WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

MISSISSIPPI RIVER COMMISSION

THE CALIFORNIA BEARING RATIO TEST AS APPLIED TO THE DESIGN OF FLEXIBLE PAVEMENTS FOR AIRPORTS

TECHNICAL MEMORANDUM NO. 213-1

U. S. WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

1 JULY 1945

Price $3.00

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SYNOPSIS

This memorandum is a report on the California bearing ratio (CBR) test which is part of the California method of design for flexible pavements. The method was tentatively adopted by the Corps of Engineers in 1942 for the design of flexible pavements for military airports. The purpose of the CBR test, which is a penetration type test, is to determine a modulus of shearing resistance of soils. The modulus value determined by the test is used in conjunction with empirical design curves (based in part on correlation with field performance) to determine the total thickness of base and wearing course required to protect the subgrade and base course against shear failure. The CBR test may be used in the design of new pavements, evaluation of the load-carrying capacity of old pavements, or as a field control in the construction of new pavements.

For design or evaluation purposes, it is necessary to conduct CBR tests on, (1) natural undisturbed samples with water content adjusted to expected field conditions, and/or (2) on remolded samples which have the same <u>molding water content</u>, <u>moisture conditions</u>, <u>density</u>, and <u>physical properties</u> that will be produced during or after construction. Therefore, <u>the CBR test can not be considered as a classification test, but is a shear test, and the CBR values obtained from the test are measures of shearing resistance, the validity of which are dependent on preparation of the test specimen to duplicate field conditions</u>. The test is considered valid only when a large portion of the deformation under penetration is shear deformation.

Included herein are a discussion of the development and limitations of the CBR test and a summary of the results of a comprehensive laboratory study, which was conducted to investigate factors leading to the development of a procedure for the preparation of remolded samples and to the development of a CBR test procedure; recommended procedures for preparation of remolded and undisturbed samples for the CBR test; a comparison of field and laboratory CBR data obtained from accelerated traffic test sections and special pavement behavior test sections, and recommendations for future investigations. Also included are appendices giving, (a) the detailed data obtained in the comprehensive laboratory study, including a discussion on the physical behavior of remolded soils, (b) a comparison of the compaction characteristics of plastic soils, and (c) a description of a recently developed field in-place CBR apparatus and its operation.

The results of this investigation show that the wide variations in the CBR test results on laboratory compacted samples are largely due to the method of preparation of the test specimen, and that similar variations are obtained in the results of unconfined compression and triaxial tests. In cohesive soils compacted by the impact method, small changes in molding moisture will greatly affect the CBR. On these soils the molding moisture controls the type of soil system formed during compaction, with the result that different physical properties are obtained in identical soils for different molding moisture contents. These properties are retained by soils which do not exhibit high swell, even though the specimens are soaked prior to testing. The variations in CBR with changes in density and molding moisture content are systematic for one compaction method. Consistent laboratory results on soils sensitive to molding water content can be obtained only when these variables are given full consideration. By conducting a series of tests, varying compactive effort and molding moisture content, sufficient information may be obtained to show the effects of moisture content and density on the CBR under a given method of laboratory compaction.

Except for clean free-draining sands and gravels, it is not known how closely the physical properties obtained by laboratory compaction methods used in this investigation correspond to those obtained by the various field compaction methods now available for use. The sensitivity of the physical properties of the soil to molding moisture and method of compaction therefore makes it difficult to produce soil conditions in the laboratory which will be the same as those produced in the field. Before a laboratory procedure is finally established for preparing samples for design strength tests, it will be necessary to study the physical properties of soils compacted by field compaction equipment. However, until more data are available, it is recommended that the remolded laboratory specimens for CBR tests be prepared using dynamic or impact compaction (dropping hammer) and not static compaction. Whenever applicable, field in-place tests, or tests on undisturbed samples of materials in cut areas should be performed adjusting the water content to the degree of saturation to be ultimately expected in the field. During construction, field in-place tests or tests on undisturbed samples taken from compacted fills or base courses should be performed, and either the design requirements or construction procedure changed if necessary. A satisfactory field in-place CBR apparatus has been developed and is described in this report.

In general, the CBR test, which is considered to be only the penetration portion of the original adopted procedure, has proven satisfactory with two changes. A surcharge load on the specimen during penetration has been added, to make the test more satisfactory for cohesionless soils, and an adjustment of the stress-strain curves obtained from the test has been made, to correct for low initial stress measurements.

Since the adoption of the CBR test by the Corps of Engineers, much interest has been aroused in regard to development of a proper design procedure for flexible pavements. Although the CBR test has several shortcomings, it is believed that the use of the test in conjunction with empirical curves offers the best solution, at least for the time being, to the design of flexible pavements for airports. For this reason, it is thought that studies and investigations of the design method and of the preparation of samples should continue. Of special importance is the necessity for the construction of closely controlled field test sections, using different types of compacting equipment, in order to compare the physical properties obtained from dynamically and statically compacted laboratory soil specimens with those of field compacted soils, and to determine the effect of traffic on soils molded on the dry and wet side of optimum water content. Laboratory compaction equipment should be specified so that the same physical properties can be obtained in both the field and the laboratory. A study of field compaction is now being performed by the Corps of Engineers.

There are no data available to show whether the present field compaction methods more closely duplicate dynamic or static laboratory compaction. However, from the results of laboratory studies, it appears that with the exception of soils exhibiting high swell, subgrades and base courses for pavements should be compacted in the field slightly dry of the optimum water content for the effort being used, in order to obtain high strength and a rigid soil mass. It appears that in all types of material except the high swelling clays, compaction on the dry side will probably be beneficial, even though over a long period of time the water content of the material beneath the pavement may increase appreciably.

PART I: INTRODUCTION

Authorization

1. A comprehensive laboratory study of the California bearing ratio test was begun in November 1942 to determine satisfactory methods for preparation of specimens for CBR design tests and to develop the CBR (penetration) test. This study was initiated by the Office, Chief of Engineers in a second indorsement, dated 17 September 1942, to a letter from the U. S. Waterways Experiment Station, dated 7 September 1942, subject "California bearing ratio test procedure." Authority to perform this work was granted by the Office, Chief of Engineers in the sixth indorsement, dated 13 November 1942, to the basic letter. The Experiment Station was directed by the Office, Chief of Engineers, in a letter dated 4 March 1944, to prepare a final report containing all available ~~data pertinent to the development and use of the CBR test.~~

Purpose of Report

2. It is the purpose of this technical memorandum to:

   a. Describe the background and limitations of the California method of design for flexible pavements and the reasons for its adoption by the Office, Chief of Engineers, U. S. Army, for the design of flexible pavements for airports.

   b. Discuss investigations, reasons for investigations, and development of the CBR test to date.

   c. Recommend procedures for the preparation of samples and for performance of the CBR (penetration) test.

   d. Give recommendations for those investigations which are considered necessary for further basic development of the test.

Scope of Report

3. The background, development, and limitations of the CBR test procedure, together with its application to design and a résumé of the principal findings of the comprehensive laboratory study of the CBR test,

are covered in the main body of this report. Also contained in the main report are procedures recommended for preparation of remolded and undisturbed samples for CBR (penetration) test; pertinent comparisons of laboratory and field CBR data from several projects, and recommendations for further investigations. The detailed results of the comprehensive laboratory studies on the preparation and penetration of CBR test specimens are given in Appendix A. The results of a comparative study of the compaction characteristics of plastic soils are contained in Appendix B. A description of a recently-developed combination screw jack and proving ring field CBR apparatus, together with a detailed procedure for operation of this apparatus, are given in Appendix C.

Definitions

4. A proper understanding of the terms and familiarity with the symbols listed below will be of assistance to the reader in the study of this report.

Bracketing soils - A term applied to a representative group of soils chosen for investigating the effects of numerous variables on the CBR. These soils were tested extensively and were termed "bracketing" because they ranged from sand to fat clay.

Physical properties of soils - As used in this report, this term refers primarily to those properties of a soil mass which are changed by variations in moisture, density, percentage air voids, grain arrangement, moisture films, or any other factors. These properties include, but are not necessarily limited to: compressibility, permeability, shearing resistance of the soil mass, and volume change in the mass with change in moisture content. The term applies to the soil mass as a whole and not to to the physical properties of individual grains, although the physical properties of the grains may affect those of the soil mass. Changes in physical properties can be measured by consolidation, shear, permeability, or other physical tests.

Soil system - Soil system refers to an intimate mixture of soil with air and/or water. A soil system may be a two-phase system, consisting of an air or gaseous phase and a soil or solid phase (dry soil), or a water or liquid phase and a solid phase. Usually, however, the soil system consists of three phases: a water phase, an air phase, and a soil phase. Arrangements of grains in the soil phase, or of water and air voids in the air and water phases, may greatly affect the physical properties of the soil mass.

California bearing ratio or CBR - A measure of shearing resistance by soils to penetration, which is determined by comparing the bearing value obtained from an arbitrary penetration type shear test with a standard bearing value obtained on crushed rock (average value of tests on a large number of crushed rock samples). The standard results are taken as 100 percent and values obtained from other tests are expressed as percentages of the standard. CBR may be modified by the terms, laboratory, field in-place, undisturbed, soaked, or unsoaked, according to the conditions under which the specimen was prepared for test or where the tests were made, or by the terms design or evaluation, according to the purpose for which the test results will be used.

Uncorrected CBR - The CBR value taken at the first 0.1 inch of penetration without making any adjustment of the stress-penetration curve.

Corrected CBR - The CBR value at 0.1-inch penetration after adjusting the stress-penetration curve. (The adjustment of the curve is covered in detail in Part II of Appendix A.)

As-molded CBR - The CBR obtained on a remolded specimen tested immediately after compacting or molding. No change in water content or density is allowed to take place before testing.

Unsoaked CBR - Same as the as-molded CBR for remolded specimens, and also the CBR obtained from a field in-place test, or the CBR on an undisturbed sample secured from a cut area or compacted fill at the field-in-place moisture.

Soaked CBR - The CBR obtained on a specimen penetrated after being soaked by one of several methods used (i.e., soaked from bottom only, top only, or top and bottom).

Field in-place CBR - The CBR obtained on a specimen tested in situ with a special field CBR testing apparatus (see Appendix C). This can either be soaked or unsoaked.

Undisturbed mold CBR - The CBR obtained by testing an undisturbed sample taken in the CBR mold. This can either be soaked or unsoaked.

CBR design test - A design test is a test made to determine a physical property of a soil mass, under the conditions which it is anticipated will exist in the completed fill, cut, or base course. The CBR design test is made on a remolded specimen prepared at the density and under the moisture conditions which will exist during construction or on undisturbed natural specimens. Before testing, the water content is adjusted to that which will, it is anticipated, eventually occur during the life of the pavement under consideration.

Evaluation CBR test - As used in this report, an evaluation test is one which is made to aid in the evaluation of an existing pavement. It may consist of either a field in-place or laboratory test on undisturbed soil at its natural field in-place moisture or adjusted for future moisture conditions. The evaluation test is performed to determine either the CBR of an existing subgrade or base course, what its CBR may be assumed to be after a long period of time, or whether the assumptions as to density and water content used for preparation of specimens for the design test are duplicated during construction.

Water content (w) - The ratio, expressed as a percentage, of the weight of water in a given soil mass to the total dry weight of solid particles.

(1) Water content, optimum (w_o) - The water content at which the maximum density is produced in a soil by a specific effort.

Water content, initial (w_i) - The water content of the soil immediately before more water is added in the performance of a soil test such as the compaction test.

Water content, molding or as-molded (w_m) - The water content of the soil at the time it is compacted, or of a test specimen at the time it is molded.

Water content gradient - In this report this term refers to the distribution of the water (expressed as a percent of dry weight of soil) from top to bottom of a soil specimen.

(1) Percent or degree of saturation (S) - The ratio, expressed as a percentage, of the volume of water in a given soil mass to the total volume of intergranular space (voids).

Saturated soil - This term, as used in airport and highway practice, refers to a soil which has absorbed the maximum amount of moisture it can hold without being disturbed or remolded. The sources of this water may be capillary water, ground-water seepage, condensation, or pavement leakage. Most soils when saturated in this manner are not theoretically fully saturated but contain varying amounts of air voids, depending on the soil type. This does not refer to remolded conditions under which soils can lose appreciable strength and in some cases may approach a semiliquid condition.

(1) Specific gravity (G) - The ratio of the weight in air of a given volume of material at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature, usually 4 degrees C. (For soils, refers to solids only.)

[1]Void ratio (e) - The ratio of the volume of intergranular space to the volume of solid particles in a given soil mass without regard to the proportions of liquid, air, or gas which may occupy the space.

Dry density (γ_d) - Dry unit weight of the soil in pounds per cubic foot.

Wet density (γ_w) - Wet unit weight of the soil in pounds per cubic foot.

Density gradient - In this report this term refers to the dry density distribution from top to bottom of a soil specimen.

Dry side of optimum - Drier than optimum water content for any effort used. The condition of the soil when the plotted point of dry density (ordinate) versus molding water content (abscissa) falls on the left side of a line showing the relation between optimum water content and dry density for a given compaction method.

Wet side of optimum - Wetter than optimum water content for any effort used. The condition of the soil when the plotted point of dry density versus molding water content falls on the right side of a line showing the relation between optimum water content and dry density for a given compaction method. .

Dynamic compaction - Compaction of the soil by the impact of a free-falling weight or hammer.

Static compaction - Compaction of the soil by gradually increasing the compacting pressure (applied by means of a piston or plunger) up to any given amount. (Rate of application in this investigation was approximately 0.05 inch per minute.) In some instances the static load was applied to the top of the specimen and, after unloading, the mold was inverted and the load applied to the other end of the specimen. This procedure has been designated as "load applied once from each end."

Compactive effort in foot pounds of energy - Numerically equal to the product of the weight of the hammer, the height of fall, and the number of blows. Significant only when the following factors are known: thickness of the compacted layers, area of the strking face of the hammer, and the unit volume over which the energy is applied. This is further described in Part IV of Appendix A.

Surcharge - A confining weight placed on top of the CBR specimen during soaking and/or penetration.

Standard AASHO (also standard Proctor) mold - A cylindrical metal mold, 4 inches in diameter by 4.6 inches in height, having a metal base and collar.

Standard AASHO (also standard Proctor) hammer - A solid metal tamper weighing 5-1/2 pounds and having a circular striking face 2 inches in diameter.

Modified AASHO (also modified Proctor) hammer - A solid metal tamper weighing 10 pounds and having a circular striking face 2 inches in diameter.

Standard CBR mold - A cylindrical metal mold 6 inches in diameter in which a specimen approximately 5 inches in height can be compacted. (Note: Specimens 4-1/2 inches in height were used in this investigation.)

Standard AASHO (also standard Proctor) compaction - Dynamic compaction in Proctor mold using 25 blows (12-in. free drop) of standard Proctor hammer on each of three equal layers, or dynamic compaction in CBR mold using 55 blows (12-in. free drop) of standard Proctor hammer on each of three equal layers. The compactive effort obtained by the latter method is equivalent to that obtained in the standard AASHO test.

Modified AASHO (also modified Proctor) compaction - A modification by the U. S. Corps of Engineers of the standard AASHO compaction method and consists of: Dynamic compaction in Proctor mold using 25 blows (18-in. free drop) of modified Proctor hammer on each of five equal layers; dynamic compaction in CBR mold using 55 blows (18-in. free drop) of modified Proctor hammer on each of five equal layers.

15-blow Proctor compaction - Dynamic compaction in Proctor mold using 15 blows (12-in. free drop) of standard Proctor hammer on each of three equal layers; or dynamic compaction in CBR mold using 35 blows (12-in. free drop) of standard Proctor hammer on each of three equal layers.

Porter static compaction - The sequence of compaction is as follows: first a comparatively high rate of compression is used until a load of 100 psi is acting; then a rate of 0.1 inch per minute is used until a pressure of 1000 psi has been applied, and finally a rate of 0.05 inch per minute is used until 2000 psi is reached. The 2000 psi is maximum for the standard test. This maximum load is maintained for one minute.

Processing base course material - Altering the distribution of the various particle sizes. In this investigation, processing consisted in removing all material larger than 0.74 inch (square mesh sieve) and replacing this with equal percentages by weight of size 0.74 to 0.37 inch and 0.37 to 0.18 inch. The percentage of material finer than 0.18 inch thus remains constant.

(1)Permeability coefficient (k) - The discharge velocity of flow of fluid through a porous mass under a unit hydraulic gradient.

Free-draining soils - As used in this report this term applies to soils having a coefficient of permeability of 10^{-3} cm per sec or greater.

Practically impervious soils - As used in this report this term applies to soils having a coefficient of permeability of less than approximately 10^{-6} cm per sec.

Pore water or neutral pressure - The portion of the total pressure exerted on a soil system which is carried by the pore water or liquid phase. Pore water pressure is a neutral stress because it does not contribute to the shear strength of the soil.

Effective or intergranular pressure - The portion of the total pressure exerted on a soil system which is carried by the solid phase or transferred directly from grain to grain. These pressures are effective in the sense that they produce frictional resistance between grains.

Detrimental swell - For base course materials, 1 percent swell or greater. For subgrade materials, 3 percent swell or greater. This swell is measured after four days soaking top and bottom in the CBR mold and is computed on the basis of the total initial specimen height.

σ_1, σ_2, σ_3 - Major, intermediate, and minor principal stresses.

Triaxial compression test - A physical test used to determine the stress-deformation characteristics of soils. Cylindrical test specimens are usually first subjected to an external pressure ($\sigma_3 = \sigma_2$) and are then sheared to failure by application of a vertical load (σ_1) applied longitudinally. The tests are classified as slow, consolidated quick, or quick, depending upon the time allowed for consolidation between application of loads.

Quick (Q) triaxial compression test - A test in which the specimen is loaded quickly to failure without allowing drainage or volume change to occur during application of load.

Consolidated quick (Q_c) triaxial compression test - A test in which the specimen is first fully consolidated under a lateral pressure (σ_3) and is then loaded quickly to failure without allowing any further volume change.

Slow (S) triaxial compression test - A test in which the specimen is first fully consolidated under lateral pressure (σ_3) and is then loaded to failure by allowing sufficient time for each increment of vertical load applied to become fully effective on the solid phase.

Deviator stress - In the triaxial test the deviator stress is numerically equal to the difference between the major principal stress and the minor principal stress ($\sigma_1 - \sigma_3$).

Atterberg limits constants - LL, PL, PI - Liquid limit, plastic limit and plasticity index.

(1) American Society Civil Engineers - Manual of Engineering Practice No. 22, "Soil Mechanics Nomenclature." Prepared by the Committee of the Soil Mechanics and Foundations Division on Glossary of Terms and Definitions and on Soil Classification. Adopted 20 April 1941.

Acknowledgments

5. The investigation was conducted by personnel of the Embankment and Foundation Branch of the Embankment, Foundation and Pavement Division of the U. S. Waterways Experiment Station under the supervision of Mr. W. J. Turnbull, Chief of the Division, and Mr. W. H. Jervis, Chief of the Branch. Immediately in charge of the project was Mr. J. B. Eustis with the following principal assistants: Messrs. J. L. McRae, C. D. Burns and J. E. Watkins. Mr. R. M. German was in charge of the project during its initial stages. Special credit is due Mr. McRae for his concept and development of the postulation of compacted soil systems in remolded soils and for his method of presenting test data. Special credit is also due Mr. Burns for careful performance of laboratory tests and assistance in analyzing data. The study was accomplished under the general direction of Mr. Gerard H. Matthes and Capt. Joseph B. Tiffany, Jr., Director and Executive Assistant, respectively, of the Experiment Station.

6. Acknowledgment is made to personnel of the Office, Chief of Engineers, for assistance and advice in connection with test and design procedures and preparation of this memorandum, chief among whom were Brig. Gen. James H. Stratton, Lt. Col. Hibbert M. Hill, Messrs. Gayle McFadden, Reuben M. Haines, Dana D. Leslie, and T. A. Middlebrocks. Acknowledgment is also made to personnel of the Soils and Pavement Section of the Little Rock District, head of which is Mr. Ralph Hansen, for providing the basic idea of the field CBR apparatus which has been further developed by the Experiment Station.

7. Consultants who assisted with the testing program and provided timely advice on design procedures were Mr. O. J. Porter, California Division of Highways, who originated the California bearing ratio test; Dr. Arthur Casagrande, Harvard University; Professor P. C. Rutledge, Northwestern University, and Mr. R. R. Philippe of the Cincinnati Testing Laboratory.

PART II: ADOPTION OF THE CALIFORNIA METHOD FOR DESIGN OF FLEXIBLE PAVEMENTS

Necessity for Pavement Design Procedure

8. In the latter part of 1940, the Office, Chief of Engineers was confronted with the problem of establishing procedures for designing flexible airport pavements to withstand anticipated magnitude of traffic and the stresses produced by heavy military and transport airplanes. It was considered necessary to establish design procedures for the following reasons:

a. To insure adequately designed pavements

b. To eliminate a wide variation in designs based on judgment of paving engineers who were not fully acquainted with the requirements of the Department or with pavement requirements for anticipated airplane traffic.

c. To limit the use of unproven theoretical design methods for flexible pavements.

d. To provide in the Department a uniform design procedure not related to arbitrary cost differentials of local and competitive materials, and to avoid reductions of pavement thickness to balance costs.

e. To secure foundations for further development of design methods through the application of data obtained by future investigations and actual behavior of pavements.

9. The pavement thicknesses required for airplane traffic in many instances are vastly different from those for highways. In 1940, engineers lacked experience in the construction of airfields for heavy airplanes and intensive traffic. Unfortunately, highway design practices, in general, did not necessitate the development of design procedures which could be used to determine the design thickness requirements for a wide range of wheel loads. Most of the State Highway Departments based the design thickness of pavements and bases on judgment from experience, limited to a more or less narrow range of materials and conditions encountered in individual states. No theoretical method of flexible pavement design was used extensively by any Highway Department or recommended by any organization of Paving Engineers. Several State Highway Departments used empirical methods for the design of flexible pavements for primary and secondary highways.

