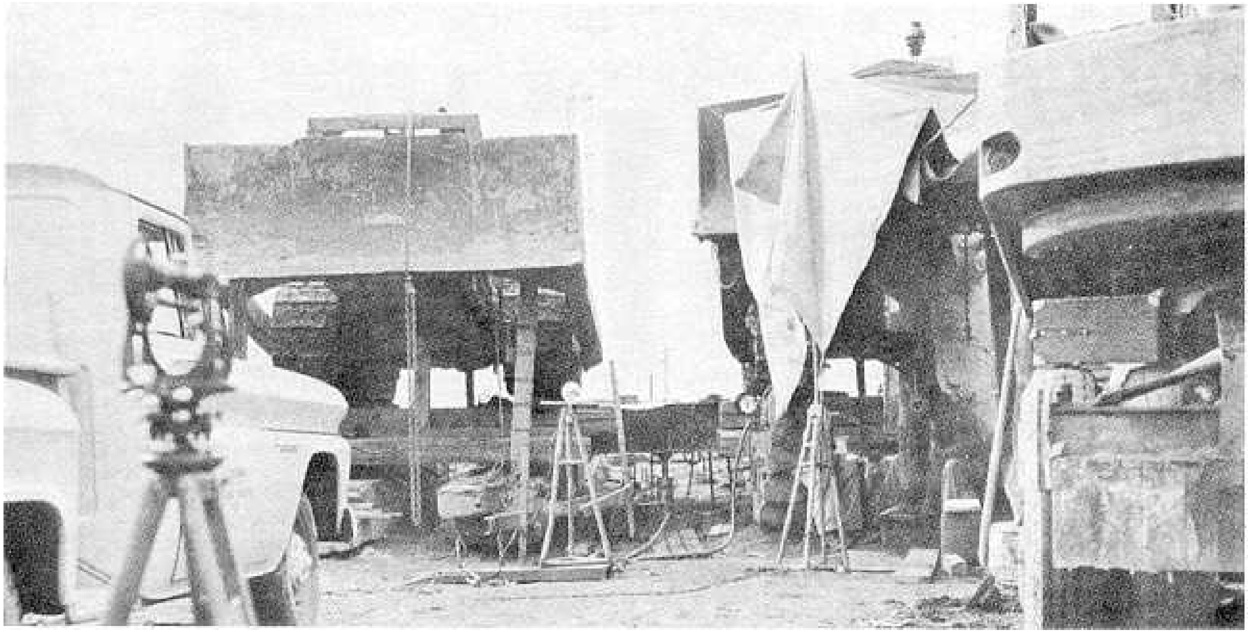

Figure 26.—Loading for piles 3 through 6.

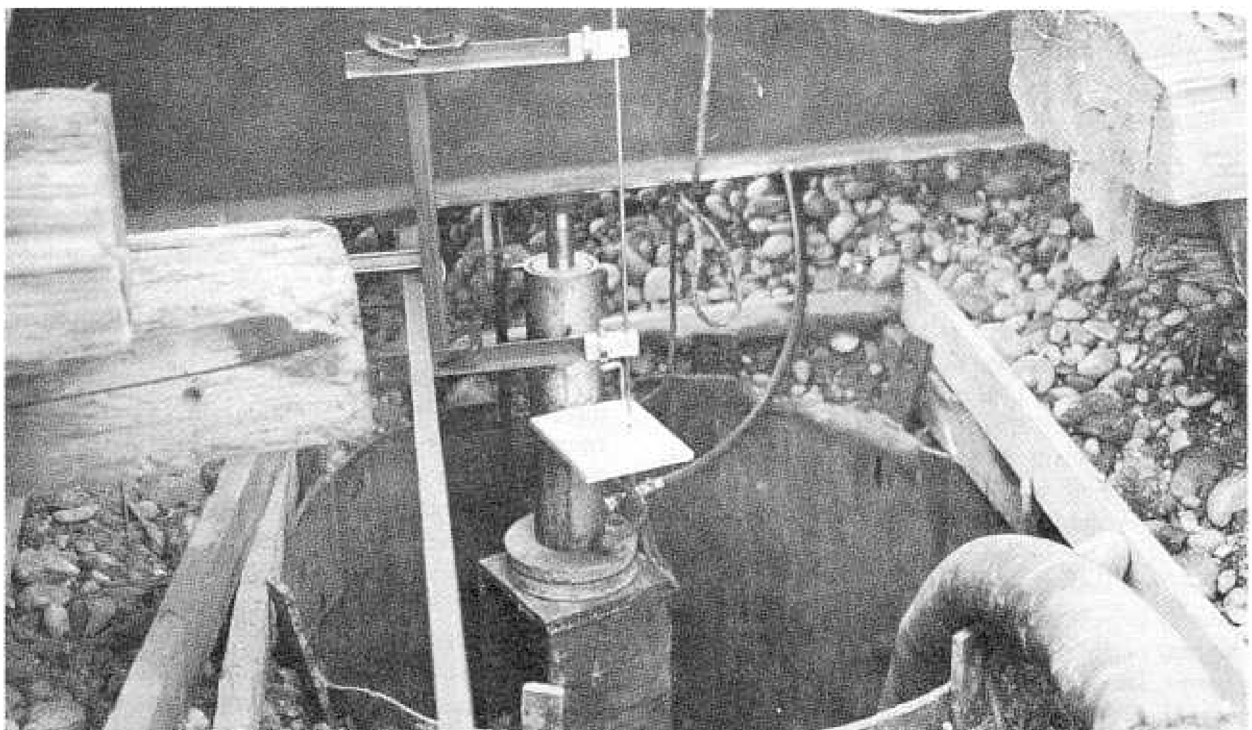

Figure 27.—Lateral movement measuring device.