Flexible Pavement Design Procedure

10. Investigations and studies were conducted by the Office, Chief of Engineers to establish a design procedure for flexible pavements. At first, all of the available design procedures developed for highway design were studied. Nearly all of these methods consisted of theoretical formulas based on an analysis of the effect of a pavement upon the stress distribution and an assumption that the "bearing capacity" of the soil subgrade could be determined by some method such as the plate bearing test. Although the methods have considerable merit they have little use, since the proper procedures for determining the "bearing capacity" of the subgrade have not been developed. In an attempt to make use of various formulas, the Department studied all available data and conducted special field investigations at Langley and Bradley Airfields and on a Virginia Highway Department test section to develop procedures for determining subgrade bearing values applicable for use in various theoretical formulas. The following conclusions were drawn:

a. Development of a satisfactory test procedure applicable to any of the formulas would require extensive investigations which could not be accomplished in time for use in the War Emergency Program.

b. In the use of the field plate bearing test, the proper deflection to determine the "bearing capacity" depends upon the basic assumptions in the formula and varies according to combinations of the following factors:

(1) Characteristics of subgrade soil.

(2) Relative characteristics of subgrade and base materials.

(3) Thickness of pavement.

(4) Magnitude of wheel load.

(5) Tire imprint area.

(6) Quantity of traffic.

(7) Expected life of pavement.

c. In most cases the plate bearing test results would not be applicable to soil moisture conditions expected ultimately to develop below a pavement, and it would be extremely difficult to develop a method satisfactory for adjusting the test results for the various moisture conditions. In view of the above, it was apparent that some method other than the use of a theoretical formula would have to be employed for at least a few years.

11. Several engineers proposed the use of plate bearing tests on pavement surfaces for the design of flexible pavements and, in order to determine the applicability of this method, field plate bearing tests were made on the surfaces of pavements and base courses. These investigations led to the conclusion that the same factors that must be considered in using the plate bearing test on subgrade soils must also be considered in determining the carrying capacity of a pavement by the plate bearing test method. In addition, the compressibility of the pavement and of the base material enter into the problem.

12. After several months of intensive study and investigation, the principles as used by the California Highway Department to develop a method for the design of flexible pavements were tentatively adopted in the early part of 1942. The principles consist of determining a modulus of the shearing resistance of the subgrade soil by a test and using the modulus with empirical curves developed by service pavement studies to determine the required combined thickness of base and pavement. The principles were tentatively adopted for the following reasons:

a. They were considered to be as sound and logical as any other basis.

b. They had been used successfully by the California State Highway Department to develop a method of design applicable for conditions in that state.

c. North Dakota and Florida had used the principles to some extent for the design of highway pavements.

d. A satisfactory method could be developed more quickly if based on these principles than if based on other principles.

e. The results of accelerated traffic tests using applicable wheel loads on existing pavements and on special test sections and the data of the empirical design curves of the California method could be used to quickly develop design curves for airfield pavements for any required traffic.

f. The CBR test developed by the California Highway Department to determine the modulus of the shearing resistance of the soil for use in the California method of design could be adopted for immediate use. (Note: It was believed that the test procedure would be satisfactory for testing all types of soils, provided the field compaction standards for airport pavements were the same as used by the California Highway Department.)

g. All soil tests could be made in the laboratory with simple equipment, which is also advantageous for design and construction in the theatres of operation during the war.

h. The soil test could be conducted on soaked samples representing the conditions that ultimately may develop beneath most pavements. If desired, tests could be conducted with the soil at other moisture contents.

13. The California Bearing Ratio (CBR) test was adopted to deter mine the modulus of shearing resistance of soils for use with empirica design curves, since it has been successfully used by the California Highway Department. Tentative design curves applicable for airfield pavement design were prepared by consultants and personnel of the Office, Chief of Engineers. After the tentative design curves had been found sufficiently accurate according to results of accelerated traffic tests on existing pavements at Corpus Christi, Dothan, Fargo, and Lewiston Airfields, and on an experimental pavement constructed at Stockton, California, a conference was held in Sacramento, California, to present the method of design. Representatives from all Divisions attended the conference. At the conference, Mr. O. J. Porter, Senior Testing Engineer for the California State Highway Department, and personnel from the Office, Chief of Engineers, described the tentative design curves and the CBR tests. In addition, construction control methods used by the California Highway Department were described. Minutes of the conference were published by the Sacramento District in a folder entitled "Lecture Course on California Method of Determining the Relative Value of Soils and Application to Design of Highways and Runways." In the first issue, June 1942, of Chapter XX of the Engineering Manual, the method of design developed, termed the California Method, was submitted to all offices in the Department, and it was stated that the method would be used for design of pavements under the jurisdiction of the Department.

14. To refine the design curves, additional data were obtained by accelerated traffic tests on specially constructed test sections at Barksdale, Langley, Eglin and Grenier Fields. In addition, accelerated traffic tests were performed on existing pavements at Santa Maria, Natchitoches, Beltsville and Richmond Airfields, and pavement deflection tests were made using airplanes on a specially constructed section at Marietta, Georgia. At the present time it is considered that no more special test sections will be required to obtain data on design thicknesses for wheel loads less than 60,000 pounds. Studies of airfield pavements under service will yield the desired information. However, a special section to be tested with a wheel load of 150,000 pounds is being constructed to furnish data for design curves beyond present limits.

15. The California Bearing Ratio (CBR) Test is essentially a penetration (shear) test to determine a modulus of shearing resistance of soils. It is considered applicable to soils in foundations below flexible pavements. For the purpose of determining the carrying capacity of existing pavements, the penetration test is made on undisturbed soil taken from beneath the pavement. At the time of adoption of the principles of the California Method of Design for flexible pavements by the Corps of Engineers, it was believed that the method of preparation of the sample and the penetration test (CBR) procedure, as used by the California Highway Department, could be used in the design of airport pavements, provided that the compaction actually achieved during construction produced densities equivalent to the densities obtained by construction and control methods used in California. To obtain compaction equivalent to that specified by the California Highway Department, the Corps of Engineers specified the use of the Modified AASHO Compaction Test for field control purposes. The control tests used by the California State Highway Department were not adopted, since they were not generally accepted or used. On the basis of limited data, it was assumed that the results obtained by the Modified AASHO Method would be nearly equal to those obtained by the California control tests.

16. Soon after adoption of the CBR test, it became evident that the compaction method used in preparing the sample for design test would have to be revised in order to utilize available equipment and in order to obtain densities required by the specifications. The procedure was revised to use a method of compaction similar to that adopted for field compaction control tests (Modified AASHO Test). As a result of some inconsistencies in test results and the need for modification in the preparation of the remolded samples for the design test, the program of investigation described in this report was initiated.

PART III: INITIAL MODIFICATIONS IN PROCEDURE FOR PREPARING REMOLDED SPECIMENS FOR DESIGN CBR TEST

General

17. From time to time after the adoption of the California method of design for flexible pavements, the Engineer Department has had to modify the method of preparing remolded specimens for design, so that the CBR penetration tests would give results more nearly applicable to field conditions. It is believed therefore that the discussion of the original California method of preparing test specimens, and of the California and Engineer Department control tests, together with initial modifications to the California method of preparing test specimens given in the paragraphs below, will be of assistance to the reader in understanding the development of the CBR method of test by the Corps of Engineers.

California Method of Preparing Remolded Specimens for Design Test

18. For the purpose of design, the California State Highway Department developed a mechanical procedure of preparing the soil for design tests. It is understood that, based on the experience in California, the results of tests on the laboratory prepared specimens are equal to those obtained by testing undisturbed soils below pavements constructed by the methods used in California after the pavements are in service a few years. The soil is prepared as follows:

a. The moisture-density relation for the soil is determined by using a static load of 2000 psi and a 6-inch diameter mold.

b. For the test, the soil is remolded and compacted at optimum moisture (as determined in a, above) under a static load of 2000 psi.

c. The test specimen is soaked from the top and bottom for a period of four days. During the soaking period the top of the specimen is confined with a surcharge weight of 12-1/4 pounds (equivalent to the weight of 4 or 5 inches of pavement).

19. The procedure as used by the California Highway Department for preparation of remolded samples for design tests can only be considered

satisfactory when the same construction methods and control are used as used in California and it is desired to design for saturated soil conditions, a criterion of design used in that state. Information gained from a study of the California procedure indicates that, if it is anticipated the usual field compaction will not be obtained, a special laboratory procedure is used for preparing the soil sample that will produce a density equivalent to that expected in the field. If the soil is not to be compacted, penetration tests are conducted on undisturbed soaked soils.

California Compaction Control Tests[2]

Static compaction

20. For control compaction tests, California uses the static load method as a standard procedure, both in the laboratory and the field. In this standard test the moisture-density relation is determined by using a static load of 2000 psi in the CBR mold.

Impact compaction

21. An impact method is used as an alternate procedure in the field for control of a large percentage of construction jobs where quick determination of any variation in the materials is required. It is understood that the maximum density obtained is equal to that obtained by the static method described in paragraph 20. Tests for determining the CBR of soils, however, can not be made with the field equipment used for the impact test. The impact apparatus consists essentially of:

a. A split cylindrical metal mold with an inside diameter of 2.86 inches and a height of 3 feet, fitted with clamps and a detachable base.

b. A metal rammer or tamping shaft 3 feet 6 inches long, having a 2-inch diameter circular face and weighing 10 pounds.

c. A metal piston of 2.85 inch diameter and 2.7 inches long, fitted with a detachable rod for removing it from the cylindrical mold.

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(2) Minutes of the conference were published by the Sacramento District in a report entitled "Lecture Course on California Method of Determining the Relative Stability of Soils and Application to Design of Highways and Runways."

The sample is compacted to an approximate height of 10 inches. It is compacted in 10 equal layers, each layer receiving 20 blows with the 10-pound tamper dropped free from a height of 18 inches.

Compaction Control Test Specified by the Corps of Engineers

22. The California static compaction control test was not adopted by the Corps of Engineers, since it was not generally accepted or used by other agencies. Based on a comparison of compactive efforts from limited data available, it was assumed that the results obtained by the AASHO (Proctor) compaction test, modified by the Engineer Department so as to increase the compactive effort, would be sufficiently close to those obtained by the California impact control tests. Therefore, the modified AASHO (Proctor) compaction test was specified as the Engineer Department control test.

Early Modifications to California Method of Preparing Remolded Specimens for Design Test

23. The first change to the California procedure for preparing test specimens appeared in Chapter XX of the Engineering Manual, as published in June 1942, which required the use of the modified AASHO compaction test as a control, and required that CBR test specimens be prepared under 2000 psi static load at an optimum water content predetermined by the modified AASHO compaction test.

24. After a short period of compacting the samples for CBR design tests in accordance with this procedure, it was evident that:

a. The densities of the samples being prepared for the CBR design test were at considerable variance with those specified for construction control.

b. The test results on poorly graded clean sands and cohesionless silts were not valid compared to results on plastic soils, because of the inability to obtain required field densities or compaction curves for sands using static compaction.

25. In view of the variations obtained by use of the partially modified California method described above and its limited applicability, it was considered necessary to further modify the method of preparing remolded specimens to design tests so that when penetrated they would actually reflect the relative stability of the soil as

compacted during construction and as later affected by moisture changes. Two methods of compacting the soil for design CBR tests were therefore considered. They were:

   a. A static load method using a variable load to produce the density desired.

   b. An impact method similar to that adopted for field compaction control tests (modified AASHO test).

Method b was selected to compact the samples for the design test, since (1) the method was similar to that used for compaction control tests by the Engineer Department; (2) the equipment for the impact method was available, and (3) loads greatly in excess of 2000 psi were required to compact sandy samples to the density desired. Laboratories were not equipped with static load equipment with capacities required and, due to the war, such equipment could not be obtained.

26. The use of the impact method for preparation of specimens for design tests was submitted as a second modification by the Office, Chief of Engineers to the field offices in September 1942. It was unfortunate that investigations could not have been conducted to determine the effect of these changes before submitting them to the field. It was assumed, however, at the time, that the main factor in the preparation of the specimens for design tests was to produce the density as set forth in the Engineering Manual.

27. At this time, the use of a surcharge during penetration was introduced into the test procedure, to make the test more applicable for use with cohesionless materials. During the test the penetration surcharge was specified to be equal to the anticipated overburden in the field.

28. For cohesionless soils it was specified that a drainage period of 15 minutes should be allowed for removing any free water remaining on the surface of the specimen after soaking and to prevent any surface disturbance or softening, which would result in obtaining low CBR values.

29. The California Division of Highways, as well as several other state highway departments and agencies, had found through experience and field observation that subgrade and base course soils (except clean sands) under impervious pavements usually increased in water content by capillarity and condensation of moisture, regardless of the ground-water elevation, and in some cases became nearly saturated. Although it is recognized that this maximum moisture condition will not occur below all pavements, it is impossible to accurately predict the ultimate moisture conditions. Therefore the Corps of Engineers specified design for maximum moisture conditions. For this reason it was stipulated that the CBR design penetration test should be performed on specimens containing the maximum water content obtained by soaking. However, provisions were

made so that the total pavement thickness could be reduced 20 percent for ideal subgrade conditions and for subgrades with low moisture. Later, due to abuse of this provision, it was necessary to remove the allowance, to prevent general underdesign.

Comprehensive Laboratory Studies

30. In view of the fact that still further modification in the procedure for preparation of test specimens and in the CBR test appeared desirable, a program of investigation was formulated in November 1942, to be conducted at the U. S. Waterways Experiment Station. The principal objectives and findings of this investigation together with results obtained from special and general field reports were used to make modifications to the CBR procedure from time to time after the initial modifications just described. Chapter XX of the Engineering Manual was revised in March 1943 and was supplemented by a circular letter, subject "California Bearing Ratio Procedure," to all Division offices in May 1944.

PART IV: MEANING AND LIMITATIONS OF THE CBR TEST AND ITS APPLICATION TO DESIGN

General

31. This part discusses and describes the establishment of design criteria and specification requirements for subgrades and base courses with which the CBR is usually identified. The limitations of the CBR test are presented, with fundamental rules for its use utilizing the results of the laboratory studies described in detail in Appendix A and available field experience.

Factors Controlling the Use of the CBR Test

Description of the CBR penetration test

32. The CBR test is a penetration shear test used to determine a modulus of the shearing resistance of soils. In this report the CBR test is considered to be only the penetration portion of the procedure and does not include the preparation of remolded test specimens. The CBR test is not a classification test, since it will give different results for the same soil compacted to different water contents and densities under different methods of compaction. This is not a weakness of the CBR test but means that care is necessary in the preparation of samples to reproduce field conditions. Similar trends are obtained in the shearing resistance of soils as measured by the triaxial compression and unconfined compression tests. Since the shear characteristics of the soil mass change with change in water content, density and method of compaction, test results as reflected by CBR or any other type of shear test should also change. The test is considered valid only when a large portion of the deformation under penetration is shear deformation.

33. The CBR test may be performed on samples remolded and compacted in the CBR mold, or on undisturbed samples secured from compacted fills or cut areas. A field in-place CBR test can be performed with apparatus described in Appendix C. The test can be made on samples in either the unsoaked or soaked condition and can be used as either a design or evaluation test (see definitions in Part I) according to the purpose for which the test results will be used. The modulus of shearing resistance obtained from any given test is divided by a standard modulus obtained on crushed rock, to obtain the CBR value. This value is then used in conjunction with empirical design curves to determine the total thickness of base course and pavement to protect the underlying

soil against detrimental shear deformation. The detailed step-by-step laboratory penetration test procedure is given in Part VI.

Shearing resistance of soils

34. Undisturbed soils. Many soils, particularly cohesive soils, have a natural structure which imparts strength to the soil mass. When the natural structure is destroyed by remolding, even at the same moisture and density, loss of shear strength results. Some materials lose a major portion of shear strength, while others are little affected. In some cases, compaction of natural subgrade soils causes more loss of strength than the remolded soil gains due to increase in density. In these cases the pavement should be constructed on an undisturbed subgrade and design tests should be made using undisturbed samples.

35. Compacted soils. Certain shear strength characteristics are found in compacted soils which vary with compacting moisture content, general field moisture conditions subsequent to construction, and density, which will be further discussed below. These variations have been investigated in the laboratory for static and dynamic compaction and the effect of these factors on the shearing resistance of laboratory compacted soils is large. Little is known at present of the effect produced by field compaction, but it may be assumed that a similar condition exists. Therefore, CBR tests must be performed on undisturbed samples taken from compacted fills, or field in-place tests must be made to determine whether laboratory test results are representative for field conditions.

36. Shearing resistance of soils during traffic compaction. During traffic, particularly with wheel loads of 15,000 pounds or greater, a large amount of compaction will occur in loose subgrades or base courses. Experience has shown that, depending on the size of the wheel load, material to a considerable depth will be compacted to densities in the range of 90 to 100 percent of modified AASHO. This compaction occurs by decrease in the volume of voids in the soil. In saturated soils which have little or no cohesion and are not free draining, such as fine sands or silts, a part of the load during traffic compaction is carried by water trying to escape from the voids and a large loss of stability may occur. Under such conditions, shear tests may not be directly applicable for design.

37. Soils subject to frost action. Soils subject to frost action suffer a loss in strength, due to increase in water content and decrease in density. Thus in areas where such soils may be adversely affected by frost action, other considerations than shear strength as shown by the test methods established herein may control the design of the pavement.

38. Free-draining cohesionless sands and gravels. Under traffic compaction, free-draining sands and gravels consolidate to a density equivalent to about 100 percent of modified AASHO compaction without detrimental shear deformation. Therefore, the CBR test performed on loose sands is not believed to be applicable to the design of a pavement and base, and design tests should be performed on specimens compacted to 100 percent of modified AASHO density.

39. Summary. Since the CBR test is a shear test and not a classification test, it should not be used where factors other than shearing resistance control the design. The results of CBR tests performed in the manner described in this report can not be directly used for design if frost action will cause loss of strength after construction or if an appreciable loss of strength due to development of pore pressure under traffic will occur. In clean free-draining sands and gravels, CBR tests should be made on samples compacted to 100 percent modified AASHO density.

Rules for Adequate Design

40. Adequate design requires that specifications and design criteria be set up which can be met in the field and which will result in a stable structure taking full advantage of available materials. Methods for testing materials in the laboratory must be devised which duplicate field conditions. Specifications and laboratory and field testing procedures must be established which are coordinated with construction procedures and with field experience.

41. To use the CBR test in the design of flexible pavements, one of the following procedures must be followed:

a. Assume CBR values, to determine a design which is based on the best practical knowledge and experience, which apply to the airfield under consideration. During construction, perform tests either on undisturbed samples with the moisture conditions adjusted to the degree of saturation to be ultimately anticipated in the field, or make field in-place tests. If the field construction and compaction procedure does not give results comparable to those obtained in the laboratory, change the design or improve the field procedure such that the CBR test values on the soil in question agree with those assumed for design.

b. Produce test specimens in the laboratory with the same conditions of water content, density and structure expected in materials to be compacted in the field. Perform tests on compacted or undisturbed samples where applicable, adjusting the water content to the degree of saturation to be ultimately expected in the field. Determine the design requirements, using the test results from these specimens. During construction, perform tests on undisturbed samples with the moisture conditions properly adjusted, or field in-place tests. If the CBR values obtained during construction are lower than the design CBR values, every effort should be made to improve the construction methods in order to increase the CBR, based, of course, on the predication that the tests made during construction correlate directly or are equal to the values obtained by the laboratory procedure. If this can not be done, then the design should be changed to agree with the field results.

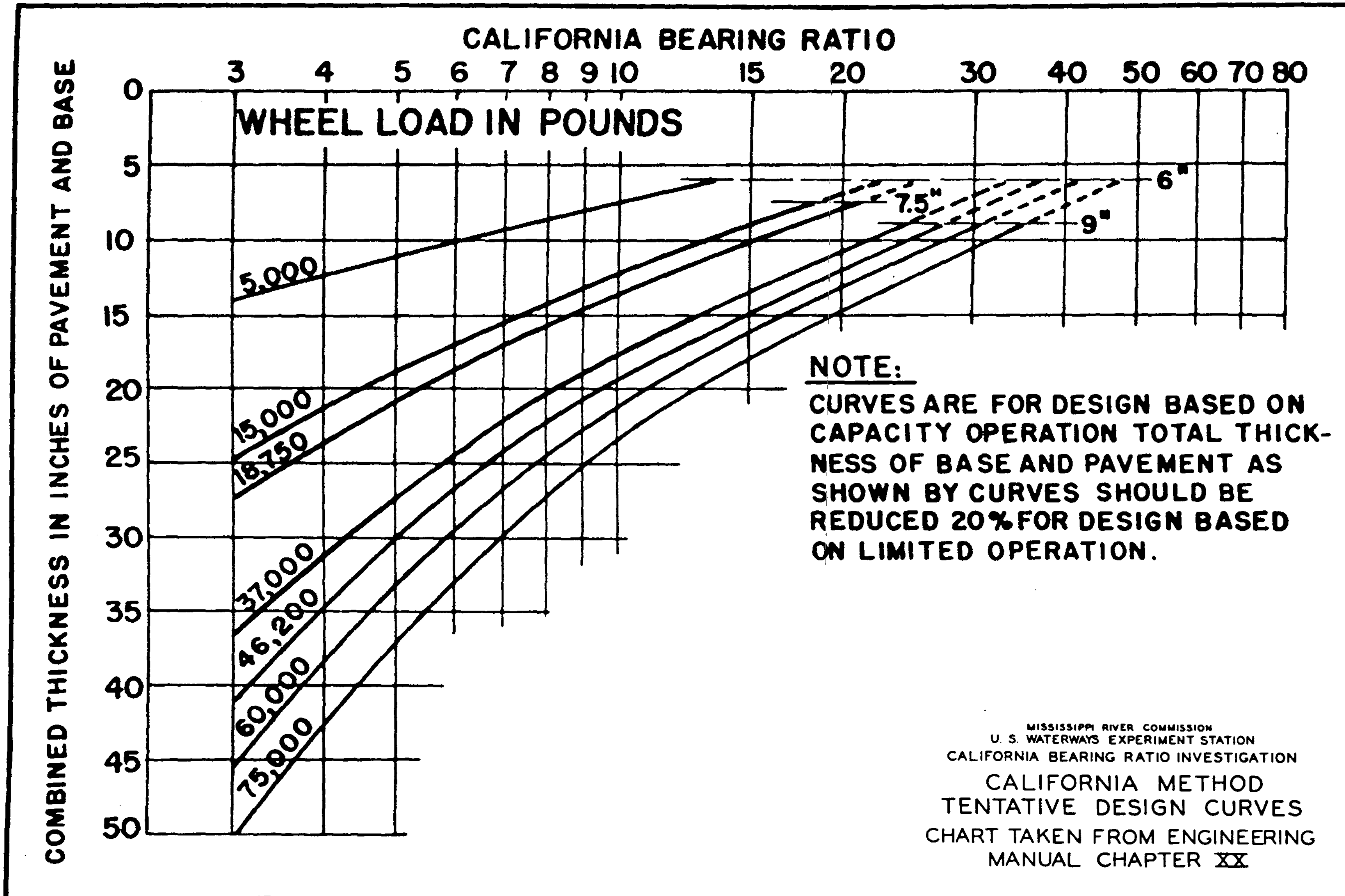
To make the California method practical for design, procedure b, above, is followed. In making field in-place tests, it is not practical to adjust the moisture condition to that ultimately expected. Therefore field in-place tests must be compared to laboratory tests on unsoaked samples and the effect of soaking determined by comparison of soaked and unsoaked laboratory tests.

Moisture criterion

42. Experience and field observation have shown that subgrade and base course soils (except clean sands) under impervious pavements will increase in water content by capillarity and condensation of moisture regardless of the ground-water elevation. There is no way at present to predict what degree of saturation will ultimately be reached at a given site. Therefore the moisture condition for design, as stated in paragraph 29, is required by the Engineer Department.

Example of Design by the California Method

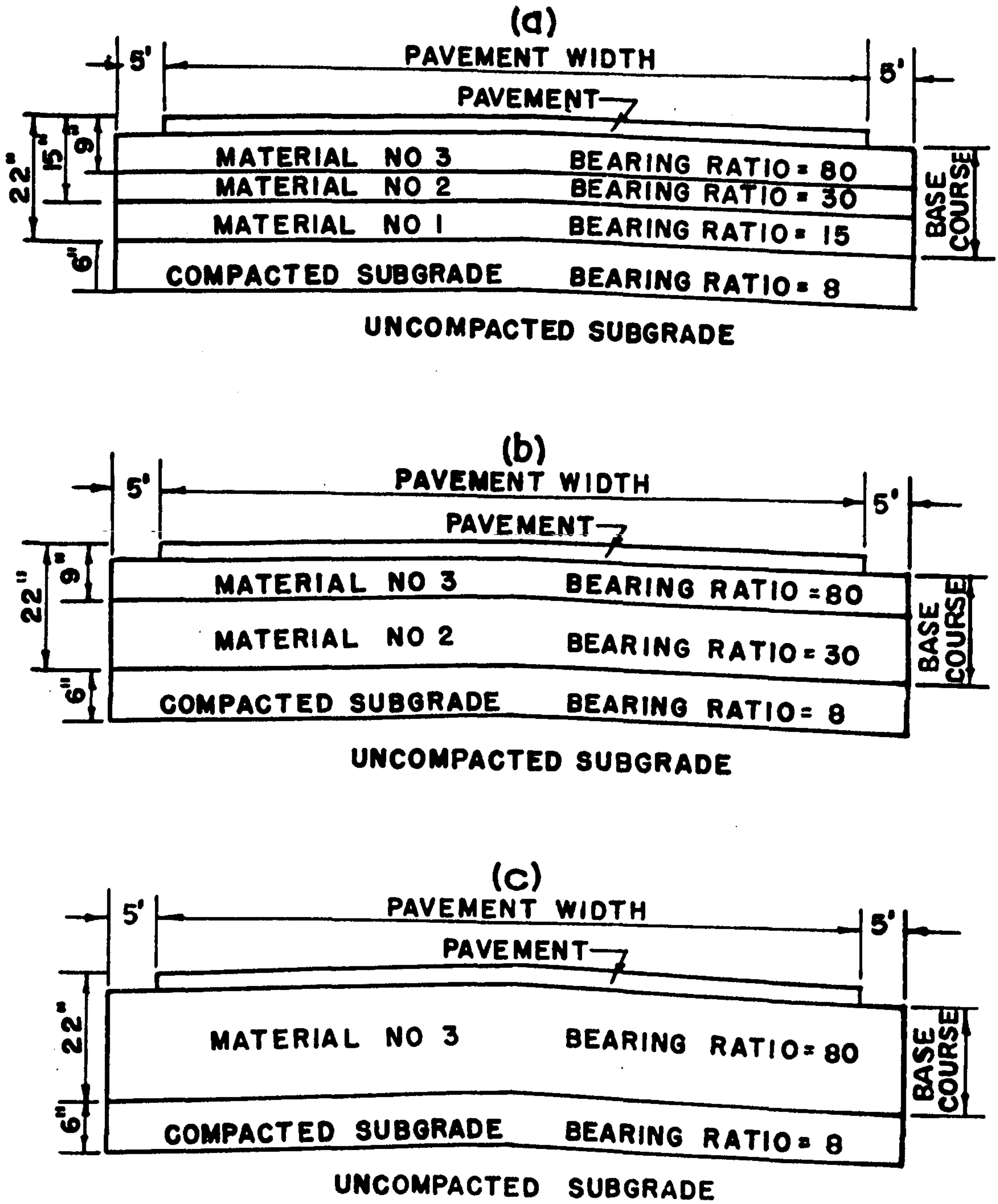
43. To show the analysis of design by the California method when the subgrade or base course soils are not affected by frost, or when detrimental settlement or development of hydrostatic excess pressure are not the principal considerations in design, assume that a main taxiway is to be designed for capacity operation with a 37,000-pound wheel load, and that the top 6 inches of subgrade will be compacted. To compensate for increase in deflection in the pavement, base and subgrade, caused by slow-moving or standing planes, the loads given for capacity operation should be increased 25 percent on the design of turnarounds, hardstandings, taxiways and aprons. The 37,000-pound wheel load therefore corresponds to a design load of 46,200 pounds. The CBR of the compacted subgrade and of the materials available for base course construction are as follows:



WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

FIGURE 1



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CALIFORNIA BEARING RATIO INVESTIGATION

DESIGN EXAMPLE

FIGURE TAKEN FROM ENGINEERING MANUAL CHAPTER XX

FIGURE 2

Material	Soil Group Casagrande Classification	CBR of Samples at Unit Weights and Moisture Conditions Expected in Prototype - in Percent
Compacted subgrade	CL	8
1	SF	15
2	SP	30
3	GW	80

The total thickness and thicknesses of the various base course layers are determined as follows:

a. Total thickness. The total combined thickness of the base course and pavement will be governed by the bearing ratio of the compacted subgrade. From the curves on figure 1 the required total thickness of base course and pavement above the compacted subgrade (CBR of 8 percent) is 22 inches.

b. Thickness of base course layers. The total combined thickness of 22 inches of base course and pavement may be composed of materials 1, 2, 3, and wearing course (pavement). The design thickness of each layer of material will depend upon the relative cost of construction and the bearing ratio of each material. The first step in design is to determine the individual layer thickness required with reference to its location in the structure, if all three materials are used. Material 1, which has the lowest CBR, would form the lower layer, and material 3, which has the highest CBR, would form the upper layer. The minimum depth of more stable material required above a layer of material 1 is 15 inches, corresponding to a CBR of 15 percent according to the curves shown on figure 1. Likewise, the minimum depth required above a layer of material 2 (CBR of 30 percent) is 9 inches (see paragraph 44, below, for explanation of minimum depth). If the cost of placing material 1 is the least and that for material 3 is the highest, the most economical base course design would be as shown in figure 2-a. However, if economical, the base course might also be designed using only material 3, or materials 2 and 3 in combination, as shown in figures 2-b and 2-c, since material with a higher CBR may be used in place of a material with a low CBR. Field experience and the results of traffic indicate that, although there may be some differential between the behavior of layered base courses as indicated in figures 2-a and 2-b and a base composed of high quality material throughout as shown in figure 2-c, this differential is fairly small. On the traffic tests in which this factor was studied, the differential in required thickness of base course was within the limits of experimental error for the test conditions. Therefore no differential in design thickness between the two types of base course is used at this time. Under this assumption, the design method outlined above allows the rapid investigation of the economical advantages derived from the use of locally available materials.

44. Base course immediately under pavement. The base course immediately under the pavement should be sufficiently stable to withstand the high stresses produced in the zone directly under the wheel of a plane. The required stability is dependent upon the type and thickness of pavement, the action and effect of a moving or skidding wheel, the type of plane, et cetera, and can not be determined by the curves shown on figure 1. For highway pavements used by heavy trucks, experience has shown it is desirable for the base course material immediately under a bituminous pavement to have a CBR of at least 80 percent. Observations and tests to date indicate that a 6-inch base course with a CBR of at least 80 percent placed directly under bituminous pavements of the minimum thickness will have satisfactory stability. This minimum thickness of pavement should not be less than 1-1/2 inch for wheel loads up to 15,000 pounds; not less than 3 inches for wheel loads up to 37,000 pounds, and for wheel loads exceeding 37,000 pounds the thickness should be designed in accordance with the requirements in each specific case, but in general should not exceed 6 inches.

PART V: DISCUSSION OF LABORATORY PROCEDURES

General

45. Certain factors which directly affect the CBR penetration test and several factors which entered into the preparation of remolded specimens for the penetration test were investigated in the laboratory. These factors were studied for the purpose of modifying test methods or determining the validity of the test results. Methods of taking undisturbed samples and preparing them for test were also studied. The above factors and procedures, all of which affect the CBR test results directly or indirectly, are described below.

CBR Penetration Test Procedure

Penetration surcharge

46. As stated in Part III, the Corps of Engineers modified the original procedure by requiring the use of a surcharge load on test specimens during penetration. On cohesionless soils which have high internal frictional resistance, the penetration surcharge greatly affects the test results by confining the material and mobilizing the shearing strength of these soils. Inasmuch as unreasonably low CBR values are obtained on cohesionless soils when penetrated without the surcharge weights, it was deemed highly necessary that the surcharge be used. Since the line of demarcation between cohesionless and cohesive soils was difficult to establish, it was desired to use a penetration surcharge on all soils, provided it did not greatly affect the results on plastic soils. The results of the laboratory studies indicated that the CBR value for cohesive soils was not greatly affected by a penetration surcharge. It was therefore concluded that a penetration surcharge should be used for all soils.

47. Magnitude of penetration surcharge. The magnitude of the penetration surcharge, as stated above, greatly affects the CBR value on cohesionless materials. However, its effect on cohesive materials is small and the more plastic the material the smaller the effect. On cohesive materials which exhibit high swell, the swell which occurs during soaking has a major effect on the CBR value and, therefore, the soaking surcharge, which acts to reduce swell during soaking, likewise has a major effect. In view of the above, it is necessary that the penetration surcharge be equal to the soaking surcharge, so that the test will be made on samples which are not undergoing active volume change.

48. Each surcharge should be equivalent to the weight of overburden anticipated in the field, except that it should not be less than 10 pounds, which is the equivalent of approximately 6 inches of overburden in the field. Examination of figure 1 shows that a comparatively large change in the CBR value above a CBR of 10 percent causes little change in the total combined thickness of base course and pavement. In this range of CBR values, it is believed that the surcharge weight should duplicate the anticipated overburden weight only to the nearest 5-pound increment. On soils in a condition in which it is expected the CBR will be below 10 percent, a small change in CBR, for instance from 3 percent to 5 percent, will greatly affect the total thickness of base and pavement required. Laboratory data show that a maximum change in the actual CBR value of about 1/2 percent occurs with a 5-pound change in surcharge. This latter condition is not an unreasonable variation, however, because normal differences between tests for reasons other than surcharge may cause this much change. It is therefore considered that it is not necessary to control the surcharge weight more closely than to plus or minus 5 pounds for all soils. The overburden surcharge should be estimated from the estimated CBR value and if this assumption does not check what the tests actually obtain, the penetration test should be repeated on a sample which has the correct equal soaking and penetration surcharge.

49. On soils in a condition in which it is expected a low CBR value will be obtained, it is advisable to apply the penetration piston and penetration surcharge weights in either one of two ways, to prevent upheaval of the soil through the hole in the surcharge weights before placing the piston. In the first method, one 5-pound annular disc surcharge weight should be applied to the soil surface, the penetration piston then seated with a 10-pound load and finally the remainder of the surcharge applied by the use of slotted 5-pound surcharge weights. In the alternate method, a special locking and alignment device as shown in figure 6 can be used.

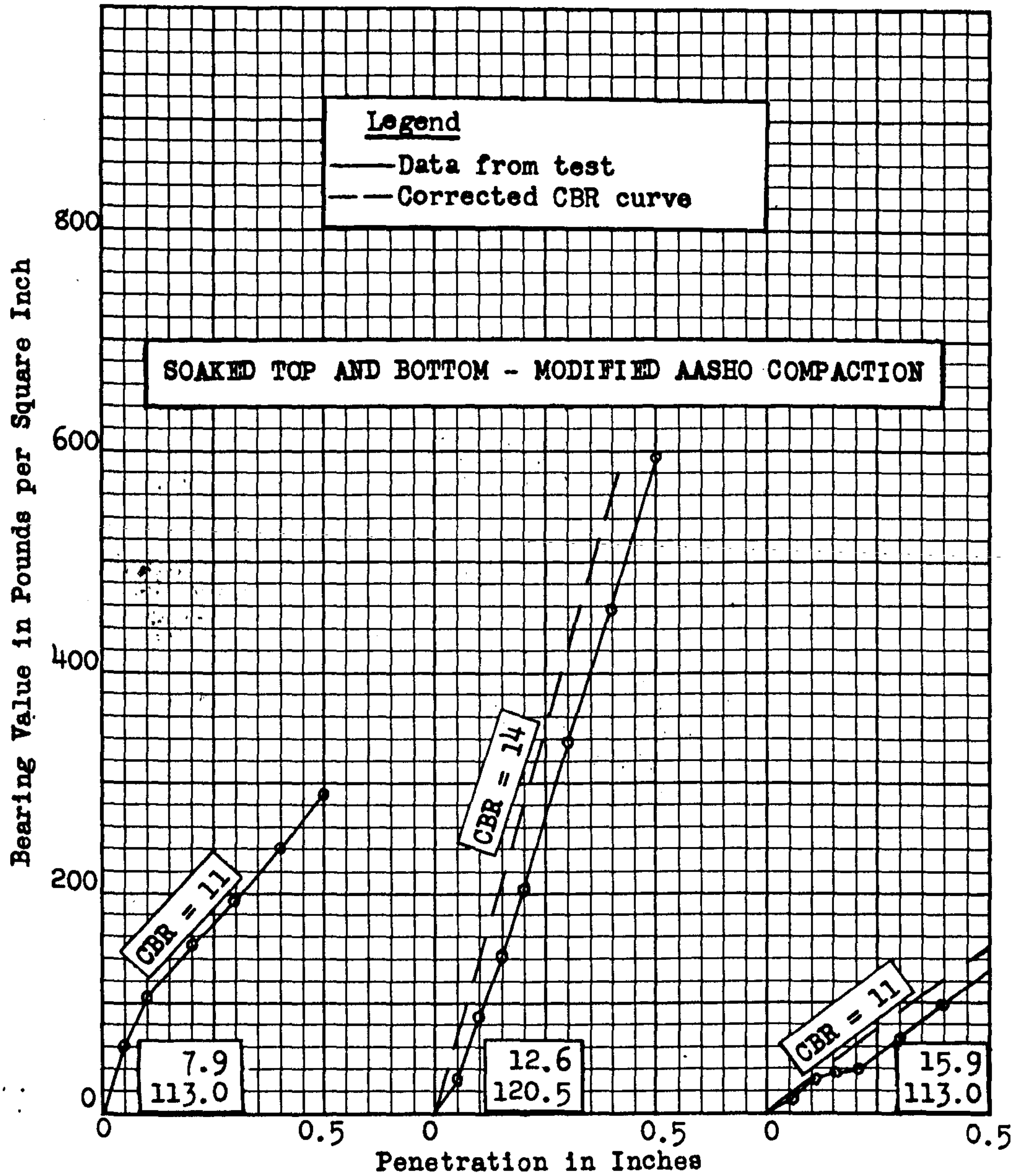
Rate of penetration

50. Tests were conducted in the laboratory, using various rates of penetration to determine whether a "quick" condition would be produced for saturated cohesionless materials during a fast rate, or if consolidation would occur for more plastic type soils when a slow rate of penetration was used. According to the test results it is concluded that the present rate of 0.05 inch per minute is satisfactory.

Correction of stress-penetration curves

51. Tests performed in the early part of the laboratory study and tests conducted by other laboratories showed that two types of stress-penetration curves were being obtained with dynamic compaction on soils exhibiting low swell. One was concave-upward and the other concave-

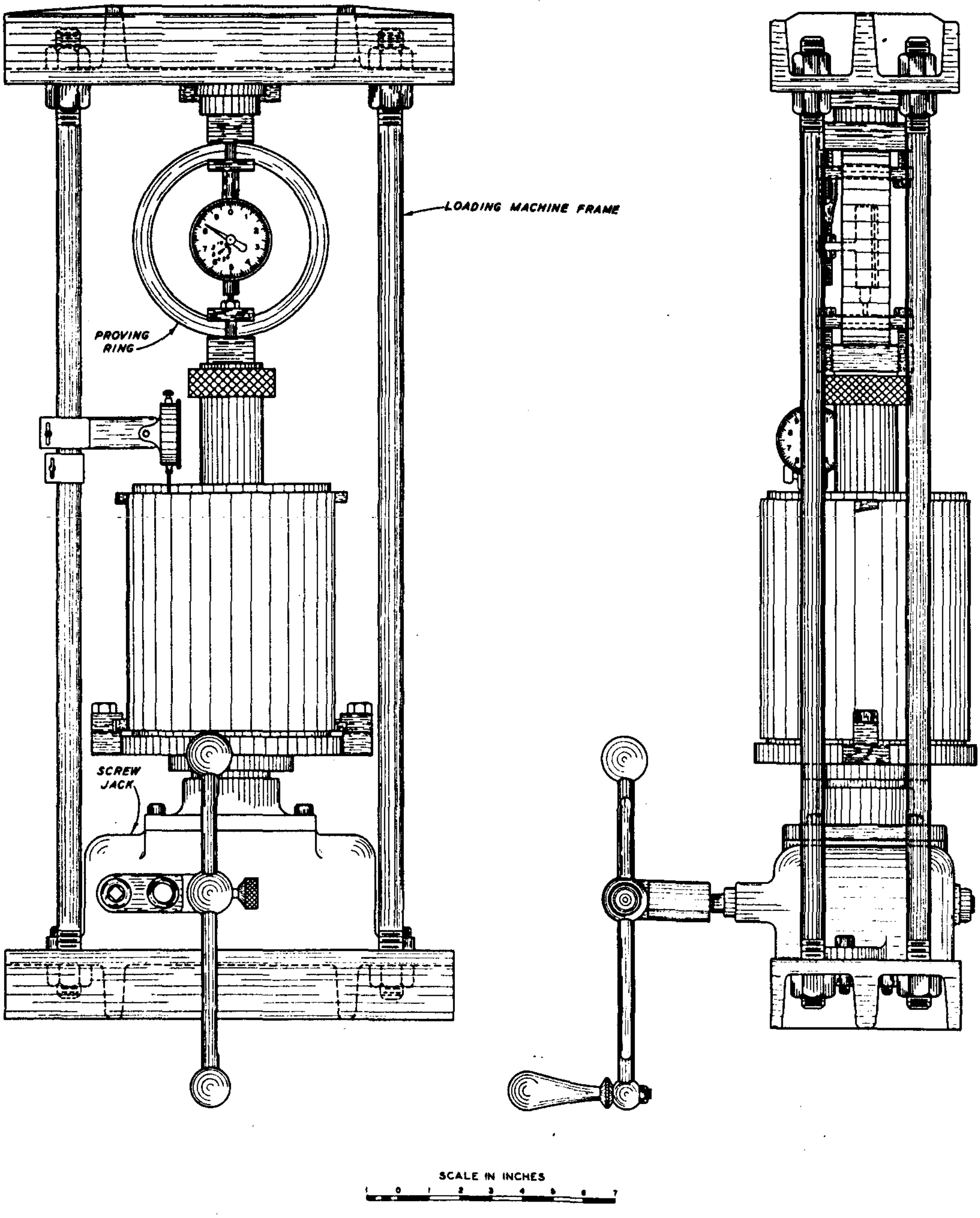
Notes: Figures at bottom of curves indicate molding water content and dry density, respectively.
Surcharges: 10 - 0



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

METHOD OF CORRECTION OF STRESS PENETRATION CURVES

Sample 18 - Berry Field Lean Clay



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CALIFORNIA BEARING RATIO INVESTIGATION