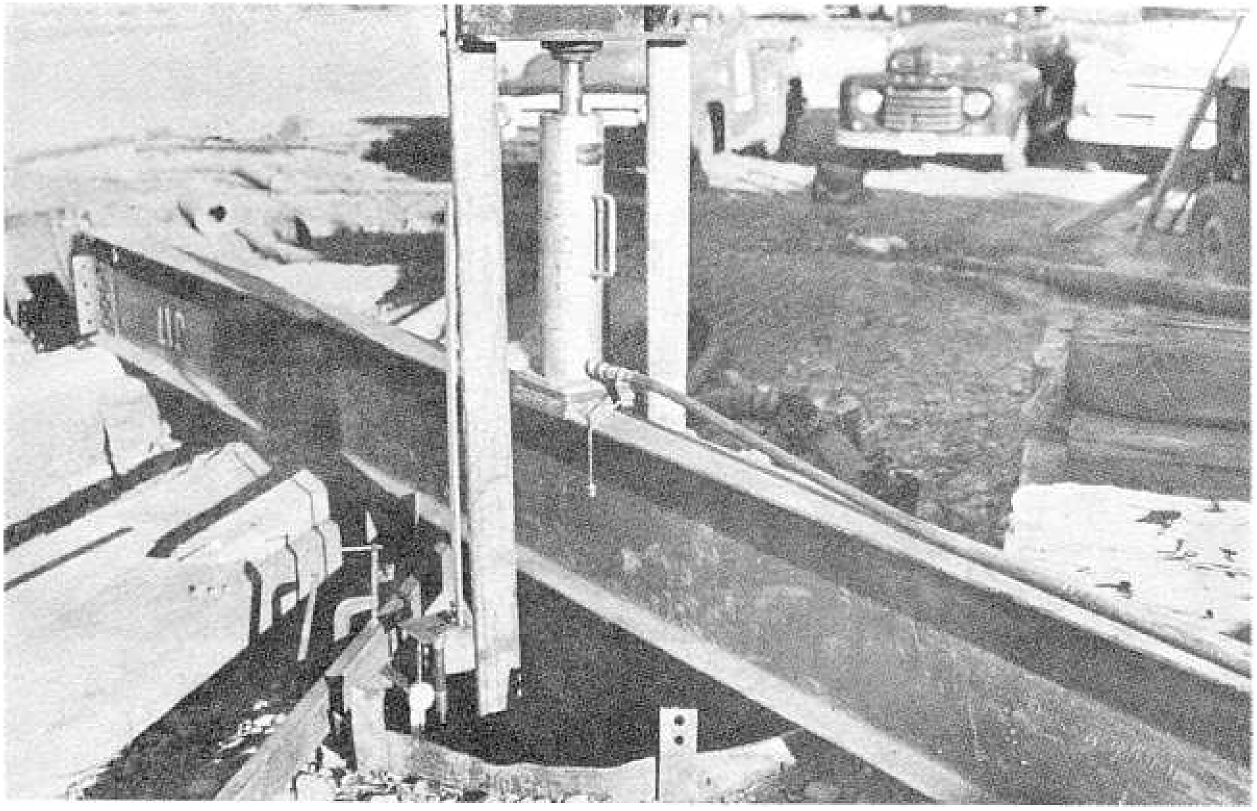

Figure 28.—**Pull test equipment on pile 5.**

Test Results

Pile driving—At each of the three drill hole locations selected for tests, two piles were driven, a long pile and a pile driven to a tip location in the upper strata of material showing relatively high values by the penetration resistance test. The depths of drive specified were to be within certain limits. The driving records given in figures 14 through 16 show the depths to which the piles were driven.

Pile 6 at Plant No. 2 (figure 16) was driven to nearly the limiting depth but also reached a bearing capacity of 20 tons by the EN formula. Some difficulty was encountered during the driving of pile 6 because this was the first pile driven and because it was a pile with a relatively large tip of more than 8 inches in diameter. This may partly contribute to a relatively high bearing capacity. The longer pile, No. 5, at this location, only reached a bearing capacity of about 11 tons by the EN formula at the same depth as pile 6, but when driven to a depth of 95 feet into the lower, relatively firm clay stratum, a value of 28 tons by the EN formula was obtained.

Piles at Plant No. 1 were likewise driven to two different depths, as shown in figures 14 and 15. However, these piles encountered occasional layers of dense sand. The driving records of piles 3 and 4 were similar, and the minimum trends of the curves indicate bearing capacity at 90-foot depth, by the EN formula, of from 16 to 30 tons, and a 40-ton value of bearing capacity was reached for the long pile (pile 3) when it approached, or just entered, the deeper firm layer. The driving record of piles 1 and 2, shown in figure 15, indicates slightly lower minimum driving resistance trends than that shown by piles 3 and 4. The penetration resistance test values, shown in figure 15, were also less for this location than those shown in figure 14.

With the exception of the high-driving resistance which developed for a short distance after each delay in driving, excessive driving resistance was developed only on pile 3, as shown on figure 14. This was within the range where jetting could be used at the direction of the contracting officer. Since the pile tip was very near the desired depth, it was decided to stop driving at 102.1 feet and not use jetting. Pile 3 drove as if the tip was already in the silt and fine sand stratum, which the log indicated was at 105 feet. Since pile 3 was the only one which developed excessive driving resistance, no jetting was used for any of the piles.

TABLE 4.—Data on test piles

Pile	Designation	Plant site	Depth of tip below surface (feet)	Elevation		Total timber length (feet)	Length embedded in soil (feet)	Location
				Tip (feet)	Top of timber (feet)			
5	Long	2	95.2	4159.3	4244.3	85.0	65	DH 565
6	Short	2	76.5	4178.0	4248.0	70.0	46	DH 565
3	Long	1	102.1	4122.4	4207.4	85.0	67	DH 572
4	Short	1	89.8	4134.7	4294.7	60.0	55	DH 572
1	Long	1	110.0	4114.5	4199.8	85.3	75	DH 573
2	Short	1	88.3	4136.2	4196.4	60.2	53	DH 573

Bearing tests—None of the bearing tests produced rapid progressive settlement or any indication of failure by the criteria given below. There are many arbitrary and empirical rules for failure criteria, but none have been established as a standard. These rules have been established by various building codes for the purpose of determining allowable working loads on the basis of pile tests. The criteria are of two basic types:

1. Failure is considered at the load where settlement increases in appreciably greater proportions than load increases, and is generally determined by observing a plotted curve of the test results.