ASSEMBLY-LABORATORY LOADING MACHINE
USING SCREW JACK AND PROVING RING

FIGURE 4

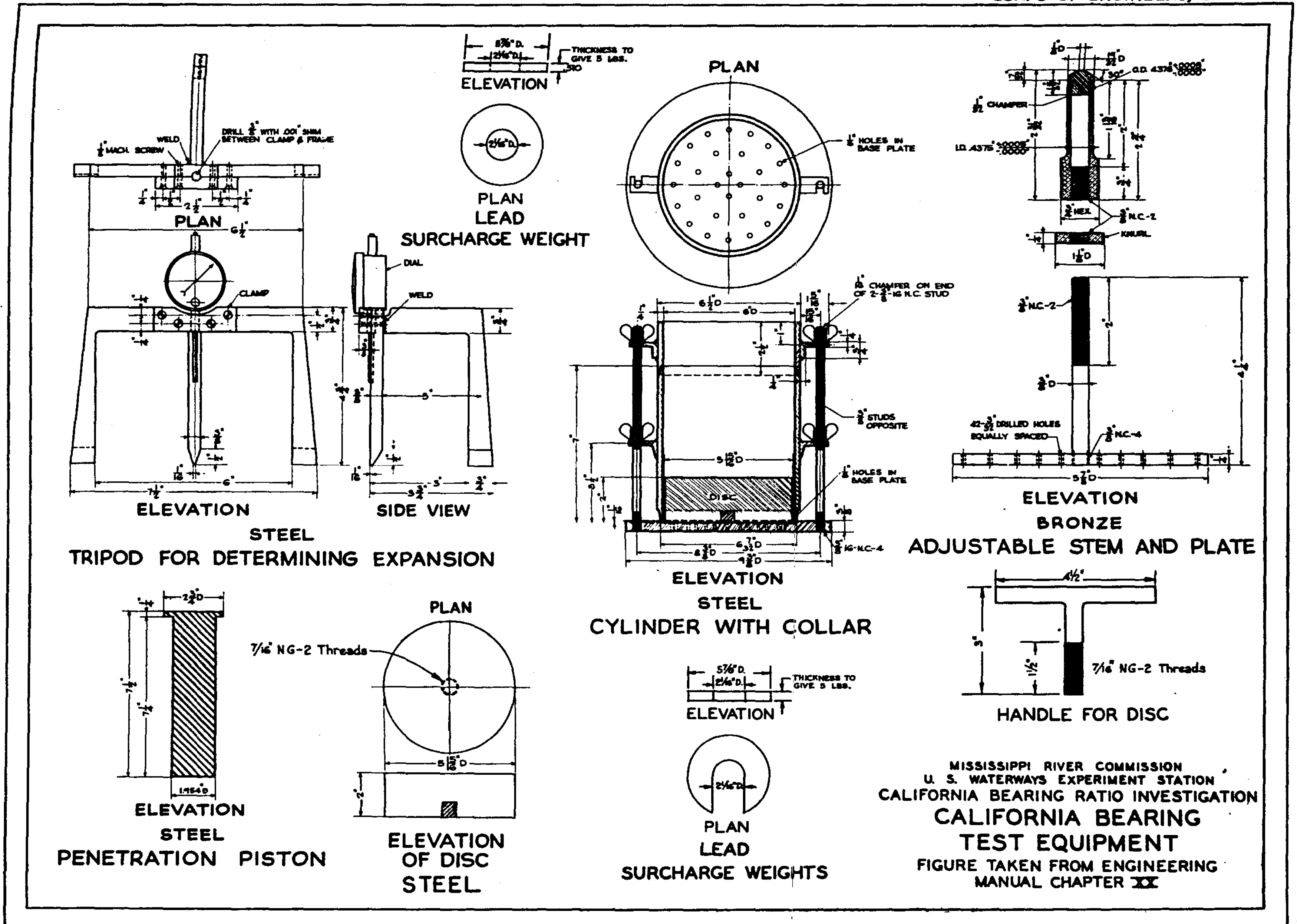
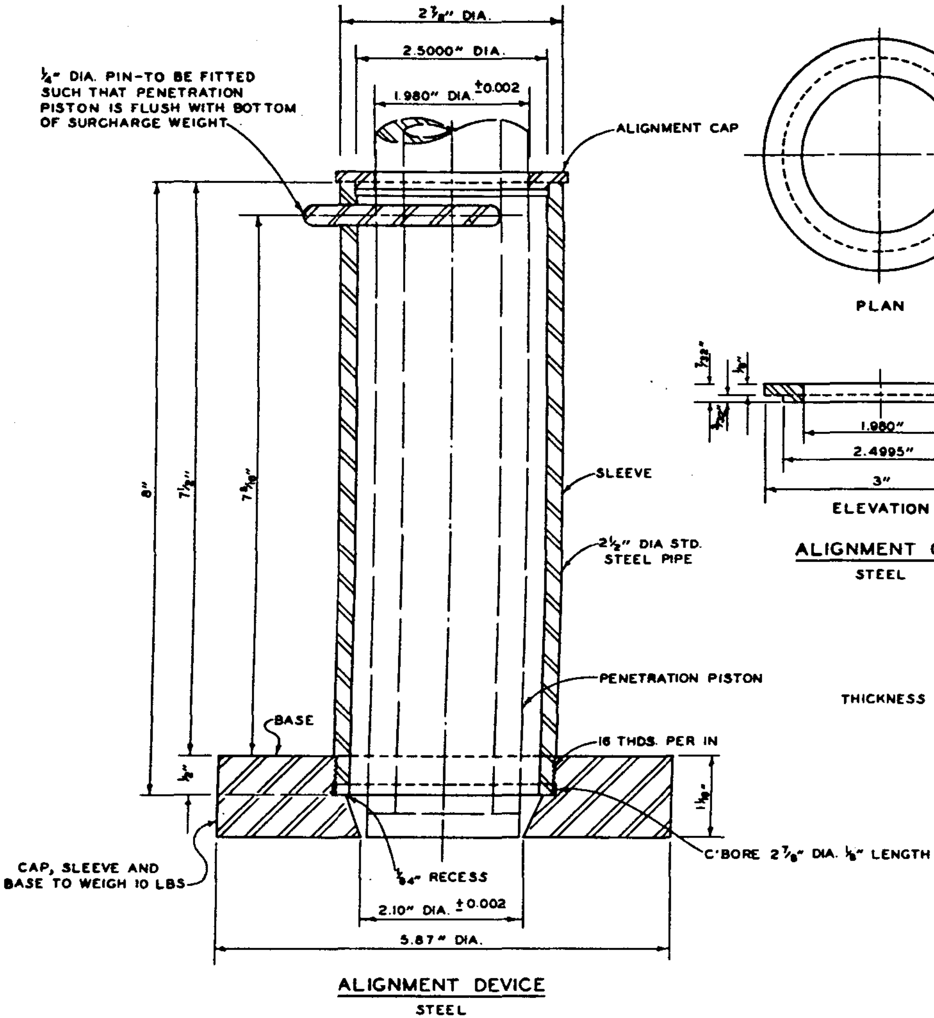
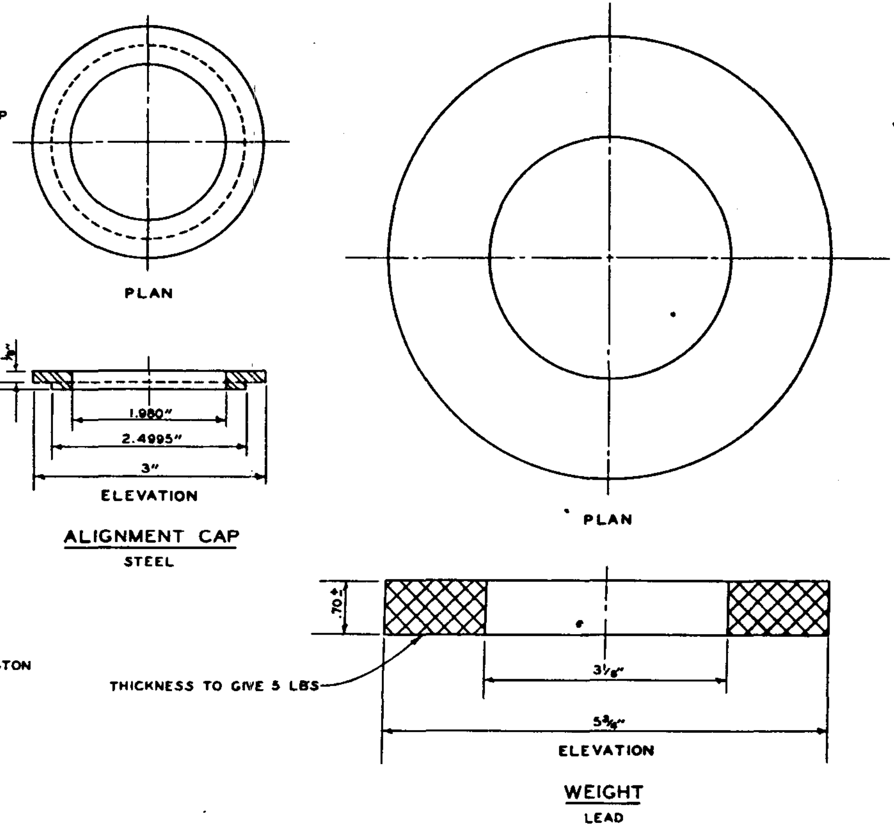

PLAN
ELEVATION
SIDE VIEW
STEEL
TRIPOD FOR DETERMINING EXPANSION
DIAL
WELD
CLAMP
ELEVATION
THICKNESS TO GIVE 5 LBS.
PLAN
LEAD
SURCHARGE WEIGHT
PLAN
HOLES IN BASE PLATE
STUDS OPPOSITE
DISC
ELEVATION
STEEL
CYLINDER WITH COLLAR
CHAMFER
KNURL
ELEVATION
BRONZE
ADJUSTABLE STEM AND PLATE
HANDLE FOR DISC
7/16" NG-2 Threads
ELEVATION
STEEL
PENETRATION PISTON
PLAN
ELEVATION OF DISC
STEEL
ELEVATION
PLAN
LEAD
SURCHARGE WEIGHTS
MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION
CALIFORNIA BEARING TEST EQUIPMENT
FIGURE TAKEN FROM ENGINEERING MANUAL CHAPTER XX

FIGURE 5

FIGURE 6



ALIGNMENT DEVICE
STEEL



WEIGHT
LEAD

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CALIFORNIA BEARING RATIO INVESTIGATION

LOCKING AND ALIGNMENT DEVICE FOR PENETRATION SURCHARGE WEIGHTS AND PISTON

SCALE IN INCHES
1 0 1 2 3

downward over about the first 0.1 inch of the penetration range. These are illustrated on figure 3. It was at first believed that this was due to a soft surface condition which was established during soaking, and a method of correcting the curves was adopted to allow for this condition. The method of correction consisted of extending the portion of the curve with a maximum slope over 0.1 inch penetration to zero stress and establishing a new origin. For the purpose of this investigation, it was assumed that the controlling CBR value was at 0.1 inch penetration, which allowed the use of the following method of correction. The correction, as shown on figure 3, is made by drawing a line through the origin parallel to the steepest slope maintained for any 0.1 inch of penetration and selecting the bearing value at the intersection of this line with the line at 0.1 inch penetration. In the case shown on figure 3, the curve for molding water content of 12.6 percent would correct from a CBR at 0.1 inch penetration of 9 percent to one of 14 percent.

52. Later studies showed that similar stress-strain curves were obtained in triaxial and unconfined compression tests on specimens compacted dynamically, and that the CBR stress-penetration curves for specimens compacted statically and specimens compacted dynamically at water contents dry of optimum seldom required correction. It was therefore concluded that the concave-upward shape of the stress-penetration curve was actually due to the lack of rigidity of the soil mass and correction was possibly not warranted. However, until more data can be obtained, it is believed advisable to continue to correct the stress-penetration curves, and corrections have been made throughout this report where curves concave in the upward direction were obtained.

Equipment

53. Experience and observations indicate that a steady rate of penetration is needed. It is therefore considered that the usual type of single-acting hydraulic jack is unsatisfactory, due to the surge effect created by the pumping operation. It is recommended that a laboratory testing machine, capable of producing a constant rate of strain, or a screw jack and proving ring arrangement similar to that shown on figure 4 and plates 158 and 159 be used. Other equipment for the penetration test, as shown in figures 5 and 6 and discussed in Part VI, is considered satisfactory.

Gravelly samples

54. The presence of gravel in samples will produce erratic test results. However, laboratory studies so far conducted do not yield sufficient data to warrant formulation of a new method of testing gravelly samples. Until further investigation can be made, all samples used in the laboratory for design tests should be processed as specified in Part VI, and it is recommended that several tests be made, to allow an average representative test result to be determined, and that this average result be utilized in design. Field in-place tests on

gravelly materials should also be sufficient in number to allow an average representative result to be obtained.

Factors Controlling the Preparation of Laboratory-Compacted Specimens

55. In preparing laboratory-compacted specimens for any type of physical test, such as consolidation, shear, CBR or permeability, it is necessary to duplicate conditions expected in the field. Originally it was thought that density was the only factor that had to be considered in preparing laboratory-compacted soaked and unsoaked specimens for the CBR test. However, the comprehensive laboratory studies showed that not only density, but also molding water content and the method of compaction were prime factors to be considered in the preparation of soil specimens of low plasticity for the penetration test. In the case of free-draining cohesionless soils, it was found that the density was the main factor, whereas for high-swelling cohesive soils molding water content and density were the main factors.

56. The results of the laboratory studies showed that variations in CBR design test results were largely due to the method of preparing test specimens. The variations, however, were found to be systematic and were caused, as stated above, by molding water content, density and method of compaction. It was found that consistent laboratory results could only be obtained when these variables were given full consideration. Of particular interest was the shape of the CBR stress-penetration curves which, under static compaction, were practically always concave-downward, whereas under dynamic compaction, except for clean sands and high-swelling clays, the curves started concave-upward. These concave-upward shaped curves were practically always obtained on specimens compacted near optimum and on the wet side of optimum, both for the as-molded and soaked condition. Another pertinent observation was the fact that under static compaction these intermediate soils between clean sands and impervious high-swelling clays always swelled when soaked, whereas under dynamic compaction they did not.

57. In order to determine whether the same variations in shearing resistance as indicated by CBR tests would also be indicated by other types of physical tests, triaxial and unconfined compression tests were conducted on a few typical soils ranging from sands to clays. The results of these tests showed that molding water content, density, and method of compaction controlled the shearing resistance of remolded soils in a manner similar to the way they controlled the CBR. Thus it is apparent that, no matter what method of test is used to determine the shearing resistance of soils, great care must be taken in preparation of test specimens to avoid erratic results.

58. Inasmuch as the molding water content is such a prime factor in controlling the physical properties of all except free-draining soils it follows that, in remolded soils, duplicate laboratory specimens can not be prepared, unless the same molding water content and method of compaction are duplicated, even though water contents and densities obtained subsequent to molding are duplicated. In other words, if a soil is molded at some given water content and then this water content is allowed to increase or decrease by a given amount, another identical soil specimen can not be reproduced, unless the whole cycle is reproduced, starting with the same molding water content.

59. The combination of molding water content, density, and method of compaction results in a certain type of compacted soil system being formed, which may or may not retain the same general physical properties in the soaked state as obtained in the as-molded state, depending on the amount of swell. The variations in shearing resistance obtained on low to medium plastic soils exhibiting little or no swell and on high-swelling soils investigated in the laboratory studies are explained in a postulation set forth in Part IV of Appendix A. The postulation covers the variations obtained from the standpoint of differences in the structural arrangement of the components in the compacted soil system.

60. Another factor entering into the preparation of test specimens for the CBR penetration test is the height of the specimen. The laboratory studies showed that it appears advisable to use a minimum height of 4-1/2 inches for all soils, in order to eliminate the influence of the rigid base plate on the bulb of pressure created by the penetration piston.

61. An extreme increase in CBR values for some soils at densities greater than standard AASHO density may be partly due to the confining effect of the 6-inch diameter mold. Present indications are that the 6-inch diameter mold is not large enough at high densities (modified AASHO) on soils exhibiting less than 3 percent swell during soaking, or for base materials containing 3/4-inch particles. However, it is not considered advisable to change mold size without considerable further investigation.

Types of Compaction for Remolded Specimens

Laboratory compaction methods studied

62. Two general methods were used for compacting specimens during the laboratory studies: static and dynamic. Static compaction consisted of placing the required amount of soil in the CBR mold and slowly adding load to it by means of a piston actuated by a testing machine. (Some laboratories used a hydraulic jack mounted in a frame.) The load

was added at about 0.05 inch per minute and the maximum load was left on for one minute. Adding a constant load to several specimens at different water contents will yield results which can be plotted on a curve of molding moisture versus density, so as to obtain for some soils an optimum water content for that effort.

63. The dynamic compaction method used consisted of compacting the soil into a cylinder under a given number of free-falling blows of a given height with a hammer of given weight and given striking area. Dynamic tests were performed in both the standard Proctor compaction cylinder, which is approximately 4 inches in diameter, and the standard CBR cylinder, which is 6 inches in diameter. Both molds were placed on a concrete floor during compaction, in order to obtain a firm support. The number of blows of the tamper to use was determined by test, so that comparable densities were obtained at a given molding moisture content. The results described in this report were, however, obtained principally from compaction tests made directly in the CBR mold.

Comparison of static and dynamic compaction characteristics

64. The laboratory studies showed that medium to high plastic soils and pumice obtained maximum densities and minimum optimum water contents using the Porter static (2000 psi) procedure, whereas cohesionless and low plastic soils obtain maximum densities and minimum optimum water contents when the modified AASHO procedure was used. Under dynamic compaction, practically all soils developed compaction curves with definite optimum water contents, which agrees with field experience. Under static compaction, the medium and high plastic soils were the only ones for which a compaction curve with a definite optimum could be developed. In addition it was found that static compaction on low plastic soils did not produce compaction curves with a definite optimum water content. This is contrary to field experience.

65. It appears from the studies that it is possible to obtain a uniform density gradient under static compaction. However, to accomplish this uniformity, the load must be applied once to each end of the specimen, or an apparatus must be used which would allow a movable piston at each end of the specimen during compaction. Under dynamic compaction it appears practically impossible to establish a standard procedure of a constant number of blows per layer, or of staggering blows to give uniform density, even if each soil type were considered individually, because of the fact that change in molding water content causes change in density gradient.

66. Breakdown of cohesionless particles under both methods of compaction, although not excessive for the soils tested, is nevertheless undesirable. Because of this breakdown and because other soils having enough cohesion to do so may retain the density in small lumps from the previous compaction, even though the soil mass is thoroughly kneaded,

material should not be used more than once in establishing each point of the water-content density curve. Neither should material be re-used in remolding specimens for any soil test.

Recommended
compaction method

67. The following factors were considered in selection of the dynamic method for preparation of remolded test specimens:

a. From the standpoint of furnishing control and design data on optimum moisture and density, and because the establishment of a definite optimum water content agrees with field experience, dynamic compaction is considered to be of more general applicability than static compaction.

b. Although the molding moisture content has a very large effect on the physical properties of the soil mass under dynamic compaction, the variations due to molding moisture are systematic and can be controlled in the laboratory by proper procedure in the preparation of specimens for test.

c. The equipment required for dynamic compaction is relatively simple, as compared with heavy loading equipment required for static compaction.

68. Since dynamic compaction is recommended, because of its overall general applicability, some means should be developed for accurately and rapidly determining the density of the top two inches of every specimen penetrated in the CBR test. This problem deserves more study than time has allowed in this investigation. Also needed is a comparison of the compaction and strength characteristics of materials compacted by the laboratory dynamic method with those obtained on the same soils compacted in the field by standard compacting equipment, such as tractor drawn sheepsfoot rollers and heavy rubber-tired equipment.

Control of Molding Water Content and Density in Preparing Laboratory-Compacted Specimens for Design Test

Control compaction tests
for remolded samples

69. For consistent and uniform results whenever the modified AASHO method of compaction, or any other dynamic compactive effort, is specified for the preparation of test specimens for the CBR penetration test, the compaction test should be performed in the 6-inch diameter CBR mold on materials processed in accordance with paragraph 95-a, below. Each layer of material should be given 55 blows, to duplicate modified AASHO

effort, and a corresponding number of blows per layer, to duplicate any other specified effort. In addition, the mold should be placed on a concrete floor or pedestal during compaction of the soil, and material should not be re-used.

Materials exhibiting
high swell (3% or more)

70. On soils exhibiting excessive swell, the effects of molding water content are not reflected in the test results on soaked specimens. The laboratory studies show that, on high plastic soils showing excessive swell, such as California adobe clay, the spread of soaked CBR values is small, regardless of the molding water content or type of compaction, as shown on plate 78 in Appendix A. However, although the spread of values is small, those specimens compacted dynamically on the dry side of optimum had the lowest CBR value, and consequently compaction on the dry side must be avoided if swell is to be kept to a minimum.

71. Since these soils will require compaction in the field on the wet side of optimum, in order to minimize swell, comparable compaction tests should be duplicated in the laboratory, in so far as is possible. Observation of swelling tests, after compaction of several specimens over a range of water content, may serve as the basis for field control, and remolded specimens for design tests should be selected from the results of the initial swelling tests, to duplicate anticipated field conditions.

Free-draining
cohesionless sands

72. This group of soils includes cohesionless materials which will compact without detrimental shear under the anticipated traffic to maximum density as specified by the modified AASHO method previously discussed in paragraph 38 of Part IV. For these soils the modified AASHO compaction test should be performed and it is only necessary to conduct the CBR penetration test on duplicate test specimens which have been compacted to 100 percent of modified AASHO density and at the optimum water content for this effort. Consequently, it may be possible to use one of the original compaction specimens for the penetration test if it is at 100 percent modified density.

Soils of low plasticity
exhibiting little or no swell

73. This group of soils is intended to include all soils other than those described above. The results of CBR and unconfined compression tests on these soils indicated that the shearing resistance obtained in the as-molded or soaked condition is greatly affected by

molding water content, as previously discussed. Inasmuch as these soils are so sensitive to molding water content, it follows that a definite control in the preparation of test specimens is needed and, as a result, several methods were investigated which are described in the paragraphs below.

74. Constant number of blows to obtain 95 percent modified AASHO density. In order to save time and to simplify methods for preparing specimens at specification density, it was decided during the course of the investigation to determine whether there was any one dynamic compactive effort which would obtain 95 percent of modified AASHO density at the optimum water content for all plastic soils. Usually, laboratory samples must be compacted to this density for design tests, since this is the density generally specified for field compaction of subgrade and base material. The results of this study, which was a cooperative study between various Engineer Districts and the Experiment Station, are covered in Appendix B. These tests showed that the dynamic compactive effort required to obtain 95 percent of modified AASHO maximum density was a function of the plasticity of the soil; the required compactive effort increasing with increase in plasticity. For 26 blows per layer using the modified AASHO method, a variation from 93 to 98 percent of modified Proctor density was obtained. This variation was considered too much and therefore a constant number of blows was not recommended. Since this method was unsuccessful, one of the two following procedures is required:

75. Method 1. It is recognized that during the war, lack of time and equipment very often prohibit following the procedure recommended in Method 2. In these instances, Method 1, which consists of a minimum number of tests, should be used. This method, which is described in detail in Part VI, consists first of performing a carefully controlled modified AASHO compaction test and rigidly establishing the optimum water content by the curve. Three specimens are then prepared for the design CBR penetration test by compacting each at modified AASHO optimum water content under a different dynamic compactive effort. The maximum allowable variation in molding water content should be not more than plus or minus 0.5 percent. From these data a curve of CBR versus molded dry density as shown on figure 7 can be obtained which will permit the determination of the design CBR based on the expected field density.