2. The ultimate load is considered when settlement reaches certain maximum values based on several types of observations, such as:

   a. Total movement of more than 1 inch should never be allowed (New York City Code). However, this alone is not sufficient, and additional criteria are recommended.

   b. Total movement should not be more than 0.01 inch per ton of test load (Department of Public Works of California, Pacific Coast Uniform Building Code, and Chicago Building Code).

   c. Net settlement after load is released should not be more than 0.25 inch (American Association of State Highway Officials and New York State Department of Public Works).

A brief summary of the bearing test results is as follows:

1. The load tests at Plant No. 2, shown in figure 16, were conducted to the specified total load of 40 tons. For both piles, maximum movement of the top measuring point on the timber pile was 0.35 inch or less, and no permanent set of the pile after the load was released was observed.

2. Pile tests 3 and 4 at Plant No. 1, shown in figure 14, were also conducted to the specified load of 40 tons. Pile 3 was then tested further to 60 tons and then reloaded to 80 tons. Pile 4 was reloaded to 60 tons. These tests showed that no permanent set occurred for either pile after the loads were removed. All test results were within the limitations of the previously mentioned failure criteria, except the 80-ton test on pile 3, which had slightly more than 0.8-inch maximum movement, or 0.01 inch per ton of load, but showed no permanent set upon release of this load. It must be remembered that the movement of the upper part of pile 3 may be greater than that occurring in normal test piles, because this pile extended above the bottom of casings and had considerable length (18 feet) that was not embedded in soil. In any event, the 60-ton load test did not show signs of failure, and none of the load tests showed any permanent set.

3. Pile tests 1 and 2 at Plant No. 1, shown in figure 15, were conducted to loads of 40, 60, and 100 tons. The 60-ton load was maintained on piles 1 and 2 for 5 days to observe movements with respect to a long period of time. The results of all these tests were well within the limitations of the previously mentioned failure criteria. During the long-time tests on piles 1 and 2, it was noted that fluctuations occurred in pile movement as a result of temperature. Temperature observations were made during the bearing tests and it was noted that, as temperature raised, movement measurements increased. Conversely, as temperatures dropped, movement measurements decreased. Where movements are very small, as they were in these tests, this comparison became important.

The performing of the pile tests at Plant No. 1 to loads higher than 40 tons was a precautionary measure to take care of the possibility that pile penetration might disturb the soft, and possibly sensitive, soil which occurs between depths of 40 and 70 feet below present ground surface, causing this remolded soil to settle and load the piles an additional amount by dragging on the piles. Recognizing that this may occur, pile tests were performed to 80 and 100 tons. Penetration resistance test values in the soft clay strata varied from 0 to eight blows per foot.

The tests at Plant No. 2 were made as originally specified and to loads not higher than twice the design load, or 40 tons. However, it is believed that the problem of sensitive clay in the upper stratum does not exist at this site. The penetration resistance test (figure 16), between the depths of 30 and 60 feet, indicates material with more than five blows per foot and as much as 12 blows. This clay is described as being medium to stiff, with unconfined compressive strengths of from 10 to 20 psi. Bureau studies would indicate a probable in-place vane shear strength of about 7 to 15 psi.

Furthermore, there were obtained from Drill Hole DH–565, at Plant No. 2, four undisturbed samples at the depths indicated on the logs in figure 16. The results of laboratory tests on these samples are shown in table 5. Also shown in table 5 are the undisturbed sample data of two samples at depths of 43 and 48 feet which were from holes within 600 feet of Plant No. 1. The one set of test data obtained from Sample 28K–227, which was taken at a depth of 55 to 57 feet in DH–565 at Plant No. 2, shows that the in-place moisture content is 5 percent less than the liquid limit, indicating no more than a medium sensitive clay when it is considered that all clays have some degree of sensitivity. In contrast to these data, the two samples tested from near Plant No. 1 showed that the in-place moisture content was either near to, or above, the liquid limit. This could mean that the Plant No. 1 soil is more sensitive, which is also indicated by the zero value of the penetration resistance and relatively low shear strength that have been observed.

Pull tests—The test data of the four pull tests are presented graphically on figures 15 and 16. Gage readings were used for plotting all except the 20-ton pull test on pile 1. After the test was started, it was discovered that the stem of the dial gage was sticking, so the level readings were used for plotting the 20-ton pull test on pile 1.

The data for the 10-ton pull test on pile 2 presented an unusual looking curve, but nothing in the test setup, equipment, or procedure offered an explanation.

The 10-ton pull test on pile 5 indicated less than 0.10-inch uplift and practically no permanent set (0.009 inch). The 20-ton pull test on pile 1 produced an uplift of 0.14 inch and a permanent set of 0.04 inch.

During the failure pull test on pile 1, the pile did not pull out. The bottom set of pulling bars failed as an attempt was made to increase the pull from 30 to 35 tons.

Vane tests—Observation of the vane test results (figure 9 and table 6) reveals that driving and testing of the piles caused little or no changes in the vane test values. In some tests, the after-driving values were even greater than the before-driving values. The small differences are well within normally anticipated differences for vane shear tests. Although most tests showed only a slight reduction in shear strength in the undisturbed portion of the curves, similar slight reductions also occurred in remolded portions of the curves. For all the tests at 45-foot depth, the undisturbed strengths ranged from 7.6 to 4.6 psi and the remolded strengths ranged from 4.0 to 1.6 psi, and at 60-foot depth the undisturbed strengths ranged from 7.0 to 6.4 psi and the remolded strengths ranged from 2.8 to 1.6 psi. It is important that the plots in figure 9 of the vane test observations be noted to see the similarity between the before tests and the after tests and the occasions when some of the after tests showed slightly higher strengths than the before tests.

TABLE 5.—*Data on undisturbed samples*

Hole No.	Laboratory sample No.	Depth (feet)	In-place dry density (pcf)	In-place moisture content (percent W)	LL (percent)	PL (percent)	PI	Liquidity index $LI=\frac{W-PL}{LL-PL}$
565—Plant No. 2	28K–227	55–57	93.1	28.1	33	18	15	0.67
565—Plant No. 2	28K–228	60–62	90.8	28.7	...	...	...	...
565—Plant No. 2	28K–229	65–67	83.2	34.9	...	...	...	...
565—Plant No. 2	28K–230	70–72	84.0	32.3	...	...	...	...
578—Plant No. 1	28K–324	47.6–49.6	74.2	46.8	37	17	20	1.49
					49	20	29	.93
579—Plant No. 1	28K–326	42.6–44.6	67.3	55.2	52	23	29	1.11

TABLE 6.—*Vane test results before and after pile driving—Test piling for Willard Pumping Plants Nos. 1 and 2*

Depth (feet)	Time of test	Friction data	Undisturbed			Remolded A			Remolded B			Remolded C			Remolded D			Remolded E		
		Gage	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi
45	Before driving piles 1 and 2.	1099	1728 1099 629	48	5.8	1419 1099 320	24	2.9	1392 1099 293	22	2.7	1406 1099 307	23	2.8	1389 1099 290	22	2.7	1380 1099 281	21	2.5
45	After driving pile 1..	1050	1599 1050 549	42	5.1	1350 1050 300	22	2.7	1324 1050 274	20	2.4	1332 1050 282	21	2.5	1304 1050 254	19	2.3			
45	After driving pile 2..	1108	1728 1108 620	47	5.7	1392 1108 284	21	2.5	1370 1108 262	20	2.4	1366 1108 258	19	2.3	1347 1108 239	18	2.2	1356 1108 248	19	2.3
45	After testing pile 1..	1081	1579 1081 498	38	4.6	1323 1081 242	18	2.2	1268 1081 187	14	1.7	1265 1081 184	14	1.7	1258 1081 177	13	1.6			
45	After testing pile 2..	1020	1851 1020 831	63	7.6	1455 1020 435	33	4.0	1422 1020 402	30	3.6	1422 1020 402	30	3.6	1393 1020 373	28	3.4	1380 1020 360	27	3.3
60	Before driving piles 1 and 2.	1062	1805 1062 743	56	6.8	1375 1062 313	23	2.8	1293 1062 231	17	2.1	1274 1062 212	16	1.9						
60	After driving pile 1..	1092	1868 1092 776	58	7.0	1370 1092 278	21	2.5	1286 1092 194	14	1.7									

TABLE 6.—*Vane test results before and after pile driving—Test piling for Willard Pumping Plants Nos. 1 and 2*—Continued.

Depth (feet)	Time of test	Friction data	Undisturbed			Remolded A			Remolded B			Remolded C			Remolded D			Remolded E		
		Gage	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi	Gage	Ft/lb	psi
60	After driving pile 2..	1091	1787			1355			1283											
			1091			1091			1091											
			696	53	6.4	264	20	2.4	192	14	1.7									
60	After testing pile 1..	1062	1816			1338			1265											
			1062			1062			1062											
			754	57	6.9	276	21	2.5	203	15	1.8									
60	After testing pile 2..	1033	1733			1270			1218			1212								
			1033			1033			1033			1033								
			700	53	6.4	237	18	2.2	185	14	1.7	179	13	1.6						

NOTE: The columns marked Remolded A, B, C, D, and E refer to repetitions of the remolded portion of the vane tests which were performed to thoroughly evaluate the remolded strength of the soil.

It was initially recognized that there could be a possibility that remolding effects would exist at Plant No. 1. However, the vane tests and other laboratory tests did not indicate this to be an appreciable amount. In comparing this characteristic to the results at Plant No. 2, it is believed that this problem is even less at Plant No. 2 because of the greater firmness and lower sensitivity. In other words, the clays at Plant No. 2 were considered to be no more of a problem than any other normal clay soil through which piles are usually driven.

Pile Heaving

During driving of piles in the softer saturated clays of the Willard Pumping Plant foundations, heaving of the surfaces was expected because the soil displaced by the piles and surrounding soil had not had time to consolidate. This is not considered unusual for the type of soils encountered at these pumping plants, and was observed during driving of test piles at the site of Plant No. 1. The observers estimated the amount of soil heaving around a test pile driven inside the 5-foot-diameter casing to be about ½ foot. These observations were difficult because of the soft materials in the bottom of the casing, and many observations were misleading because of the inflow of material into the bottom of the casing, resulting from the dewatering and loosening of the soil during the casing installation.

A heaving of the foundation surface was expected when the piles were driven. The heaving caused by the later piles driven may actually lift previous piles driven. Therefore, piles were carefully checked for elevation.

Application of Driving Formulae

All literature on pile driving emphatically states that pile driving formulae have no relationship to pile capacity when driving is in a clay. In the case of Willard Pumping Plant, the formulae were recommended as a guide to make the foundation piles similar to those that have been satisfactorily tested.

The reason that formulae are not applicable is best illustrated by Chellis on page 27 of his publication, "Pile Foundation". During driving, resistance is primarily end resistance; after driving, the pile sets up, and its support is gained by friction along the pile before its end is stressed and thus the end resistance shown by driving is not being utilized, although it apparently is still available. Although Chellis indicates no end bearing in his example, it is believed that even the short piles tested at Willard Plant No. 1 are partly supported by end bearing on firm sand strata with high penetration resistance test values. If the end-bearing viewpoint is not accepted for the case of Willard Plant No. 1, these piles will have at least 10 and as much as 20 feet of their lower end length embedded in soils with considerably greater stiffness than the soft clay above.

Maximum Design Loads

Indications from the tests were that piles would stand the maximum load for timber piles of 25 tons, with 20 tons dead load being a reasonable design value:

1. A condition assuming no drag effect on piles due to remolding of the soft clay was analyzed. If a 25-ton load is placed on each pile, the safe capacity of a single pile in the group with respect to the soil when group effect is considered to be 70-percent reduction is:

$$\frac{25}{0.70}=35.7 \text{ tons.}$$

If a safety factor of 2 is used, the load test must show no failure for a maximum load of $2\times35.7=71.4$ tons. Piles 3 and 4 were tested to 60 tons without any indication of failure, and piles 1 and 2 were tested to 100 tons with no failure.