76. As can be seen, Method 1 is suitable for preparation of test specimens when specifications call for 95 percent of modified AASHO density. This approach can be used, because quite often higher densities will be obtained in the field, although specifications require a minimum of 95 percent of modified AASHO density. If some other density is specified, the procedure must be changed.

77. It is emphasized that this method will give satisfactory results in the laboratory only when the molding water content is closely controlled within the tolerance shown above and it is desired for specimens to be compacted in the field at 100 percent of modified AASHO

optimum water content. Since the tests are performed for only one molding water content, no indication (either quantitative or qualitative) is given as to how this soil group will behave if placed in the field at any water content other than that for which the laboratory tests are performed.

78. Method 2. In general, this method, which is described in detail in Part VI, must be followed, in order to obtain a complete picture of variations due to molding moisture. The results of this method of test permit a full study of the variation of CBR with density and water content, and the approach for arriving at a CBR value to be used in design on this group of materials is as follows:

a. Assume a practical working range of water content and density to be expected in the field.

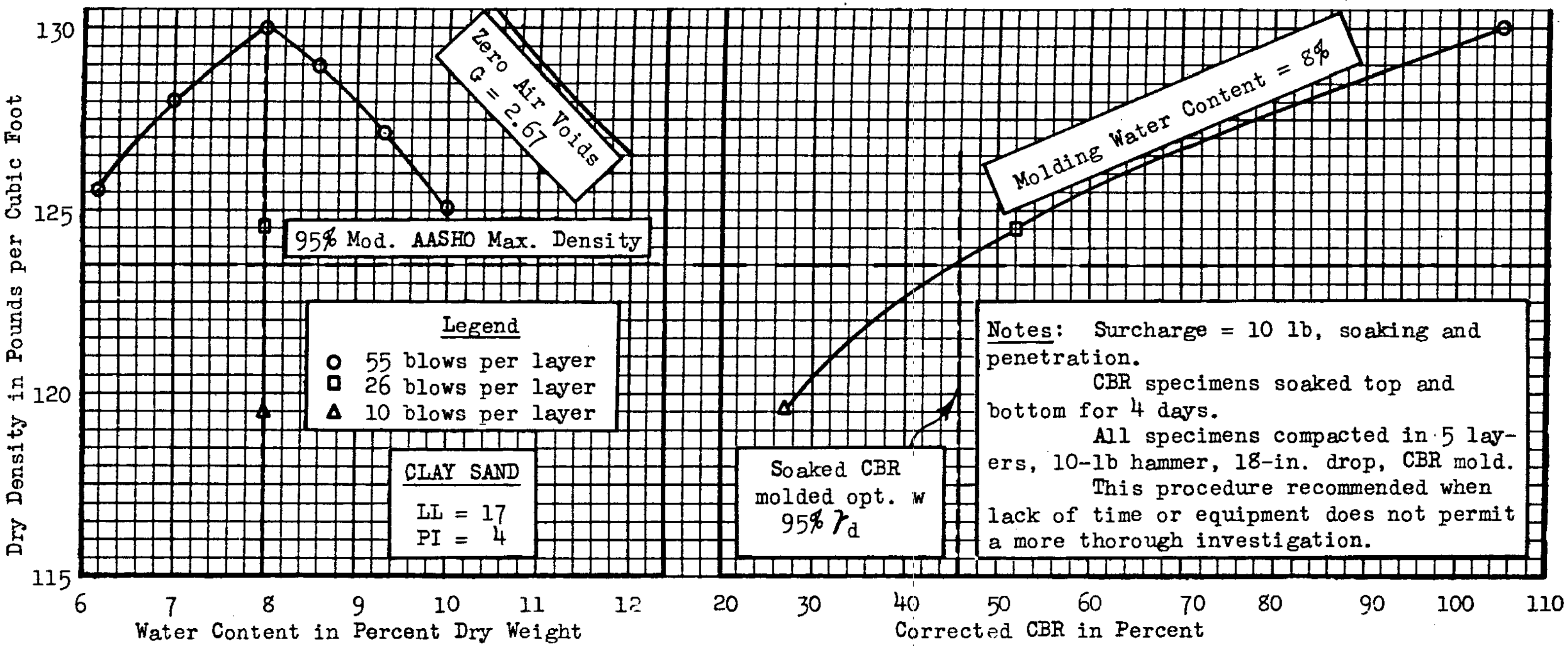
b. Perform penetration tests on all specimens used in the development of compaction curves for three compactive efforts. The efforts used should cover the assumed working range of density and water content, so that the variation of CBR within this range can be established. All specimens should be soaked prior to penetration.

c. From these data a family of curves can be developed as shown on figure 8 and the CBR value selected which is most nearly representative of the probable field conditions. The minimum value may be the controlling value, or it may be possible to use higher values if field conditions appear favorable.

79. The validity of the results obtained by the above procedure is dependent upon the ability of the laboratory method of compaction to produce specimens whose water content, density and soil system duplicate those of the field compaction method. It is not known whether present compaction methods do this. Until additional research on this correlation can be accomplished, the test results as obtained by the above method should be used with the full understanding that the variations obtained may be only qualitatively valid.

Method of Soaking Specimens

80. As stated in paragraph 29, the Corps of Engineers requires soaking of CBR design test specimens, in order to simulate maximum anticipated moisture conditions. Three methods of soaking were investigated in the laboratory studies: soaking from bottom only, from top only, and from top and bottom. These tests showed that soaking from the bottom only was impractical, because of the time required, and that it did not give CBR values comparable to those obtained by soaking from top

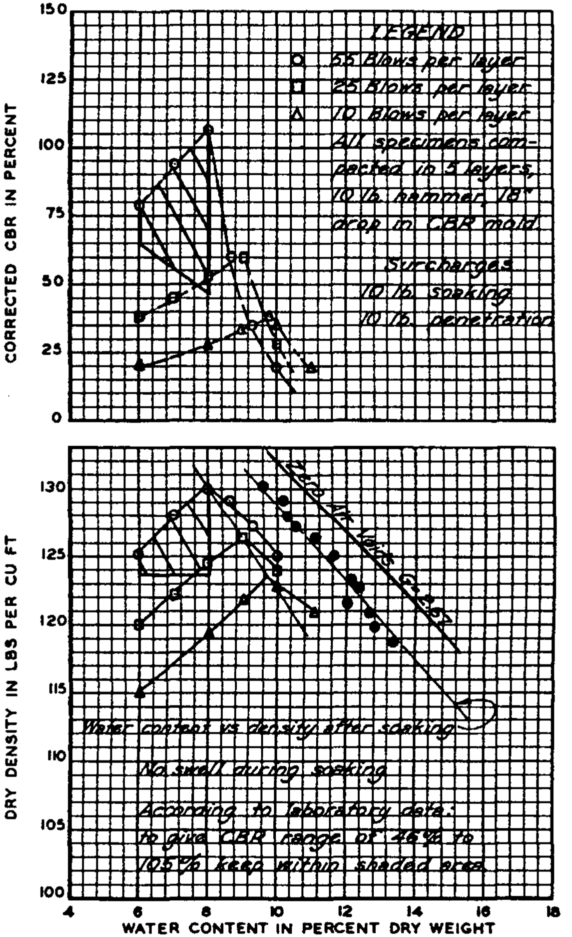


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

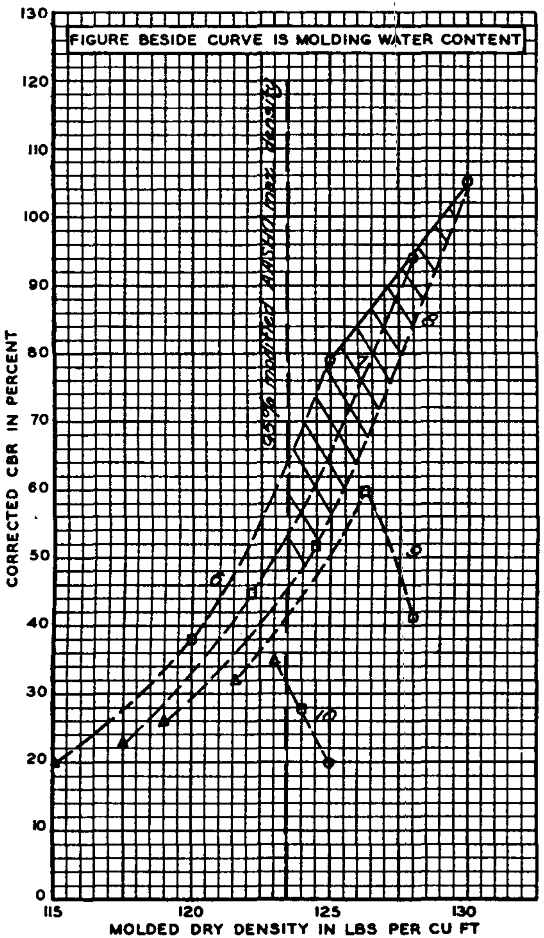
RECOMMENDED PROCEDURE FOR PERFORMING LABORATORY TESTS FOR DESIGN
(Assumed Specified Field Control: Minimum of 95% Mod. AASHO Max. Density at Mod. AASHO Opt. w)
METHOD 1

FIGURE 7

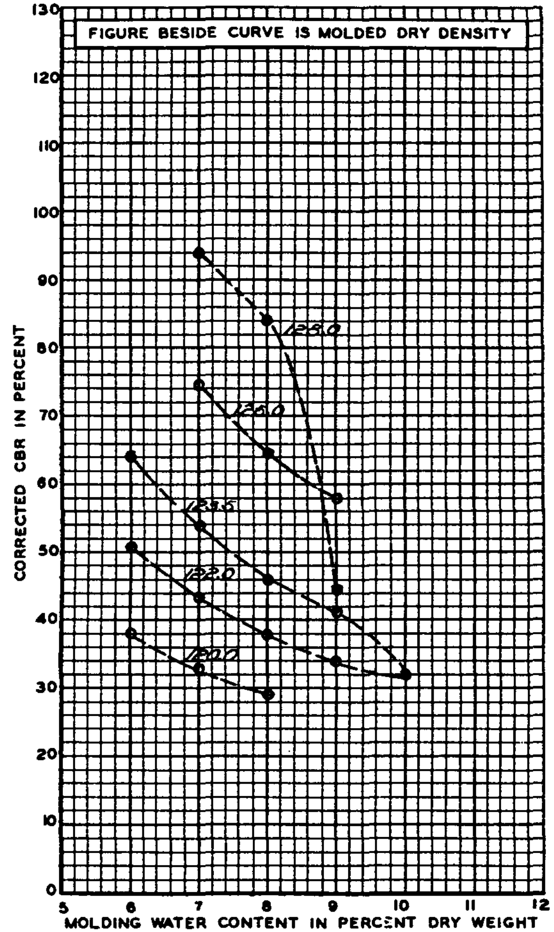
ALL SPECIMENS SOAKED TOP AND BOTTOM FOR FOUR DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

MATERIAL: CLAY SAND
LIQUID LIMIT: 17
PLASTICITY INDEX: 4

ASSUMED SPECIFIED FIELD CONTROL:
MINIMUM DRY DENSITY = 95% MOD. AASHO MAXIMUM
WATER CONTENT RANGE = 6% TO 8%

NOTE: THIS PROCEDURE RECOMMENDED WHEN A THOROUGH INVESTIGATION IS DESIRED AND CAN BE ACCOMPLISHED

METHOD 2

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

RECOMMENDED PROCEDURE FOR PERFORMING LABORATORY CBR TESTS FOR DESIGN and bottom, as originally recommended by Porter. Soaking from the top only likewise did not produce results comparable to those in the original method. Soaking top and bottom therefore appears the most practical method and is recommended. Additional tests also indicated that a soaking period longer than four days is unnecessary. The water content gradient obtained by soaking from the top and bottom results in a lower moisture content in the middle portion of the sample, but this does not appear to materially affect the test results, probably because the principal effect of the test is on the top inch of the sample.

81. The soaking surcharge would affect the CBR only if it prevented swell or caused consolidation during soaking, and its effect increases with increase in plasticity of the soil. It greatly affects medium to high plastic soils, moderately affects low plastic soils, and has practically no effect on cohesionless soils in the range of water contents and densities for the soils tested.

82. In order to remove all free water from the top of sandy samples, the surface of which would be easily disturbed in removing the base plate and spacer disc, a drainage time of 15 minutes should be allowed after soaking. This time appears sufficient for all of the soils tested.

Comments on Field Compaction

83. It is not known how closely the stress-strain characteristics and shearing resistance of specimens obtained by laboratory compaction duplicate those obtained in the field by present construction methods, but the information which has been obtained to date appears to bear out laboratory experience in ranges of CBR values below about 10 percent. These data are described in Part VII and are extremely limited. Unfortunately, no other data are available at this time, but the Engineer Department has started field compaction investigations which will obtain more data on this subject. Until such time as these data become available, it appears, from the results of laboratory test data, that it is advisable to compact fills of low cohesive soils exhibiting little or no swell at slightly dry of optimum water content for the compaction equipment being used, in order to take maximum advantage of the strength which can be gained by compaction.

Methods of Taking Undisturbed Samples for CBR Tests

General

84. The obtaining of undisturbed samples from natural subgrades or compacted fills for any type of testing requires considerable care and patience, if disturbance is to be kept to a minimum. Undisturbed

sampling of materials for the CBR test is even more critical, in that the test must be performed on a 6-inch diameter specimen supported uniformly on the sides. If the proper lateral support and retention are not provided on the sides of the sample, the CBR value for unsoaked specimens may be low, and on soaked specimens lateral swelling may occur, which will affect the specimen very much differently from the swell which will occur in a specimen compacted directly in the mold. Therefore, special means must be taken to provide suitable support for undisturbed samples as taken, because the usual procedure of pushing a CBR mold with special cutting edge into the ground has not proven entirely satisfactory.

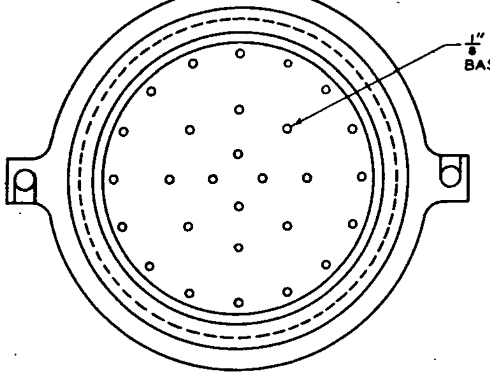
Equipment

85. Any one of three types of sampling equipment can be satisfactorily used for sample containers: the 7-inch mold shown on figure 9, the 7-inch galvanized metal containers shown on figure 10, or cubical wooden boxes, usually of about 10 inches inside dimensions, with removable top and bottom. The first two sample containers are usually used for fine-grained materials and the third for samples containing gravel.

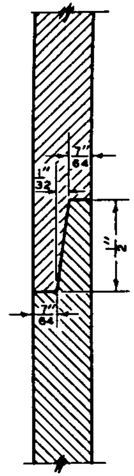
Sampling methods

86. In order to obtain suitable samples, a test pit should be excavated, leaving a pedestal in the center about 12 inches square. If the 7-inch mold is to be used, the mold with cutter attached is placed on top of the pedestal and forced gently into it, trimming away soil from around the cutter so as to avoid the use of a large load to force it into the material. If this is carefully done, the volume of the sample can be computed from the size of the cutting edge and the length of the sample. The annular space between the sample and the 7-inch mold is filled with paraffin, or a mixture of paraffin and 10 percent rosin, which the South Atlantic Division has found to support the sample better than straight paraffin. The sample is dug out and the top and bottom cut off flush with the mold and covered with wax paper and paraffin for shipment to the laboratory. The 7-inch mold can be used with the oversize base plate and the collar shown on figure 9 to obtain soaking from the top and bottom, if it is desired to soak the sample.

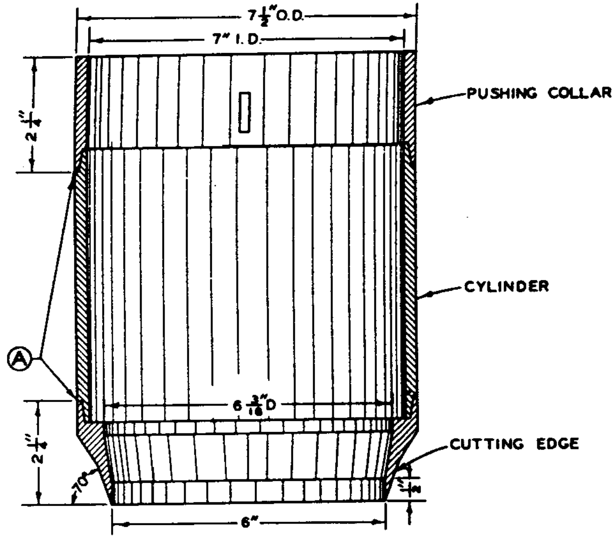
87. Sampling is performed in a similar manner with the metal jacket, except that the pedestal has to be trimmed by hand to approximate size and the density has to be determined by a displacement method taking into account the weight and volume of the paraffin or by nearby field density determinations. If it is desired to soak the specimen, the top paraffin can be removed and the sample soaked and penetrated from the top.



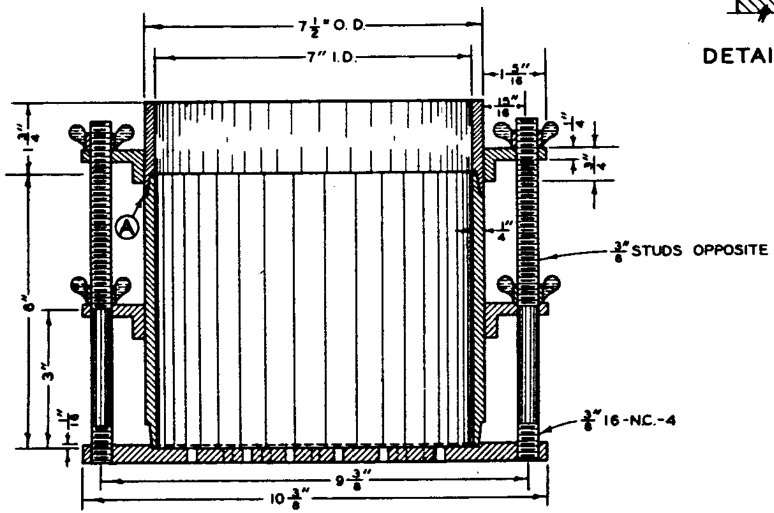
PLAN



DETAIL Ⓐ



CYLINDER WITH PUSHING COLLAR AND CUTTING EDGE



ELEVATION

CYLINDER WITH SOAKING COLLAR

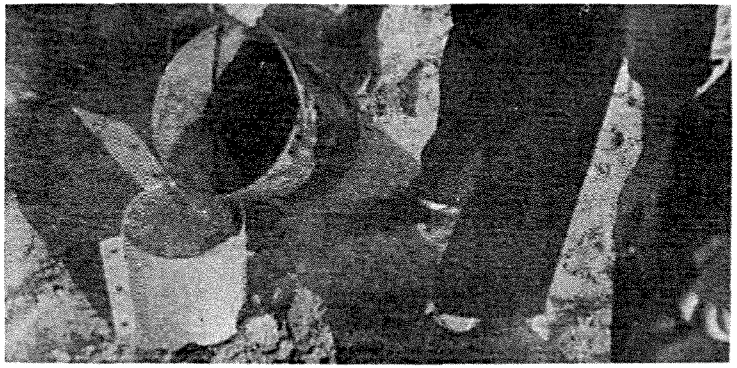
MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

EQUIPMENT FOR OBTAINING UNDISTURBED SAMPLES FOR CBR TEST

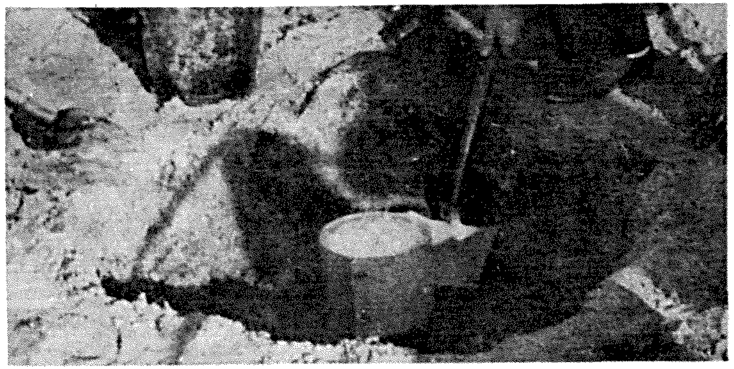
FIGURE 9

a. Test pit has been excavated and rough soil pedestal is being trimmed

b. Soil pedestal has been trimmed to final size, wax paper has been placed on top of the soil and a brush coat of paraffin and rosin mix has been applied to the wax paper. Galvanized container surrounds the pedestal and the paraffin-rosin mix is about to be poured around the pedestal



c. Paraffin-rosin mix being poured around and on top of sample



d. Paraffin-rosin mix has hardened and soil sample and container are being removed from test pit. Container will be inverted and bottom of sample will be trimmed, so as to allow 1/2 inch of paraffin-rosin mix to be poured in bottom of the container

Photographs courtesy of
U.S.E.O., Wilmington, N.C.

METHOD OF TAKING UNDISTURBED SOIL SAMPLE FROM TEST PIT

88. Sampling and testing methods using the box sample container are similar in all respects to those used with the metal jacket. The only advantage of this method is that a somewhat larger sample is taken and the effect of large gravel which may be encountered near the edge is minimized.