2. Although test data indicated that detrimental remolding of the soil would not occur due to pile driving, the condition assuming maximum drag on the pile from the soft clay was analyzed. For the worst condition, this was assumed to be the total buoyant weight of the clay for a prism of soil 4 feet square and 35 feet deep (see "Foundation Engineering" by Peck, Hanson, and Thornburn).

$$\text{Drag per pile}=\frac{A\gamma' H}{n}\;\frac{(4\times4)(115-62.4)(35)}{1}$$

$$=\frac{16(52.6)(35)}{1}=29{,}500 \text{ pounds}$$

$$=15 \text{ tons.}$$

If 25 tons were placed on the pile, the safe capacity of a single pile would be $25+15$ and, with adjustments for the group effect, would become:

$$\frac{25+15}{0.70}=57 \text{ tons.}$$

No piles showed failure, and piles 1 and 2 were loaded to 100 tons with no indication of failure. Again, it was believed the pile driving in this material would not result in completely remolding the soil. Even if it did, the piles would have a safety factor against failure of at least $\frac{100}{57}=1.75$.

SECTION IV.—CONSTRUCTION SETTLEMENT TESTS

Synopsis

This section of the report relates the settlement studies at Willard Pumping Plants Nos. 1 and 2 to construction activities and natural phenomena, including two earthquakes that occurred during construction of the plants.

Plant Construction

Based on the favorable results of the pile load tests, the plants and their foundations were designed such that the pile foundations and the necessary excavation would be as shown in figure 29. At both plants, the piles were driven to depths of about 80 feet below the bottom of the plant, transferring the structure load to the more competent firm clay at about 105 to 120 feet below ground surface. Pile driving and plant construction at both plants were begun early in 1962. After completion of the pile foundation and the preparation for the concrete base, settlement benchmarks were installed on augers in the subsoils between piles and in the concrete base floors of each plant, figure 30. Elevations of the benchmarks were determined by reference to established benchmarks located outside the construction area.

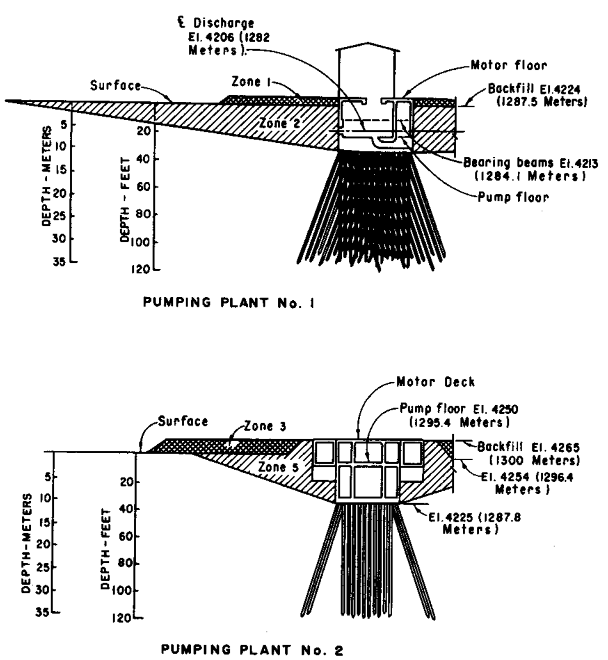

Figure 29.—Sections of pumping plants and backfill.

Construction Sequence

Excavation and pile driving at the site for Plant No. 1 were completed, and construction was begun at this plant on May 28, 1962. The sequence of construction events is shown in table 7, and structure and backfill elevations are shown in figure 29.

Excavation and pile driving at the site for Plant No. 2 were completed and construction of the plant was begun on March 30, 1962. The sequence of construction events is shown in table 8, and structure and backfill elevations are shown in figure 29.

Settlement Observations

As previously discussed, settlement observation points and benchmarks were placed on earth augers

TABLE 7.—*Sequence of construction for Plant No. 1*

Item	Date begun	Date completed	Elevation	
			Feet	Meters
Plant construction:				
Pump floor...	May 28, 1962	Aug. 1, 1962	4201	1280.5
Bearing beams.....	....	Oct. 1, 1962	4213	1284.1
Motor floor...	....	Dec. 20, 1962	4229	1289.0
Superstructure...	Jan. 9, 1963	Mar. 14, 1963	....	......
Backfill:				
Zone 2......	Aug. 30, 1962	Sept. 13, 1962	4206	1282.0
		Oct. 6, 1962	4213	1284.1
		Nov. 23, 1962	4224	1287.5
Zone 1......	Feb. 25, 1963	May 20, 1963	4229	1289.0