PART VI: RECOMMENDED PROCEDURES FOR PREPARATION OF TEST SPECIMENS AND PERFORMANCE OF THE CBR PENETRATION TEST

Equipment

Remolded samples

89. The equipment required for preparing and testing remolded specimens is listed below. Most of the major items are shown on figures 4 and 5.

- a. Cylinder mold 6 inches in diameter and 7 inches high provided with a collar extension about 2 inches long and a perforated base plate. The base plate and collar should fit and clamp on either end of the cylinder.

- b. ~~A disc~~ 5-15/16 inches in diameter and 2 inches high for insertion as a false bottom in the cylinder mold during compaction.

- c. A compacting hammer or tamper similar to that used in the modified AASHO compaction test (10-lb weight and 2-in. diameter striking face).

- d. Adjustable stem and perforated plate, tripod and dial gauge (reading to 0.001 in.) suitable to measure the expansion of the soil.

- e. One annular disc weight weighing 5 pounds and several slotted weights weighing 5 pounds each, suitable to apply as surcharge loads on the soil surface during soaking and penetration.

- f. Penetration piston 1.95 inches in diameter and approximately 7-1/2 inches high.

- g. Laboratory testing machine or screw jack and frame arrangement shown on figure 4, either of which can be used to force the penetration piston into the specimen at a rate of 0.05 inch per minute.

- h. Other general laboratory equipment such as mixing bowls, spatulas, straightedges, scales, soaking tank, ovens, moisture content boxes, et cetera.

For samples which become sufficiently soft to push up through the hole in the surcharge weights before placing the piston, a locking and alighment device for penetration surcharge weights and piston as shown on figure 6 can be used.

Undisturbed samples

90. The equipment required for obtaining and testing undisturbed samples is as follows:

a. In the case of samples to be obtained in steel cylinders approximating the size of the CBR mold, a special oversize cylinder, pushing collar, cutting edge, soaking collar and perforated base plate as shown on figure 9 will be required. The procedure to be followed in obtaining samples in these special cylinders has been previously described in Part V.

b. In the case of samples to be obtained in metal jackets approximating the size of the CBR mold, a special galvanized sheet metal jacket approximately 7 inches in diameter and 6 inches in height as shown by the photographs on figure 10 is used. The procedure for obtaining these samples has been explained in Part V.

c. In the case of samples to be obtained in boxes, cubical wooden boxes approximately 10 inches inside dimensions with removable top and bottom are usually used. The procedure to be followed in obtaining these samples has been described in Part V.

d. Wax paper, paraffin, rosin, spatulas, straightedges, digging tools, moisture content boxes, scales, ovens, soaking tank, etc.

e. Same as 89 d, e, f and g, above.

Field in-place tests

91. The equipment required to conduct field in-place CBR tests is listed and described in Appendix C.

CBR Penetration Test

92. Inasmuch as the actual penetration test procedure is constant and the same for all types of specimens, it will be described prior to methods of preparing remolded or undisturbed samples for the penetration test. The step-by-step procedure for this phase of the design test is as follows and should be used on field in-place, undisturbed or remolded samples after the testing surface has been prepared.

a. Apply a penetration surcharge on all soils sufficient to produce an intensity of loading equal to the weight of the base material and pavement (within ± 5 lb) if a pavement is to be constructed, which will overlie the soil in the prototype represented by the sample, except that the weight shall not be less than 10 pounds. The weight applied must be estimated and, if it does not produce the intensity present in the final design, the test should be repeated. If the sample has been previously soaked, the penetration surcharge should be equal to the soaking surcharge, which in turn should have been estimated and governed by the conditions described above. On soils in a condition in which it is expected a low CBR value will be obtained, it is advisable to apply the penetration piston and penetration surcharge weights in either one of two ways, to prevent upheaval of the soil into the hole of the surcharge weights. In the first method, one 5-pound annular disc surcharge weight should be applied to the soil surface, the penetration piston then seated with a 10-pound load and finally the remainder of the surcharge applied by the use of slotted 5-pound surcharge weights. In the alternate method a special locking and alignment device, as shown in figure 6 can be used.

b. Seat the penetration piston with a 10-pound load and set the dial gauge to zero. The purpose of a 10-pound load before starting the penetration test is to insure satisfactory seating of the piston and should be considered as the zero load when determining stress-strain relations.

c. Apply load on penetration piston so that the rate of penetration is approximately 0.05 inch per minute. Obtain load readings at 0.025, 0.050, 0.075, 0.1, 0.2, 0.3, 0.4 and 0.5 inch deformation. In using manually operated loading devices it may be necessary to take more load readings as an assistance in controlling the rate of penetration.

d. Determine the moisture content in the upper one inch and, in the case of laboratory tests, also for the entire depth of the sample.

e. The penetration load in pounds per square inch should be computed and the stress-penetration curve drawn. In order to obtain true penetration loads from the test data, the zero point of the curve should be adjusted to correct for the initial concave-upward shape if present.

f. Determine the corrected load values at 0.1 and 0.2 inch penetration. Next, determine the corrected California bearing ratio for 0.1 and 0.2 inch penetration by dividing the load at 0.1 inch by the standard load of 1000 pounds per square inch and the load at 0.2 inch by the standard load of 1500 pounds per square inch. Multiply each ratio by 100 to obtain the ratio in percent.

g. The California bearing ratio usually selected is at 0.1 inch penetration. If the CBR at 0.2 inch penetration is greater than that at 0.1 inch penetration, the test should be rerun. If check tests give similar results, the CBR at 0.2 inch penetration should be used.

Control Compaction Tests for Remolded Samples Prepared for CBR Penetration Test

93. Whenever the modified AASHO method of compaction is specified for the preparation of samples for the CBR penetration test, the compaction test should be performed in the 6-inch diameter CBR mold using 55 blows per layer on ~~materials as described in paragraph~~ 95 a, below. The mold should be placed on a concrete floor or pedestal and material should not be re-used.

Preparation of Remolded Specimens for CBR Penetration Tests

94. As previously explained, all soils prepared for the design test should have the same density and moisture conditions expected in the field. The following procedures are applicable for specifications of the Corps of Engineers.

Low plastic soils exhibiting little or no swell → para. 98

95. Method 1. The step-by-step procedure for preparation of remolded materials for test is as follows:

a. All material over 3/4 inch in size should be removed and replaced with an equal proportion of material between 0.18 inch (no. 4 sieve) and 3/4 inch in size.

b. Conduct control compaction tests with a sufficient number of test specimens to definitely establish the optimum water content for 100 percent of modified AASHO density. Four or five specimens should be compacted with water contents within plus or minus 2 percent of optimum water content so that the optimum condition can be rigidly established. The height of fall of the hammer must be carefully controlled and the blows must be uniformly distributed over the specimen. This procedure establishes the moisture content at which specimens for CBR tests should be molded.

c. For the CBR tests the mold should be fitted with an extension collar and base plate. Clamp the mold with the fitted extension collar to the base plate and insert the spacer disc over the base plate. Place a 6-inch diameter coarse filter paper or wire mesh on top of the disc.

d. When results are required for a soil at 95 percent of modified AASHO density, three specimens should be compacted at the optimum water content for 100 percent of modified AASHO compaction, using a different number of blows for each specimen, i.e., at 55, 25 and 10 blows per layer. The maximum allowable variation in the molding water content should not be more than plus or minus 0.5 percent. Any specimens not falling within this range should be discarded and a new specimen compacted that does meet this requirement. If specifications call for other than 95 percent of modified AASHO density or other than 100 percent AASHO moisture, revisions must be made in this procedure in order to obtain specimens at the required density.

e. Remove the collar, trim the specimen, place a screen or a 6-inch diameter coarse filter paper over the top of the specimen and clamp a perforated base plate to the top of the test mold.

f. Invert the test mold, remove the base plate and spacer disc and determine the density of the specimen.

When soils are to be soaked, the following steps should be taken:

g. Place the adjustable stem and plate on the surface of the specimen and apply an annular weight to produce an intensity of loading equal to the weight of the base material and pavement within plus or minus 5 pounds, except the weight shall not be less than 10 pounds.

h. Immerse the mold and weights in water so as to allow free access of the water to the top and bottom of the test specimen. Take initial measurements for swell using the dial gauge and tripod. Allow specimens to soak for four days. A shorter period of time may be used for more pervious materials. Take final swell measurements at the end of the soaking period and compute the swell in percent of initial specimen height.

i. Take the specimen out of water and remove free surface water, taking care not to disturb the surface of the specimen. Allow the specimen to drain downward for 15 minutes. When removing surface water from impervious samples, it is necessary to tilt the samples. When this is done, the weights should be firmly held in place. The perforated plate and surcharge weights should then be removed and the specimen weighed. The specimen is then considered ready for the penetration test.

j. When three specimens are prepared as described in subparagraph d, above, the results of tests on all specimens should be plotted to show the relation between density and CBR, as illustrated on figure 7. For design purposes, the soaked CBR at 95 percent of modified AASHO density should be used.

It is emphasized that this method will give satisfactory results in the laboratory only when the molding water content is closely controlled within the tolerance specified above. Since the tests are performed for one molding water content, no indication (either quantitative or qualitative) is given as to how this soil group will behave if placed in the field at any water content other than that for which the laboratory tests are performed.

96. Method 2. CBR test results are affected by the density and molding water content of the soil specimens. The effects are great for some low plastic soils. It is recommended that the variation of test results with molding water content and density be determined for at least one or two typical soils encountered. The series of test specimens described below should be prepared and tested.

a. Prepare all specimens in a manner similar to that outlined under Method 1, above, except that each specimen used in the development of the 55-blow compaction curve should be penetrated. In addition, the complete compaction curves for the 25-blow and 10-blow per layer compactive efforts should be developed and each test specimen compacted should be penetrated. As previously stated, all compaction is performed in the 6-inch diameter CBR mold using the 10-pound hammer dropped 18 inches on each layer. It may be necessary to include an effort greater than 55-blow effort in the event heavier compaction is required. Attention is invited to Appendix B, which shows that a semilog plot of density versus compactive effort gives a straight-line relationship. This method of plotting compaction data is believed to be a valuable aid in determining the validity of compaction test data.

b. Plot the data from these tests as shown on figure 8. The above procedure is valuable to obtain test results on soils which are greatly affected by small changes in density and molding water content and gives a picture of the CBR characteristics, within the range of the field control expected, which will be useful in establishing the limiting CBR values. The test results, as obtained by the above method, should be used in connection with the design curves with the full understanding that the variations obtained may be only qualitatively valid.

Swelling soils

97. The procedure for preparation of these specimens is the same as Method 1, above, for low plastic soils, except that the test specimens should be prepared at a water content and density as specified on the basis of swelling tests. If 95 percent modified AASHO density is specified, specimens similar to those prepared in steps b and d under Method 1 for low plastic soils should be prepared, except that close control of water content is not necessary.

Cohesionless sands and gravels

98. This group includes cohesionless soils (P.I. less than approximately 2) which will readily compact under traffic to maximum density as specified by the modified AASHO method. Samples of sand which do not readily compact under traffic should be prepared as described in paragraph 95 for low plastic soils. The procedure for preparation of these specimens is the same as Method 1 under paragraph 95, except that only one specimen should be prepared at 100 percent modified AASHO maximum density for the penetration test. Ordinarily, soaking will not lower the CBR of cohesionless sands and gravels. In cases where this is determined, soaking should be omitted.

Preparation of Undisturbed Samples for CBR Penetration Test

Undisturbed samples in molds or jackets

99. If these samples are to be soaked, the method of soaking is the same as for remolded samples in the CBR mold described in paragraph 95. After removal of wax paper and paraffin from the ends, the large 7-inch diameter mold is fitted with the base plate and testing collar, as shown on figure 9. This testing collar allows the use of the same adjustable stem, perforated plates, weights and tripod for taking swell measurements as used for the standard CBR molds. If samples taken in metal jackets as shown in figure 10 are to be soaked, the same soaking equipment as just described can also be used, with the exception that no base plate will be required, inasmuch as only one end will be opened and soaking allowed from the top only. After soaking, taking swell measurements, allowing drainage and taking density determination, the samples are considered ready for the penetration test.

100. If this type of sample is not to be soaked, the wax paper and paraffin from one end of the container is removed, the surface made level and the penetration test performed in the usual manner.

Undisturbed
samples in boxes

101. For soaking, a method can be improvised whereby swell readings can be obtained using the same perforated plate, surcharge weights and tripod as used with the standard CBR mold. Since the boxes are usually boiled in paraffin before being sent to the field, the swell in the boxes themselves is negligible.

102. In the event soaking is not required, the wax paper and paraffin can be removed from one end, the surface leveled with a thin cover of sand, if necessary, and the penetration test conducted with the standard testing equipment and in the usual manner.

Preparation of Test Areas for Field In-Place CBR Penetration Test

103. The detailed step-by-step procedure for preparation of the subgrade or base course soil surface in the field prior to penetration is described in Appendix C.

Test Procedure for Soils Containing Gravels

104. The present CBR test procedure has not proven entirely satisfactory for testing samples containing gravel particles. It has been found necessary to conduct a number of tests in order to determine a reasonable average value. This method should be followed until a more satisfactory procedure is developed. In some cases inconsistent test results can be avoided by removing the stones or particles which are not present in sufficient quantity to affect the stability of the soil.

PART VII: COMPARISON OF CBR ON FIELD-COMPACTED AND LABORATORY DYNAMICALLY COMPACTED SOILS AND FIELD COMPACTION EXPERIENCE

General

105. As previously stated, the CBR penetration test was performed by the California Department of Public Works on statically compacted laboratory specimens. However, after adoption of the California method of design, the Engineer Department changed to dynamic compaction. The differences obtained in the physical properties of soils compacted dynamically and statically in the laboratory have been described briefly in Part V and are discussed in detail in Appendix A. There have likewise been introduced since the adoption of the California method by the Engineer Department, field in-place CBR tests, made with the apparatus described in Appendix C. A comparison of CBR values obtained from field in-place penetration tests performed on field-compacted samples and samples dynamically compacted in the laboratory will be presented in this Part, to bring out evidence of mold effect, or differences in field and laboratory compaction. In addition to these comparisons, the effect of traffic compaction on materials sensitive to molding moisture variations will be described. Finally, a brief discussion of field and laboratory compaction experience on soils from several projects will be given.

106. The results of field and laboratory CBR tests made in connection with construction and testing of various field traffic tests are used in making the comparisons presented herein. Comparative CBR test data are included from pavement behavior tests made at Marietta, Georgia and Eglin Field, Florida. Compaction experience from traffic tests on airplane landing mats at Vicksburg, Mississippi and pavement behavior tests at Marietta, Georgia and Langley Field, Virginia are also included. The results of Atterberg limits and compaction tests for soils used in these test sections are tabulated on figure 11. Grain size distribution curves for these soils are shown on figure 12.

Comparison of Field In-Place CBR Tests on Compacted Materials with CBR Tests on Laboratory Remolded Samples

Marietta pavement behavior tests

107. Clay subgrade. Field and laboratory CBR and compaction data on the clay subgrade utilized for the pavement behavior tests at Marietta, Georgia are shown on figure 13. The field density and CBR data were obtained on the material immediately after compaction with a

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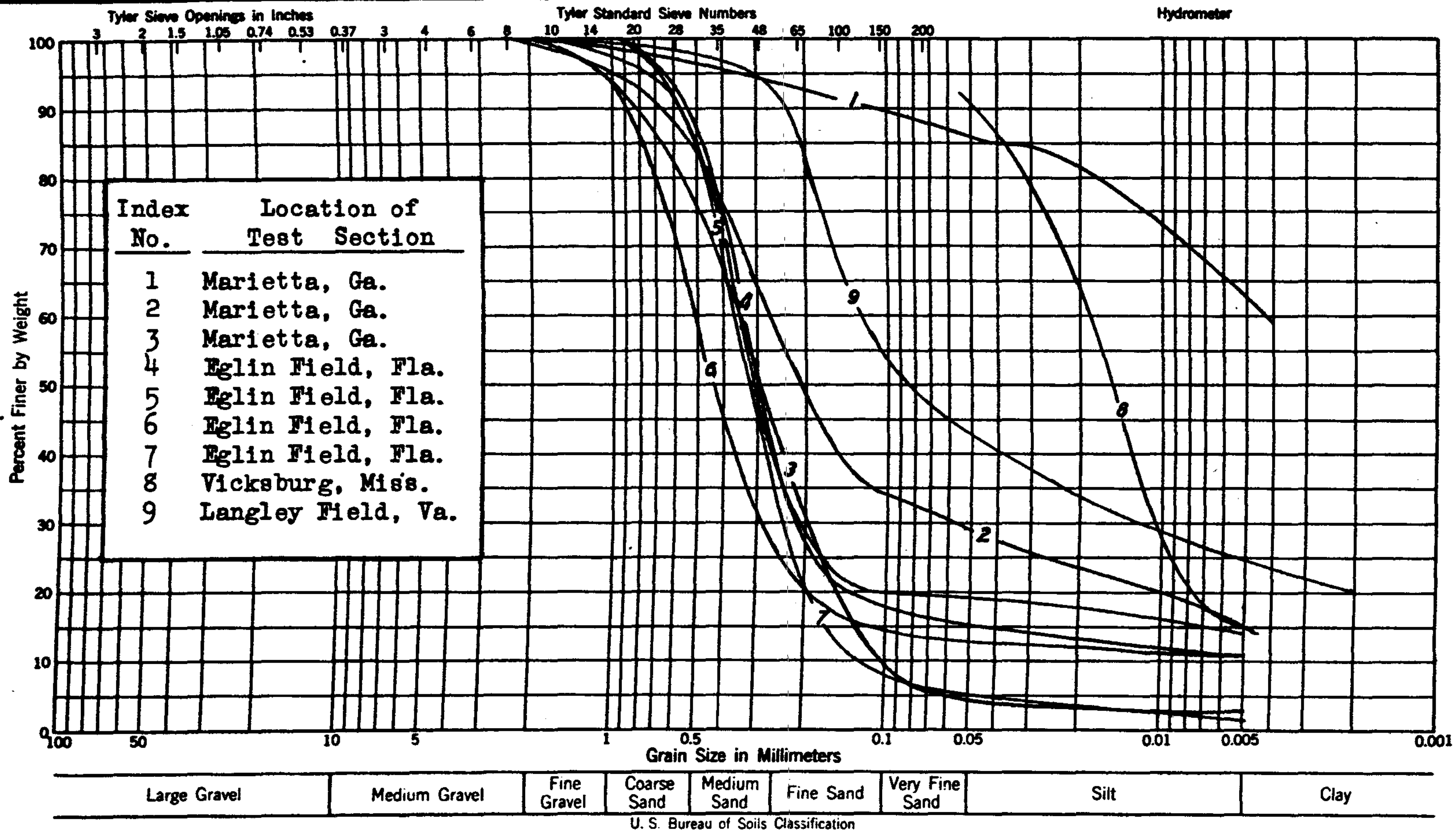
CLASSIFICATION AND COMPACTION DATA ON SOILS FROM VARIOUS TEST SECTIONS

Index No.	Location of Test Section	Material	L.L.	P.I.	Modified AASHO w_o % Dry Wt.	Modified AASHO γ_d Lb/Cu Ft
1	Marietta, Georgia	Clay subgrade	73	46	19.0	104.5
2	Marietta, Georgia	Sand subgrade	Nonplastic		4.5	116.0
3	Marietta, Georgia	Clay-sand base	22	5	9.5	128.5
4	Eglin Field, Florida	Sand-clay base (A)	22	3	9.7	123.7
5	Eglin Field, Florida	Sand-clay base (B)	20	3	8.9	126.2
6	Eglin Field, Florida	Sand-clay base (C)	23	3	10.3	121.7
7	Eglin Field, Florida	Sand subgrade	Nonplastic		9.9	112.6
8	Vicksburg, Mississippi	Clay-silt subgrade	41	15	16.0	106.5
9	Langley Field, Virginia	Silty sand subgrade	22	4	9.5	128.0

FIGURE 11

FIGURE 12



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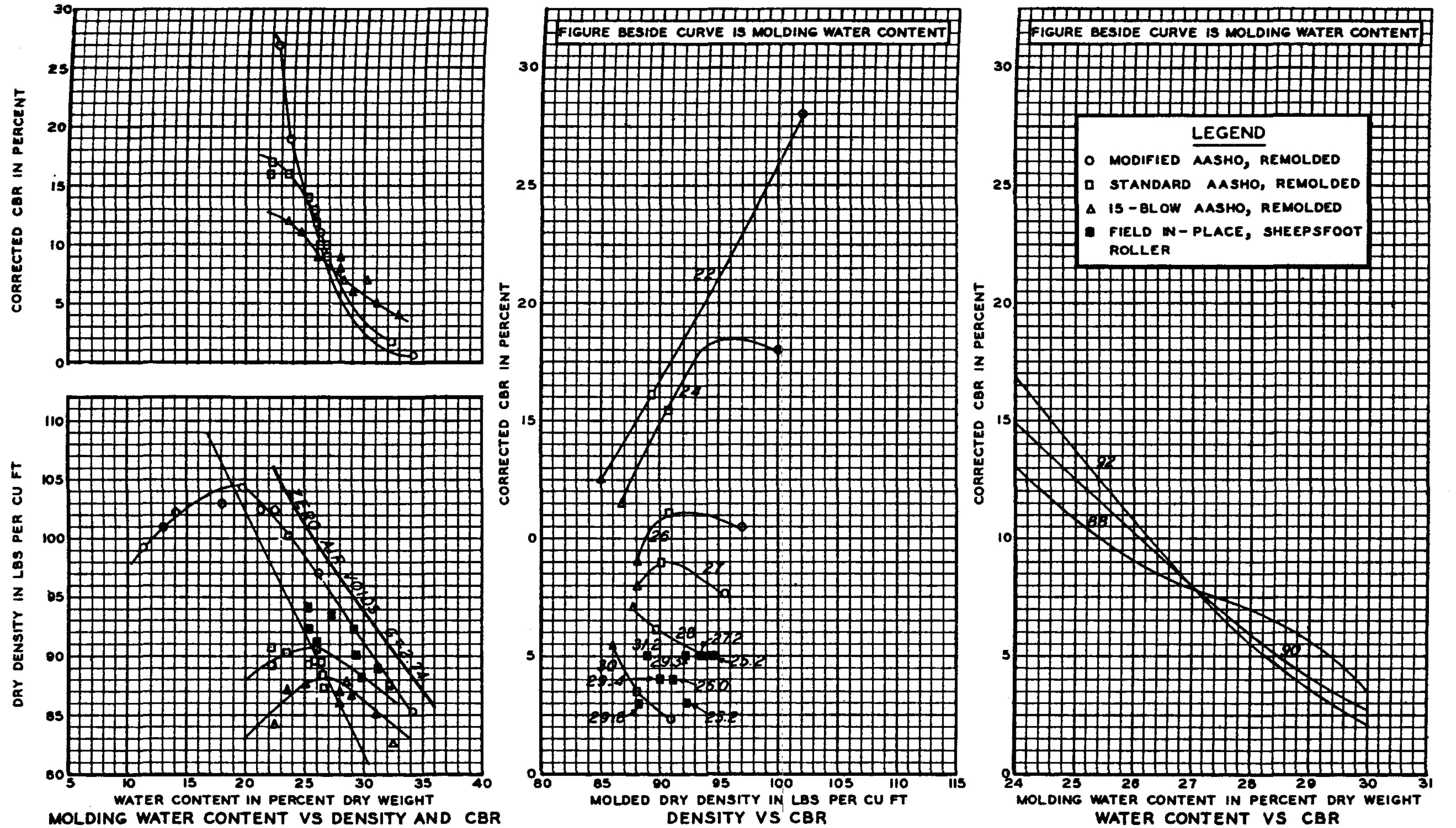
GRAIN SIZE DISTRIBUTION

Soils used in Construction of Various Test Sections



NOTES: TESTS PERFORMED IN CONNECTION WITH PAVEMENT BEHAVIOR TESTS AT MARIETTA, GEORGIA.

FIELD CBR TESTS PERFORMED BEFORE TRACKING.

ALL TESTS ON UNSOAKED SOIL.

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VARIATION OF CBR WITH DENSITY AND WATER CONTENT FOR LABORATORY AND FIELD COMPACTION

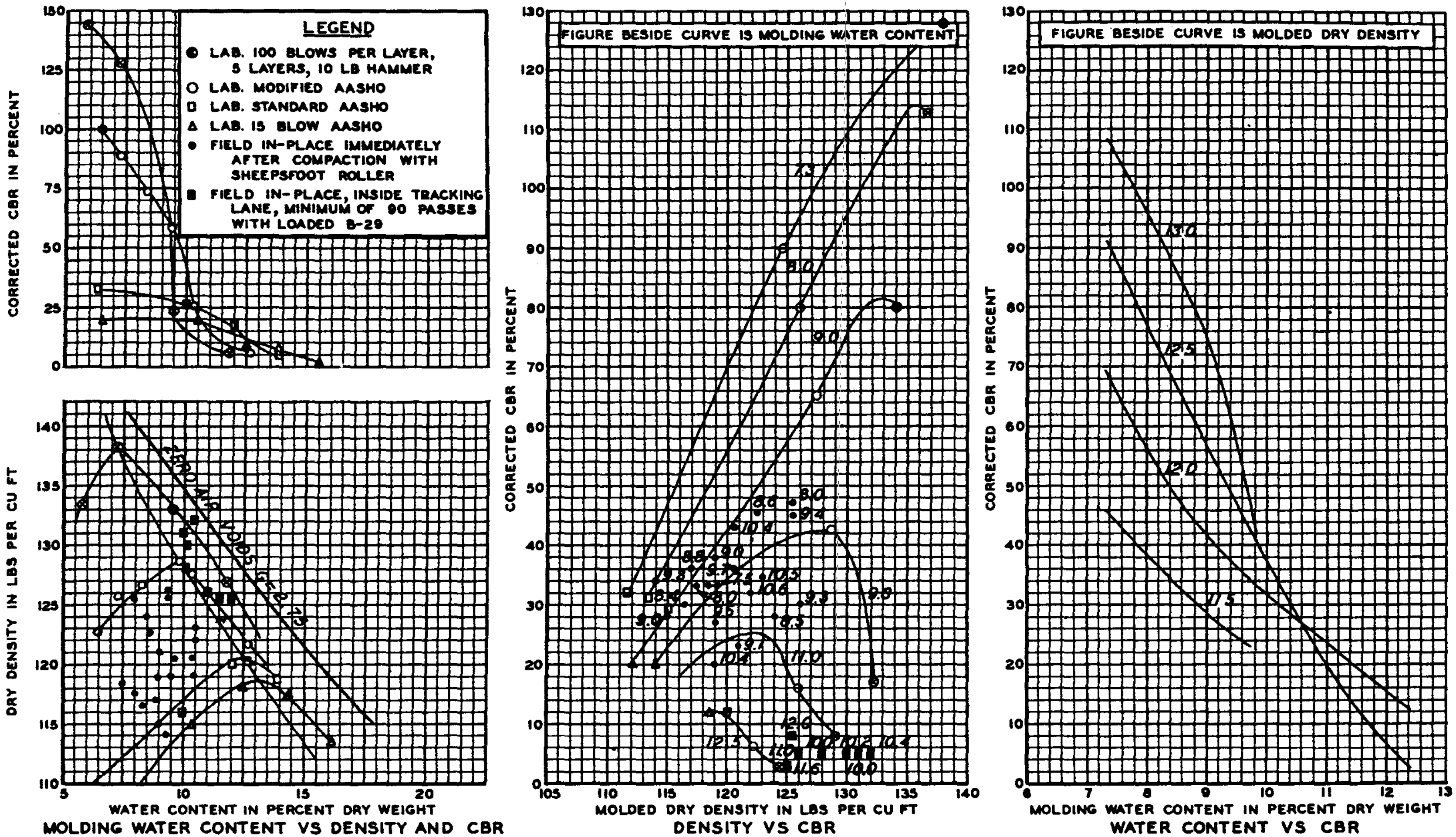
MARIETTA CLAY SUBGRADE

FIGURE 13

FIGURE 14

WAR DEPARTMENT

CORPS OF ENGINEERS. U.S. ARMY



NOTE: ALL LABORATORY SPECIMENS TESTED AS MOLDED

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VARIATION OF CBR WITH DENSITY AND WATER CONTENT FOR LABORATORY AND FIELD COMPACTION

MARIETTA, GA. MIX — CLAY - SAND

sheepsfoot roller. The laboratory compaction and CBR data as shown are for 15-blow, standard, and modified AASHO efforts on unsoaked material. It can be seen that the compactive effort in the field using a sheepsfoot roller is between standard and modified AASHO efforts. It can also be seen that the majority of the specimens representing the field-compacted material are on the wet side of the standard AASHO optimum water content values. All are on the wet side of modified AASHO optimum water content. These data also show that in four instances the CBR values from the laboratory remolded tests are greater than, and in two instances are equal to, the field in-place values on this soil in the condition tested. In two instances the laboratory value was less than the field in-place value.

108. Sand subgrade. This material was compacted with a sheepsfoot roller in the field as subgrade for one portion of the flexible pavement. Field and laboratory compaction data for this sand subgrade showed that the average density under modified AASHO effort was 116 pounds per cubic foot and that the field compaction gave the same value. The average CBR, however, for the laboratory was 59 and for the field only 27. The laboratory remolded samples were at a lower water content, with the exception of one specimen, than the field samples. However, it has been shown in this investigation that cohesionless sand is ~~not sensitive to~~ molding water content. It is therefore believed that two factors largely account for the difference between laboratory and field CBR data. These are mold effect and density gradient. Laboratory data on cohesionless soils have shown the bottom of the specimen as compacted dynamically (the bottom is the end penetrated in the CBR test) to be two to three pounds per cubic foot denser than the average computed density. CBR tests on this material show that a two to three pound increase in density between standard and modified AASHO compactive efforts results in the actual CBR value increasing by as much as 20 to 30. It therefore appears that the laboratory remolded samples at approximately modified AASHO density, being tightly compacted in the mold, are affected considerably by the mold.

109. Clay-sand base course. Figure 14 shows field and laboratory compaction and CBR data on a clay-sand used beneath the pierced plank mat portion of the test section as a base course material. The field in-place tests were performed immediately after the clay-sand had been compacted with a sheepsfoot roller and after tracking with a loaded aeroplane. It can be noted that in nine out of nineteen instances for sheepsfoot roller compaction the laboratory remolded unsoaked CBR values are higher than the field in-place unsoaked values. In six instances the laboratory values are equal to the field values and in the remaining three instances the laboratory values are less than the field. It can also be seen that on all field specimens tested after compaction by the roller, the water content of the material as placed in the field was on the dry side of the standard AASHO optimum value and the majority of the field test specimens had water contents on the dry side of modified AASHO optimum water content. Figure 14 also shows field compaction and CBR data for the clay-sand after tracking. In this instance the laboratory CBR values are all greater than the field in-place values.

Eglin Field
test section

110. Sand subgrade and base course materials. The tests at Eglin Field were conducted by the Mobile District. The following tabulation is taken from the report on this test section. The values given are for tests in the unsoaked condition. The subgrade of the test section was graded to the desired elevation but was not compacted other than by incidental construction traffic prior to placing the base course. The base materials were compacted with sheepsfoot and rubber-tired rollers to densities necessary to give the required CBR values. In general the water content was slightly dry of optimum during compaction.

Material	Location	Field Moisture Percent*	Field Dry Density Lb/Cu Ft*	In-Place	Laboratory (R)**
Base A	Inside	7.7	117.4	67	64-95
Base B	Inside	6.4	120.8	86	36-84
Base C	Inside	5.8	118.4	61	68-78
Base A	Outside	7.2	118.2	52	68-104
Base B	Outside	6.4	119.1	46	28-73
Base C	Outside	6.1	115.9	37	48-60
Subgrade	Inside	5.3	110.2	35	33
Subgrade	Outside	4.7	109.3	31	31

* At end of traffic test
** (R) = CBR of recompacted samples at in-place density. First value is at optimum moisture, second value is at field moisture

The tabulation above shows that there is good agreement between in-place and laboratory remolded test values for the subgrade. However, there is considerable variation in the results of the tests on the base materials. These variations are believed to be due mainly to the effect of molding moisture content, which resulted in differences in the physical properties of the soil mass. It is considered that the value of the field test should fall within the range of the laboratory values shown, in which case there appears reasonably good agreement between the two values.

Comparison of Field and Laboratory Compaction Experience

General

111. Although the data contained herein are rather limited, it is believed that a few worth-while observations may be made concerning a comparison of field and laboratory compaction. It should be borne in mind that when the subgrade and/or base after construction shows further compaction under the traffic to which it is subjected, then this

additional compaction may, in a sense, be considered as a continuation of the construction compaction.

112. It has been found from laboratory studies that all except free-draining cohesionless soils and clays which exhibit high swell show a decrease in CBR with increase in density for a constant water content when compacted dynamically on the wet side of optimum in the laboratory. This was very pronounced in samples subjected to laboratory dynamic compaction, but there was only an occasional slight tendency for this to take place in samples subjected to laboratory static compaction. Three instances are cited below where field compaction produced the same phenomenon as stated above for laboratory dynamic compaction. The soils in which this occurred were: the clay subgrade and the clay-sand base at Marietta, Georgia, and the silt subgrade of the Vicksburg landing mat tests. A special soil condition experienced at Langley Field is also described.

Marietta pavement
behavior test

113. Clay subgrade. The compaction of the clay subgrade at Marietta was accomplished with a sheepsfoot roller and the compaction and CBR data shown on figure 13 were obtained immediately after compaction with the sheepsfoot roller. It can be seen on this figure that the majority of the specimens representing field-compacted material are on the wet side of the standard AASHO optimum value and, even though the density is greater than that for the laboratory standard and 15-blow AASHO compaction, most of the CBR values are less.

114. Clay-sand base course. As shown on figure 14, the clay-sand base material at Marietta was placed initially on the dry side of the laboratory optimum between standard and modified AASHO effort. However, this figure also shows that under traffic compaction a large part of the material was compacted sufficiently to place it on the wet side of the laboratory optimum corresponding to the effort exerted by traffic compaction, which is higher than modified AASHO. It can be seen that, even though the density was increased under traffic to a value greater than that initially obtained with the sheepsfoot roller, the CBR showed a considerable decrease -- the in-place values after traffic being less than either the laboratory modified AASHO values or the in-place values before traffic. On most of the tests the material is at a density approximately equal to that obtained in the laboratory using dynamic compaction consisting of 100 blows per layer on 5 layers with a 10-pound hammer dropped 18 inches in the CBR mold. It can be seen, however, that the CBR values from the field are all lower than those obtained under 100 blows in the laboratory.

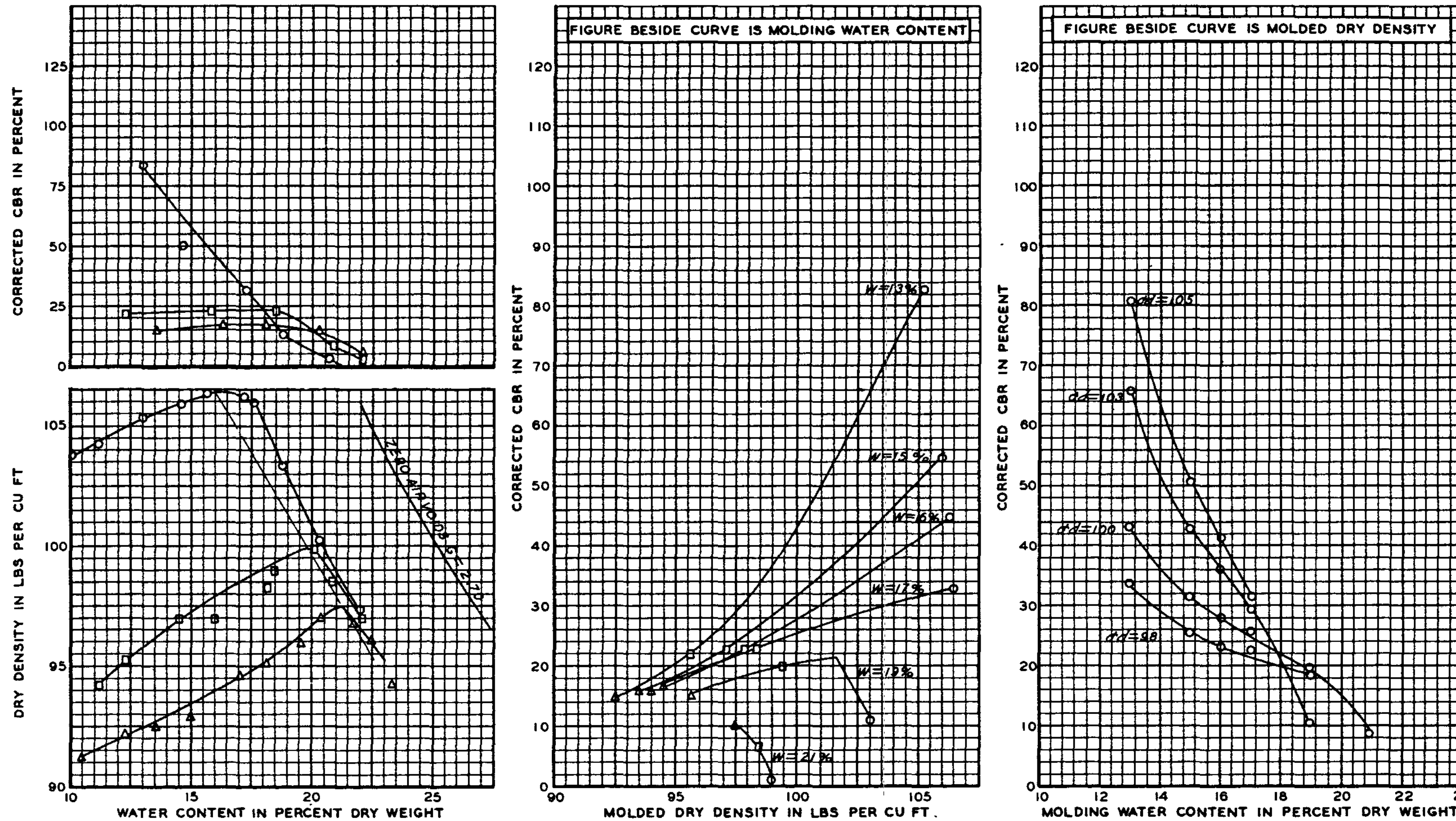
Vicksburg landing
mat traffic tests

115. Figure 15 shows the variation of CBR with density and water content for the silt subgrade material used in connection with some landing mat traffic tests at Vicksburg, Mississippi. The data on this figure show that the CBR of this silt material is sensitive to both density and compacting water content. Examination of this figure will show that for a constant molding water content the CBR of this material may decrease with additional compaction on the wet side of the optimum values. This is attributed to the development of pore-water pressure during compaction, or to the establishment of a plastic soil system in the specimen being compacted. This phenomenon was also exhibited under traffic by the silt subgrade of the test section. It was found that a section without mat or base course, with a CBR of 8 percent and a water content of 20 percent, failed at 172 coverages. Normally this section would have consolidated to a CBR of 20 percent and withstood 1000 coverages. However, its water content was great enough that, after a certain amount of traffic compaction, the compactive effort for this water content was passed. Then pore-water pressure developed and failure occurred.

Langley Field
traffic tests

116. Data from the Langley Field test section indicate that if the excess water contained in the soil can be forced out at a reasonable rate under traffic compaction then the material may pass on beyond the stage of reduction in strength and finally reach a stable condition, provided the traffic is continued long enough and the surfacing can stand up under the large deformations that take place before stability is reached. The subgrade soil at the Langley Field test section was composed of a previously constructed hydraulic fill. The deposit, to a depth of 4 feet, consisted of a heterogeneous combination of grayish-brown to brown fine cohesionless silt and sand containing small balls of clay and very thin lenses of fine clean sand. When remolded it is classified as a slightly plastic silty sand. The subgrade had a natural average dry density of 112.5 pounds per cubic foot with an average field water content of 16.8 percent. Remolded, it had a standard AASHO dry density of 117.5 pounds per cubic foot at an optimum water content of 13.5 percent and a modified AASHO dry density of 128 pounds per cubic foot at an optimum water content of 9.5 percent. The subgrade was not compacted during construction (hydraulic fill) but was compacted by traffic below a crushed stone base and asphalt-concrete pavement of total thickness of about 24 inches. The material was very close to saturation at the beginning of the traffic tests. The following information is taken from the report "Accelerated Traffic Test, Langley Field, Va.," prepared by the Norfolk District: At Langley Field, testing with a 20,000-pound wheel load was continued until 6,667 coverages had been completed. Upon completion of the test no failures of the pavement due to shear deformation of the subgrade had occurred. Early in the traffic test the subgrade soil began to compact

SPECIMENS TESTED AS MOLDED



MOLDING WATER CONTENT VS DENSITY AND CBR

DENSITY VS CBR

WATER CONTENT VS CBR

LEGEND

O MODIFIED AASHO	CLAY SILT
□ STANDARD AASHO	LL = 41
△ 15 BLOW AASHO	P.I. = 15

NOTE: PENETRATION SURCHARGE 0 LB.
DATA OBTAINED FROM LANDING MAT REPORT.

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VARIATION OF C B R WITH DENSITY AND WATER CONTENT

SILT SUBGRADE OF LANDING MAT INVESTIGATION

FIGURE 15

and excess hydrostatic pressure began to develop. This was visible in the form of excessive springing, which increased until it reached a maximum of 2 inches and then began decreasing until at 2,667 coverages it had practically ceased. No density or CBR tests were taken prior to the traffic tests. However, at the completion of the traffic, tests on the top 8 inches of the subgrade outside the traffic lanes indicated an original CBR value of 15 and a dry density equal to approximately 88 percent of modified AASHO density. In the unfailed areas at the completion of the traffic tests the CBR increased to approximately 30 in the 20,000-pound traffic lane and the dry density was increased to 94 percent of modified AASHO.

PART VIII: CONCLUSIONS

117. The following principal conclusions appear warranted as a result of information and data gathered from this investigation:

<u>a</u>. The CBR test is considered to be only the actual penetration of the soil and the procedure described in Part VI is considered satisfactory.

<u>b</u>. For design or evaluation purposes it is necessary to conduct CBR tests on: (1) natural undisturbed samples with water content adjusted to expected field conditions, and/or (2) on remolded samples which have the same <u>molding water content</u>, <u>moisture conditions</u>, <u>density and physical properties</u> that will be produced during, or after, construction. <u>Therefore, the CBR test can not be considered as a classification test, but is a shear test</u>, and the CBR values obtained from the test are moduli of shearing resistance, the validity of which are dependent on preparation of the test specimen to duplicate field conditions. The test is only considered valid when a large portion of the deformation under penetration is shear deformation.

<u>c</u>. Wide variations in CBR design test results on remolded samples are largely due to the method of preparation of the test specimens. The variations, however, are <u>systematic</u> and are caused primarily by the effects of <u>molding water content</u>, <u>density</u> and <u>type of compaction</u> used in preparing test specimens. The variations are probably qualitatively valid but may not be strictly quantitatively valid. Consistent laboratory results can be obtained only when the above variables are given full consideration. Satisfactory methods have been developed for preparation of test specimens which take these factors into account.