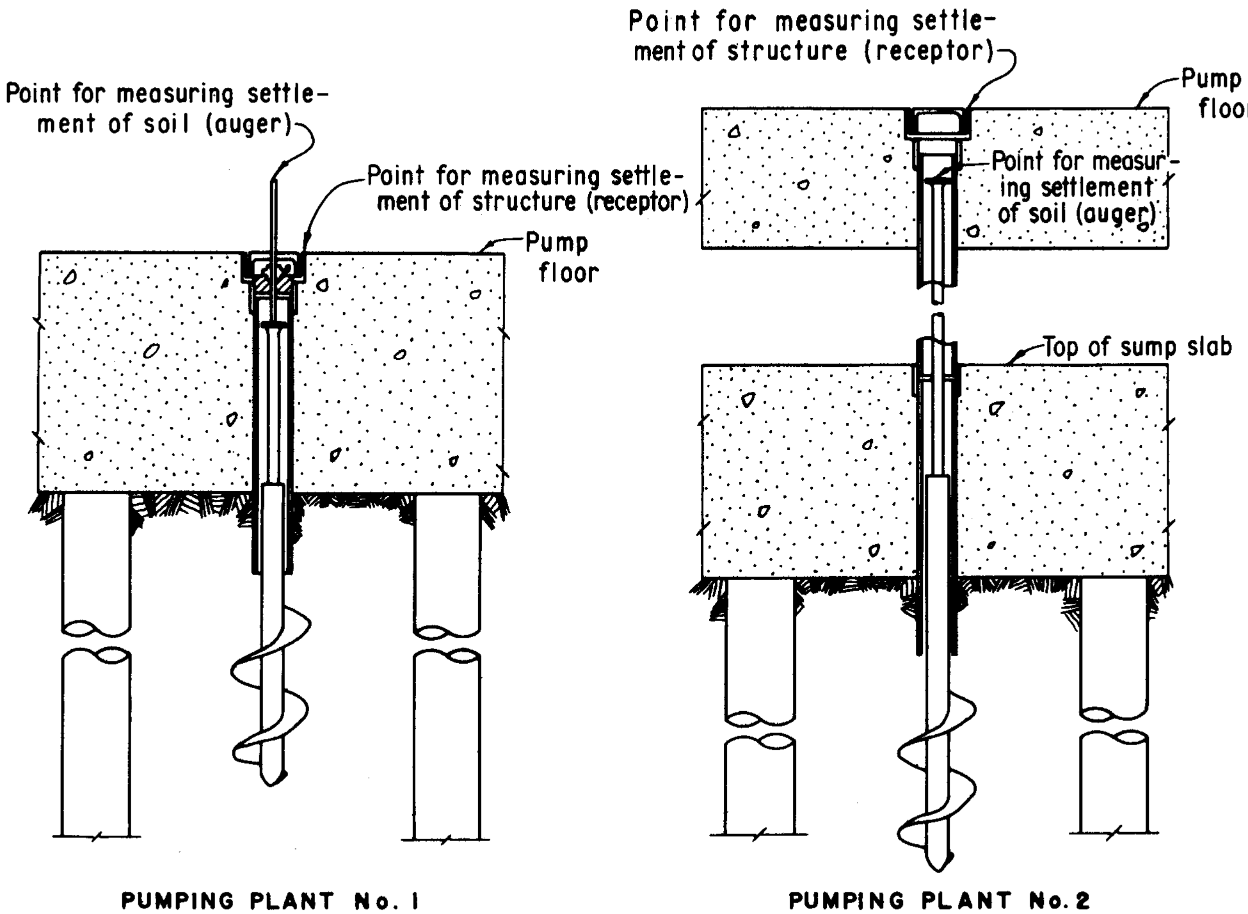

Figure 30.—**Typical settlement observation point installations.**

TABLE 8.—*Sequence of construction for Plant No. 2*

Item	Date begun	Date completed	Elevation	
			Feet	Meters
Plant construction:				
Bottom slab	....	Mar. 28, 1962	4225	1287.8
Walls	....	May 28, 1962	4250	1295.4
Pump floor	....	Aug. 1, 1962	4250	1295.4
Motor floor	....	Oct. 11, 1962	4265	1300.0
Backfill	Sept. 11, 1962	Nov. 15, 1962	4265	1300.0

set between piles and in the plant floors. Initial elevations were obtained for each of these. The settlement observation points and benchmarks were located at important points in the plant areas.

Observations were begun in the spring of 1962. Initially, the observations were made at 10-day intervals. However, this interval was extended to 30 days when indications were that settlement rates had decreased sufficiently, and was further extended to 3 months when settlement was virtually complete.

Foundation conditions at Plant No. 1, shown by the logs and penetration resistance data in figures 14 and 15, were far more susceptible to settlement than at Plant No. 2. The data in table 9 typify the difference in the foundation soils below the two plants. The difference in natural density, consolidation, and sensitivity indicates the greater susceptibility to settlement at Plant No. 1.

Also, the load applied to the foundation soils was greater at Plant No. 1 because of the large volume of backfill required. During the construction period, records were kept of the settlement at each benchmark and of the construction activities. Figure 31 shows a typical curve of settlement with time. Also shown are the dates of the construction activities.

Foundation conditions at Plant No. 2, as shown by the log in figure 16, were not unusual and settlement was not of as much concern as it was at Plant No. 1.

TABLE 9.—*Foundation soil comparison*

	Depth (feet)	Natural density (pcf)	L L (percent)	P I	One-dimensional consolidation (100 psi load 0/0)	Tan	Cohesion	Qualitative estimate of soil sensitivity
Plant No. 1	45	74.3	37	20	24	0.6	1.0	Sensitive.
Plant No. 2	56	93.1	33	15	13	0.5	2.7	Nonsensitive.

Settlement records were kept and construction activities noted, as was done at Plant No. 1. Figure 31 presents a typical curve of settlement with time, and relates the settlement rate to the construction activities.

Discussion of Settlement

General—For the purpose of clarity, settlements at the two pumping plants are discussed separately. Greater attention is paid to the settlement at Plant No. 1 because most of the problems were connected with the foundation at this plant. The events that caused settlement at each plant are discussed in chronological order and the rate of settlement is related to them at the time they occurred. Settlement curves for both plants are presented in figure 31, and the construction activities are shown at the time of their occurrence. Settlement readings at all other observation points were so similar to those presented that it was obvious that differential settlement was not significant at either Plant No. 1 or 2. The earthquakes that occurred during construction did not cause significant settlement.

Plant No. 1—Settlement readings at this plant were begun on May 28, 1962, when the pile foundation was complete and the pump floor was under construction. For the first 94 days after readings were begun, settlement occurred at a constant rate of about 0.15 foot per year. This settlement can be attributed to compression of the piles and to recovery from the heave that occurred during driving of the pile foundation. Total settlement in this period was about 0.04 foot. At the end of 94 days, backfill operations were begun, and two earthquakes of moderate intensity occurred. Simultaneous with these events the rate of settlement increased to 1.5 feet per year and remained essentially constant at this rate until 110 days after the start of observation.

Between 110 and 125 days, ground water was allowed to rise to the pump floor, and the direction of foundation movement was reversed. Subsequently, settlement continued at a nearly constant rate of 0.50 foot per year until about 180 days had elapsed. At this time backfilling was completed to the original ground surface, and thereafter the settlement rate decreased. Within the period from 94 to 180 days the structure was nearly completed to the motor floor at elevation 4229. The total settlement during this period was about 0.16 foot.

During the period from 180 to 273 days no backfill was placed and construction of the superstructure was nearly completed. Total settlement during this period was 0.01 foot, a rate of 0.04 foot per year, which is negligible when compared to that in the previous period.

Placing of Zone 1 backfill material above the ground surface in a 5-foot layer for a limited area around the plant was begun after 273 days and was completed between that time and 357 days. During this period the plant superstructure was completed and the pumps and motors installed. The rate of settlement during this period increased to about 0.09 foot per year and the total setttlement was about 0.02 foot.

After 358 days the plant was completed and, from that time until 1,600 days had elapsed, an additional settlement of only 0.04 foot occurred.

In review of the events that occurred during the construction of this plant and their relationship to the settlement of the plant, it is evident that backfilling of the excavated area was the greatest contributing factor to settlement. The structure itself undoubtedly had some part in causing the settlement, but from the observed data this must have been a minor amount.

Plant No. 2—At this plant, settlement observations were begun on March 30, 1962. From the beginning of observations, settlement occurred at a nearly constant rate of 0.20 foot per year until about 250 days. During this period, construction of the structure was nearly completed and the backfill placed. The individual effect of each of these events on the settlement is not distinctly separable from the effects of the total construction activities, but settlement was concurrent with them. The total settlement during this period was about 0.14 foot. After 250 days the settlement rate decreased to 0.007 foot per year and remained constant with the exception of minor and temporary discrepan-

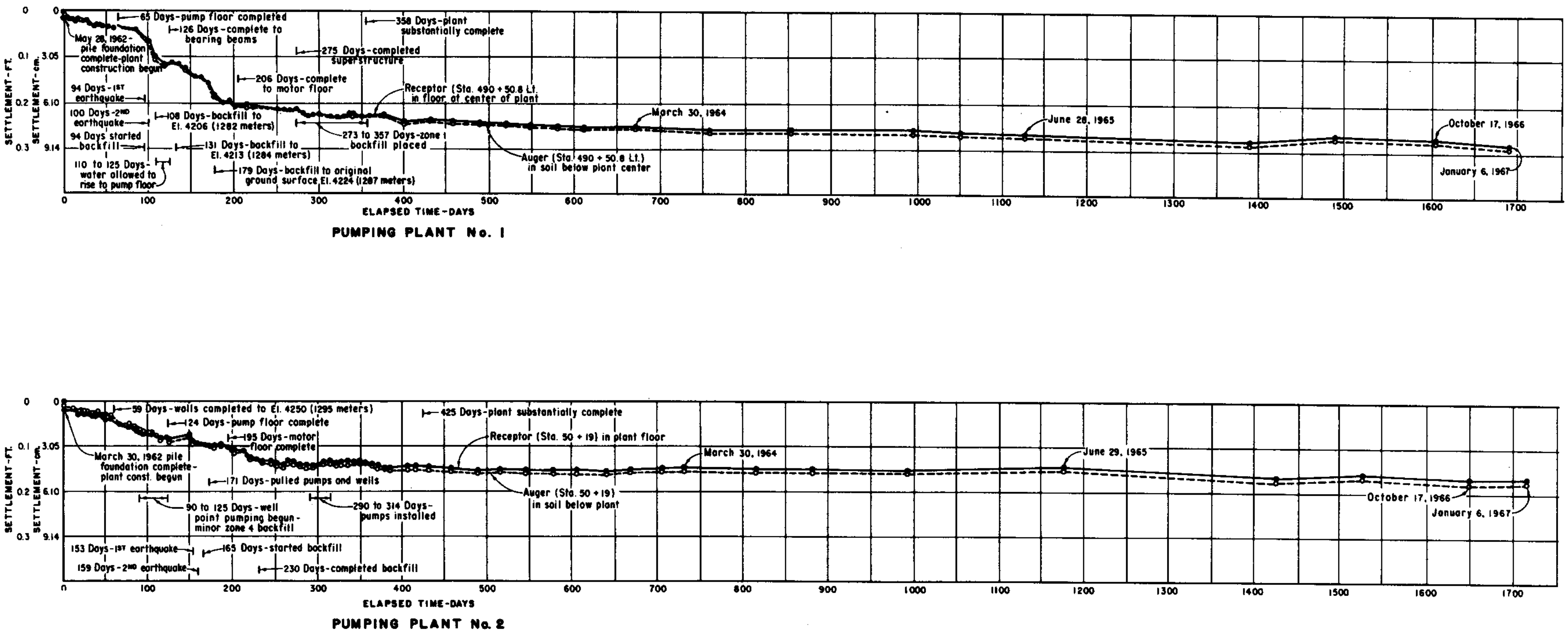

Figure 31.—Typical settlement observations—Pumping Plants Nos. 1 and 2.

cies. The total settlement during the period between 250 and 1,650 days was about 0.03 foot.

There was nothing unusual about the settlement record of this plant; the settlement resulted from backfill and structural loads as at Plant No. 1. The amount and rate of settlement are within normal limits for structures of this type founded on piles in this type soil.

Summary and Conclusions

Each of the two pumping plants presented different foundation conditions and consequently different settlement characteristics were exhibited. The foundation soil underlying Plant No. 1 is mostly soft silts and clays. The conclusions related to this plant are best illustrated by figure 32. When the plant's settlement is compared to that of the nearby bypass canal and discharge structures, which are not pile supported and involved considerably less load changes, the critical nature of the foundation becomes apparent. It is obvious that, if the heavier pumping plant involving more backfill had not been pile supported, its settlement would have been greater than those of the bypass canal and discharge structures.

The pile foundation supported the structure on firmer soils at greater depth, but did not completely prevent settlement caused by the extensive backfill operation. Although the backfill load is only a slight increase over previously existing loads, the soils are so critical that the reloading consolidation is significant and is the obvious reason for the rapid settlement during construction. After completion of construction, it appears that the plant is being adequately supported by the pile foundation.

Plant No. 2 is founded on a moderately stable soil foundation. The settlement at this plant occurred with no unusual deviations from an essentially constant rate of settlement during construction, with a gradual tapering to a nearly stable condition after completion of construction.

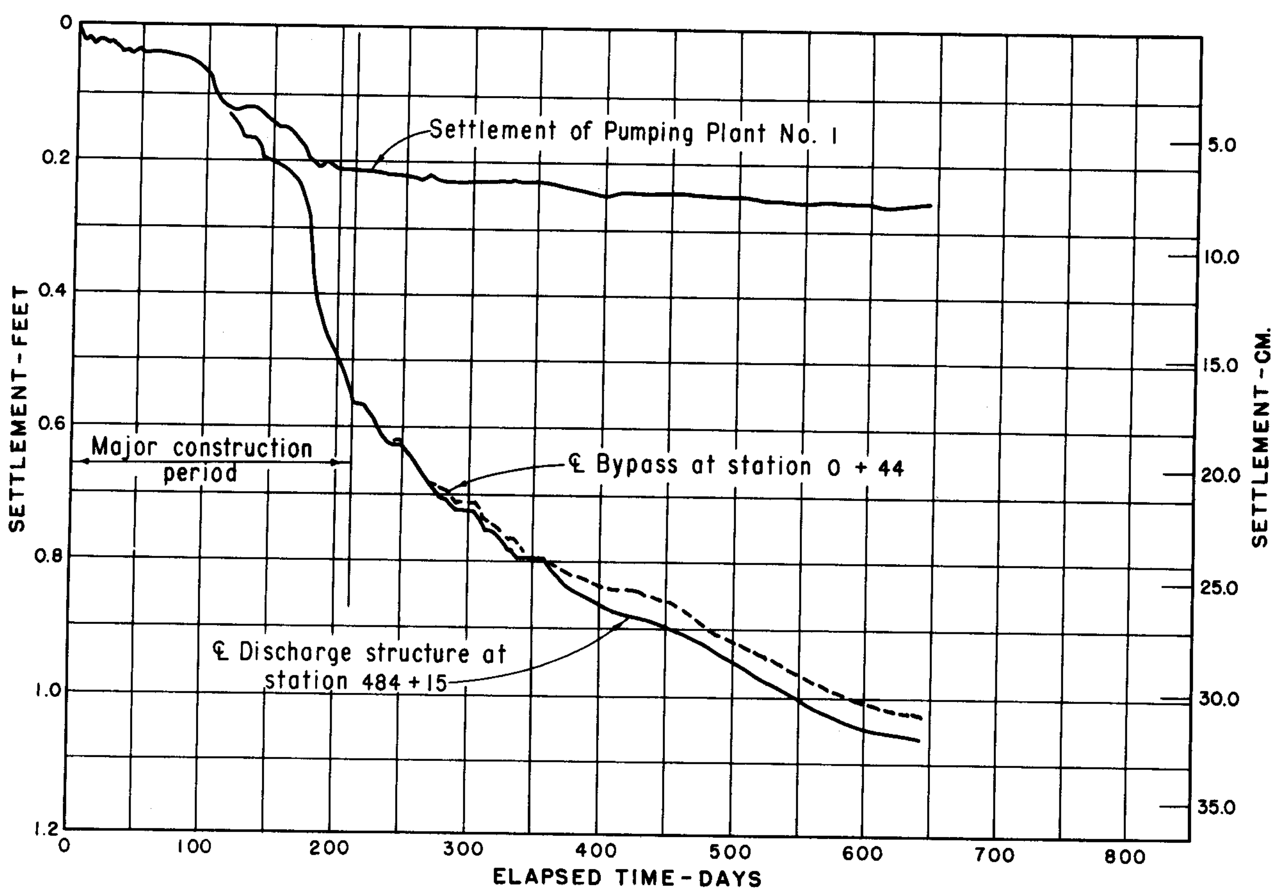

Figure 32.—**Comparison of settlement of Pumping Plant No. 1 with other structures.**

ABSTRACT

Engineering experience derived from the foundation investigations, design, and construction of two pile-supported pumping plants on soft, lean-clay lake sediments is reported. A complete discussion of treatment of the foundation problems is presented, with details of several novel test methods and full-scale field observations. Project requirements are outlined which necessitated siting the plants in the poor foundation areas. With general soil characteristics known from Willard Dam investigations, the exploration program was designed to locate specific sites where soils of the greatest firmness existed. Field investigations included undisturbed sampling, in-place penetration tests, pile loading tests, and vane shear tests adjacent to the test piles. Special laboratory tests were conducted to evaluate changes in soil compressibility due to pile driving and to study changes in pile bearing strength with time. Procedures for the field pile testing program are outlined, and settlements recorded during and after foundation construction are discussed. Validity and value of the foundation testing program were confirmed by the results of settlement observations.

DESCRIPTORS—*pile foundations / *foundation investigations / vane shear tests / stability analysis / friction piles/pumping plants/*settlement/model tests/ test procedures / soil compression tests / fine-grained soils / pile driving / wood piles / penetration tests / backfills / excavation / clays / soil investigations / sediments / silts.

IDENTIFIERS—*pile tests / pile-driving formulas / Willard Pumping Plant, Utah / Utah.

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Partial List of
WATER RESOURCES TECHNICAL PUBLICATIONS*

ENGINEERING MONOGRAPHS

No.
31 Ground-Water Movement
32 Stress Analysis of Wye Branches
33 Hydraulic Design of Transitions for Small Canals
34 Control of Cracking in Mass Concrete Structures
35 Effect of Snow Compaction on Runoff from Rain on Snow
36 Guide for Preliminary Design of Arch Dams
37 Hydraulic Model Studies for Morrow Point Dam

RESEARCH REPORTS

1 Research—Engineering Methods and Materials
2 Aquatic Weed Control Studies
3 Soils Tests Computer Programs
4 Hydraulic Downpull Forces on Large Gates
5 Park Range Atmospheric Water Resources Program—Phase I
6 Annual Report of Progress on Engineering Research—1965
7 Effects of Monolayers on Insects, Fish and Wildlife—A Reservoir Evaporation Reduction Study
8 Synthetic Resin Primer for Coal-Tar Enamel
9 Vibration Studies of Monticello Dam
10 Annual Report of Progress on Engineering Research—1966
11 Pile Supported Structures in Lake Deposits
12 Buried Asphalt Membrane Canal Lining
13 Removal of Saline Waters from Aquifers

TECHNICAL RECORDS OF DESIGN AND CONSTRUCTION

Trinity River Division Features of the Central Valley Project
Twin Buttes Dam
Morrow Point Dam and Powerplant Foundation Investigations

*For a pamphlet listing other Bureau of Reclamation publications, address Office of Chief Engineer, Attention 841, Denver Federal Center, Denver, Colo. 80225.