<u>d</u>. Small changes in density greatly affect the CBR, especially at high densities in the order of magnitude of modified AASHO. Small changes in molding water content <u>greatly affect</u> the CBR of <u>unsoaked</u> laboratory specimens, except on clean sands and gravels. At a constant density in the unsoaked condition, the higher the molding water content the lower the CBR. This general trend is normally expected for soils in an as-molded or unsoaked condition which for a constant density are known to decrease in shearing resistance as the degree of saturation increases.

e. For soaked laboratory specimens, except cohesionless sands, compacted by the static method, the CBR increases with increase in the molding water content at a constant density. This is a reversal of the behavior of unsoaked specimens and is caused by the fact that the drier the molding condition the greater the swell during soaking. The same trend occurred for soaked specimens of impervious high-swelling soils compacted by the dynamic method.

f. For soaked specimens of low plastic soils with little or no swell, compacted by the dynamic method, the CBR decreases appreciably with a slight increase in the molding water content at a constant density. This is especially significant, since it is generally believed by most engineers that the shearing resistance is not sensitive to the molding moisture within normal laboratory control for soaked specimens compacted to a given density.

g. The results of unconfined compression and triaxial shear tests indicate that, when the shearing resistance is chosen at low strains, similar trends were found to occur as just described for the CBR test.

h. Inasmuch as the molding water content is such a prime factor in controlling the physical properties of all except free-draining soils, it follows that, in remolded soils, duplicate laboratory specimens can not be prepared, unless the same molding water content and method of compaction are duplicated, even though water contents and densities obtained subsequent to molding are duplicated. In other words, if a soil is molded at some given water content and then this water content is allowed to increase or decrease by a given amount, another identical soil specimen can not be reproduced, unless the whole cycle is reproduced, starting with the same molding water content.

i. Wherever applicable, field in-place tests or tests on undisturbed samples should be performed.

j. Remolded CBR test specimens prepared for design study purposes should be compacted in the 6-inch diameter CBR mold, using compaction performed by the impact of a free-falling hammer weight, such as the AASHO method. Procedures given in Part VI are considered best.

k. All control compaction for CBR tests should be performed in the 6-inch diameter CBR mold. Material should not be re-used and the mold should be placed on a concrete floor or pedestal for firm support during compaction.

l. The most practical method for soaking specimens for CBR tests is by submergence (top and bottom) for four days with the surcharge applied as specified in Part VI. Less time is required for cohesionless soils.

m. Test results on samples containing stones are erratic and modification of the test procedure is needed for these soils. Until a more suitable procedure is developed, several tests should be performed, in order to obtain average representative results.

n. Either a closely-controlled constant strain type of loading machine or a combination screw jack and proving ring arrangement should be used for the penetration test, in order to eliminate the surge effect created by the single-acting hydraulic jack originally used. A satisfactory field in-place CBR apparatus has been developed.

o. The extreme increase in CBR values for some soils above standard AASHO density may be due partly to the confining effect of the 6-inch diameter mold. Present indications are that the 6-inch diameter mold is not large enough at high densities (modified AASHO) on soils exhibiting less than 3 percent swell during soaking, nor for base materials containing 3/4-inch particles.

PART IX: RECOMMENDATIONS FOR FURTHER INVESTIGATIONS

118. Inasmuch as the laboratory studies have indicated a marked difference in the physical properties of a given soil prepared by different compaction methods in the laboratory, it can be assumed that a similar difference in physical properties occurs when the soil is compacted by different methods in the field. Likewise it is entirely possible that the physical properties obtained with the usual types of field compaction equipment may be entirely different from those which are obtained in the laboratory. It is therefore recommended that field test embankments be constructed using various types of field compaction equipment. Molding water content and weight and speed of field equipment should be more closely controlled than is usually done in construction work, so as to evaluate the effect of molding water content and compaction method on the physical properties of the compacted soil. As a result of these tests, laboratory compaction methods should be devised which will duplicate field compaction obtained by presently available field methods. Such test embankments are now being constructed by the Engineer Department.

119. Other studies which are recommended to obtain greater perfection in preparation of CBR test specimens are:

- a. Further studies of the effect of gravel on the CBR value.
- b. Further studies of the confining effect of the CBR mold on the values obtained from the penetration test.
- c. Development of a laboratory mechanical compactor which will eliminate inconsistencies in laboratory compaction tests.
- d. Development of compaction apparatus which will yield a uniform density throughout the specimen.

APPENDICES

APPENDICES

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APPENDIX A

LABORATORY STUDIES ON THE CALIFORNIA BEARING RATIO TEST

APPENDIX A

LABORATORY STUDIES ON THE CALIFORNIA BEARING RATIO TEST

PART I: INTRODUCTION

Authorization

1. This study was requested by the Office, Chief of Engineers in the second indorsement dated 17 September 1942 to a letter from the Experiment Station dated 7 September 1942, subject "California bearing ratio test procedure." Authority to perform the work was granted by the Office, Chief of Engineers in the sixth indorsement dated 13 Novem-13 November 1942 to the same letter.

Purpose of Laboratory Studies

2. For design purposes it is necessary to prepare remolded laboratory samples for the CBR test which will duplicate conditions to be expected in the field. The purpose of these studies was to investigate the CBR test in the laboratory with a view toward obtaining data to support a general investigation made for the following purposes:

- a. To determine a suitable method of preparing remolded samples for the CBR test.
- b. To determine necessary modifications to the CBR (penetration) test as originated by the California Highway Department.
- c. To develop a loading apparatus for making CBR field in-place tests and laboratory tests which would eliminate the surge effect of the original hydraulic jack arrangement used for penetration of the sample.

3. The objectives of the study were accomplished by conducting a comprehensive laboratory investigation on numerous types of soils. The soils chosen for the study and the schedule of tests followed were outlined in October 1942 and at conferences held in November and December 1942. The schedule of tests was expanded at conferences held in February and April 1943. The development of a field CBR apparatus (see Appendix C) was accomplished by incorporating various improvements to a combination screw jack and proving ring arrangement, the basic idea for which was developed by the Soils and Pavement Section of

the Little Rock District. This apparatus can also be used in the laboratory by inserting it in a frame, as described in Part V of the report.

4. A complete list of soils originally considered for the study and a schedule of variables investigated is shown in table 1. This table together with plates 1 through 23 shows classification and compaction data for the majority of the soils. Eight bracketing soils finally chosen for complete investigation are indicated by asterisks in table 1. To determine a suitable method for preparing remolded samples for the CBR test, the following factors were studied:

a. Method of compaction	g. Soaking surcharge
b. Density	h. Drainage time
c. Water content	i. Processing base course soils
d. Height of specimen	j. Density gradient
e. Method of soaking	k. Water content gradient
f. Period of soaking	l. Crushing of particles

5. The principal factors studied regarding the CBR penetration test were:

a. Penetration surcharge

b. Rate of penetration

c. Correction of stress-strain curves

d. Mold and piston diameter

6. Penetration tests were performed on remolded samples to determine the effect on the CBR of each of the variables listed in paragraphs 4 and 5, with the exception of water content gradient and crushing of particles. Triaxial and unconfined compression tests were conducted on some of the soils to determine if the same trends in shearing resistance occurred as indicated by the CBR tests. A few consolidation tests were conducted to obtain data regarding the effect of molding water content.

7. During the course of the investigation it was decided to determine the effect of the variation in the gradation of cohesionless soils on the CBR and in addition it was decided to correlate, if possible, the modulus of soil reaction "k" as determined from field plate bearing tests on several soils with the CBR. In connection with the variation of gradation of cohesionless soils versus CBR, the Office, Chief of Engineers in January 1943 requested each Division to prepare reports on the correlation of CBR with grain size characteristics of sands and gravels, if such data were available. This correlation was desired in order to determine if classification methods would be satisfactory for determining the CBR of cohesionless soils. Copies of all reports prepared were forwarded to the Office, Chief of Engineers for review, after which they were furnished the Experiment Station for further study and

to determine whether a correlation were possible. No satisfactory correlation has been found to date. Additional data which were found lacking were requested of certain Divisions supplying reports. It is planned to continue this comparative study as the additional requested information is received and as higher priority work permits.

8. In connection with the study of "k" versus CBR, the original program of tests outlined provided for the compaction of several types of soils in a 9 ft by 9 ft test pit. A series of plate bearing and CBR tests were then to be performed on the compacted materials in the as-placed and saturated conditions and the results compared. Due to the difficulty experienced in obtaining satisfactory compaction and the fact that a fully saturated condition could not be obtained in a reasonable length of time, it was decided to abandon this procedure in favor of tests on undisturbed materials. Consequently, the testing program was altered to provide for tests on a silt, lean clay, and heavy clay at the natural field water content condition and after saturation. A cooperative program was established with the Committee on Sampling and Testing, Soil Mechanics and Foundation Division of the American Society of Civil Engineers, in which the Experiment Station was to perform all plate bearing tests and secure undisturbed samples for laboratory tests. All laboratory work, with the exception of the CBR tests, was to be performed by the Committee on Sampling and Testing. These included consolidation and unconfined compression tests. A summary of the work performed by the Committee, which includes comparison of field plate bearing tests, unconfined compression and triaxial compression tests on the three soils, has been released by them in the form of Technical Progress Reports 1, 2 and 3, dated 8 November 1943, 15 January 1944 and 30 March 1944, respectively. These three reports have been incorporated as an appendix to a report prepared by the Experiment Station on the study entitled "Rigid Plate Bearing Test Investigation," dated 1 March 1945. Although these tests showed the effect of water content and plate area on the modulus of soil reaction "k", they were not conclusive in so far as a correlation between CBR and "k" was concerned because of their limited number. A copy of this report can be obtained on a 30-day loan basis from the Engineer Department Research Centers Library located at the Experiment Station.

9. In addition to the original soils listed for study, several other materials studied in connection with the construction of the Pavement Behavior Test Section at Marietta, Georgia and the Airplane Landing Mat Test Sections at Vicksburg, Mississippi were investigated and the results incorporated herein. These soils are listed at the bottom of table 1 together with the results of classification and compaction test data and a schedule of variables investigated.

PART II: STUDY OF THE CBR PENETRATION TEST

General

10. In order to evaluate all factors affecting the CBR value obtained, it was necessary to know whether certain variables that entered into the test procedure subsequent to the preparation of the test specimen created any influence on the magnitude of the CBR. In addition, a few tests were performed in order to study the effect of sand particles on the CBR. The soils used for these studies are described in Part III.

Description of CBR test

11. The actual CBR penetration test used in this study was accomplished by placing the required penetration surcharge weights on top of the specimen, seating the penetration piston under a 10-pound load, adjusting the zeros of the recording dials on the loading equipment and the dial indicator used for measuring penetration depths. The loading equipment used was a Baldwin-Southwark hydraulic testing machine of 66,000-pound capacity. Except for special tests, a constant rate of penetration of 0.05 inch per minute was maintained during the actual penetration of the specimen, load readings being recorded usually at penetration depths of 0.025, 0.05, 0.075, 0.1, 0.15, 0.2, 0.3, 0.4 and 0.5 inch.

Penetration surcharge

12. A penetration surcharge is used to duplicate the confining effects of the pavement or base and pavement. All test specimens used to evaluate this surcharge loading were compacted dynamically (see "Definitions" in Part I of main report and description of dynamic compactor in paragraph 26 in this appendix), soaked, and then penetrated under various penetration surcharge loads. The results of tests on cohesionless soils are shown on plates 24 to 29, inclusive; on soils of low plasticity on plates 30 to 33, inclusive; and on soils of medium to high plasticity on plates 34 to 36, inclusive. Reference to applicable plates shows that an increase in the penetration surcharge results in a marked increase in the CBR for cohesionless soils, a moderate increase for soils of low plasticity, and practically no change for soils of medium and high plasticity.

Rate of penetration

13. It was not known whether a fairly rapid rate of penetration would produce a "quick" condition in saturated cohesionless materials, thus effecting a reduction in strength, or would cause an increase in

strength of saturated cohesive soils because of the occurrence of viscous resistance. Likewise it was not known whether a comparatively slow rate of strain or penetration would allow consolidation to occur and thus effect an increase in strength over that normally expected. To evaluate these factors, test specimens of a cohesionless sand and a soil of low plasticity were compacted dynamically, soaked, and then penetrated under different rates of strain ranging between 0.025 and 0.1 inch per minute. In addition, test specimens of a soil of high plasticity were compacted dynamically and then penetrated as molded under rates of strain between 0.025 and 1 inch per minute. The results of these tests are shown on plates 37 and 38.

14. Variation of the rate of penetration between 0.025 and 0.1 inch per minute had little or no effect on the CBR for the cohesionless soil shown on plate 37 for the particular conditions of the test, that is, molded at modified AASHO optimum and soaked. The CBR increases appreciably with increase in rate of penetration on the low plastic soil for the same range of penetration rates as above and when molded at modified AASHO optimum and soaked before testing (see Vicksburg loess on plate 37). There is no change in the CBR with increase of rate of penetration from 0.025 to 1.0 inch per minute for specimens of adobe clay tested as molded at 95 percent of modified AASHO maximum density on the wet side (see plate 38). The specimens of adobe clay were tested as molded on the wet side to insure a uniform distribution of water and to eliminate the soaking period. It is believed that the use of the original rate of 0.05 inch per minute will give satisfactory results for all soils.

Correction to stress-
penetration curves

15. Tests performed in the early part of these studies and tests conducted by other laboratories showed that for some soaked samples the stress-penetration curves were concave-upward near the origin. Typical CBR stress-penetration curves for all bracketing soils plus several others are shown on plates 39 to 52, inclusive. To correct the concave-upward portion of the curve, the portion of the curve with maximum slope over 0.1 inch penetration was extended to zero stress and a new origin established. For the purpose of this investigation, it was assumed that the controlling CBR value was at 0.1-inch penetration, which allowed the use of the following method of correction.

16. When required, the correction as shown on the above-mentioned plates, is made by drawing a line through the origin parallel to the steepest slope maintained for any 0.1-inch of penetration, and selecting the bearing value at the intersection of this line with the line at 0.1 inch penetration. Although this correction can be made graphically, it is usually made mathematically by taking the greatest accumulative load change that occurs in any 0.1 inch of penetration as the bearing value at the first 0.1 inch. All corrections in this investigation were accomplished mathematically.

17. In this investigation the penetration curves for soils of high plasticity never required a correction, and those for soils of medium plasticity seldom required a correction (see penetration curves on plates 46 and 47). Clean sands, when properly compacted and surcharged, seldom required correction (see plate 45). The soils in which the stress-penetration curves practically always develop a concave-upward shape in contrast to the standard curve shape (consistently concave-downward) are the intermediate ones between clean sands and soils of high plasticity (see plates 39 through 43 and 48 through 52). However, these intermediate soils never require a correction when molded on the dry side of optimum for any given dynamic compactive effort, provided the surface of the specimen is not disturbed in some manner, such as swell in the upper portion. Statically compacted specimens of any and all the soils tested seldom required correction. It is on the wet side of optimum under dynamic compaction that this concave-upward shape develops. This is due to the lack of rigidity of the soil mass or the plastic type of structure or soil system obtained on the wet side of optimum. As will be shown later, similar stress-strain curves were ~~obtained in triaxial and unconfined compression~~ test specimens compacted dynamically. In view of the above, the correction of the curves is possibly not warranted. However, for consistency and more uniform results in this research investigation, all penetration curves that did not show the greatest accumulative load change to occur in the first 0.1 inch penetration were corrected. Until more data can be obtained it is recommended that the curves be corrected.

Shape of cohesionless particles

18. Very limited data on the effect of this variable were obtained. A subrounded and a subangular sand were compacted dynamically, soaked and penetrated. The results of these tests are shown on plates 25 and 29, which show that there is no practical difference in the CBR for these two sands.

Mold and piston diameter

19. The standard CBR mold is 6 inches in diameter and the CBR penetration piston is 1.95 inches in diameter. In order to study the confining effect of the mold and the effect of the relation between the diameter of the mold and the diameter of the piston, a series of tests on a cohesionless soil, a silt of low plasticity and a soil of high plasticity was conducted using the standard CBR mold and piston and using a 12-inch diameter mold and 4-inch diameter piston. In addition, the effect of mold and piston diameter on the CBR of three natural and processed base course soils was studied.

20. It appears that for fine-grained soils the 6-inch diameter mold (using a 2-inch diameter piston) may possibly offer enough

confinement to prevent the value obtained on remolded laboratory specimens from checking field in-place values, particularly in materials having high CBR values (see pictures of specially prepared specimens, plates 53 through 58; note especially plate 55). Data for the cohesionless, low plastic and very cohesive soils, using 2- and 4-inch diameter penetration pistons in molds of 6- and 12-inch diameters, respectively, are shown on plates 59, 60 and 61. On the sand and loess, the concave-upward shape of the penetration curves are more pronounced with the 4-inch piston in the 12-inch mold than with the 2-inch piston in the 6-inch mold. On adobe clay, both curves are concave-downward and fall practically one on top of the other. For the sand, the corrected CBR value with the 2-inch piston in the 6-inch mold is 113 percent greater than the corrected CBR value with the 4-inch piston in the 12-inch mold; for the loess this relationship is 64 percent, and for the adobe clay there was no difference.

21. Data showing the effect of mold and piston diameters on the natural and processed base course soils are given in table 2. The following results are pertinent:

<u>a</u>. CBR values of natural material in the 6-inch mold (2-inch piston) are from 50 to 200 percent greater than CBR values of natural materials in the 12-inch mold (4-inch piston).

<u>b</u>. CBR values of processed materials in the 6-inch mold (2-inch piston) are from 165 to 275 percent greater than CBR values of processed materials in the 12-inch mold (4-inch piston).

<u>c</u>. CBR values of natural materials in the 12-inch mold are from 25 to 300 percent greater when using a 4-inch diameter penetration piston than when using a 2-inch diameter piston.

<u>d</u>. CBR values of natural materials using a 2-inch diameter penetration piston are from 285 to 500 percent greater in the 6-inch mold than in the 12-inch mold.

22. It is recognized that CBR values taken at 0.1-inch penetration using a 4-inch diameter piston should not be compared directly with those obtained using the standard 2-inch diameter piston without due consideration for the difference in stress distribution, particularly the depth to which the stresses are effective. This should be kept in mind in studying the data shown for these tests. If the results are plotted in terms of unit load against the ratio of deformation in inches to the diameter of the piston in inches, then the stress deformation curves are generally closer together than when the deformation is expressed in inches. This assumes the major portion of the stresses to be dissipated within depths proportional to the diameters.

Equipment

23. The principal items of equipment used for making the penetration test on remolded samples consists of the 6-inch diameter mold containing the sample, a steel penetration piston 1.95 inches in diameter (area 3 sq in.), penetration surcharge weights, and a laboratory loading machine or screw jack and frame arrangement. The mold, piston and weights are shown on figure 5 and the screw-jack loading arrangement is shown on figure 4 of the main report. The original loading arrangement specified in Chapter XX of the Engineering Manual was a single-acting hydraulic jack which has not proven satisfactory because of the surge effect created during application of load. Either a closely controlled laboratory constant strain type of machine or the arrangement shown in figure 4 for laboratory or field should be used for applying load increments, in order to eliminate this surge effect. Appendix C describes and illustrates this screw-jack type of loading equipment used in the field and which can be easily duplicated in the laboratory by shifting the jack to the frame shown in figure 4 of the main report. This type of arrangement is necessary in order to insure a smooth rate of penetration.

PART III: STUDY OF THE PREPARATION OF REMOLDED SAMPLES FOR THE CBR TEST

General

24. This part of the appendix first presents data of a preliminary nature which includes results of classification tests on the materials investigated. It then gives data on compaction characteristics produced by dynamic and static compaction and data on certain variables that entered into the preparation of remolded specimens for CBR tests. Next, the effects of these same variables on the results of tests other than CBR are discussed and finally the effects of a few other variables that entered into the preparation of remolded specimens for CBR tests is given, some of which could be evaluated only by the actual performance of the CBR penetration test.

Preliminary Tests

Classification

25. As previously stated, the results of mechanical analyses, specific gravity and Atterberg limits tests performed on all soils studied are shown on plates 1 through 4 and table 1. It is believed that the data shown thereon give all the information necessary in order to obtain a clear picture of the characteristics of the soils, with the exception of Vicksburg loess, Texas caliche and California pumice. These materials may be considered special soils and hence a brief description of each is given below. The Vicksburg loess is a very uniform silt of low plasticity with approximately 5 to 10 percent clay sizes. Caliche deposits belong to the limestone family. Two caliche materials were investigated -- one was a soft, very light gray deteriorated limestone from the vicinity of Georgetown, Texas, and the other was a very soft, pink-gray calcareous material from the vicinity of Mission, Texas. Caliches are commonly used in highway and airport construction throughout many of the southwestern states. The California pumice is a very porous, nonplastic volcanic ash that shows considerable breakdown under modified AASHO compaction.

Compaction Characteristics of Soils Under Dynamic and Static Compaction

Types of compaction used

26. Specimens for CBR and other tests were compacted in the 6-inch diameter CBR mold to a height of 4-1/2 inches, using two methods of

compaction; namely, static and dynamic. In static compaction the soil was placed in the test mold in one layer, rodded lightly, and compacted by a piston with the same diameter as the inside diameter of the mold under static load in a manner similar to the Porter or California method. In dynamic compaction the soil was placed in several layers in the mold and compacted by several blows of a free-falling hammer weight, such as is used in the AASHO method. The compactive effort was varied by changing the number of blows per layer or the amount of the statically applied load. The compaction characteristics for most of the soils studied are shown on plates 5 through 23.

Static compaction

27. Shape of compaction curves. It can be noted, by reference to plates 5 through 23, that for the range of compactive efforts used, soils of medium to high plasticity and pumice show maximum densities and minimum optimum water contents using the Porter static (2000 psi) procedure. It can also be noted that under static compaction the soils of medium and high plasticity are the only ones for which a compaction curve with a definite optimum water content can be developed similar to that as obtained dynamically. Some cohesionless sands and soils of low plasticity under static compaction have, in general, flat-shaped curves with little or no tendency to develop a definite optimum water content, which is contrary to field experience. A smooth curve for the wet side of optimum for soils other than those of medium to high plasticity can be developed by altering the static compaction procedure such that the load is stopped as soon as free water appears.

28. Density gradient. At the beginning of these studies it was realized that it would be highly desirable to have a uniform distribution of density within the test specimen. In order to obtain data on the density gradient of remolded specimens compacted statically, a series of tests were performed on a sand, a silt, and a lean clay, using a 2000 psi static load. All specimens were compacted at the optimum water content for the 2000 psi effort according to the standard procedure for this method. Under Porter compaction (2000 psi) with the load applied from one end only, the top as compacted was the most dense and the bottom the least dense on all soils tested. The difference in density between the top and the bottom of the specimens was as follows: for the silt, about 5 pounds per cubic foot; for the sand, about 2 pounds per cubic foot; and for the lean clay, about 1 pound per cubic foot (see plates 62, 63 and 64).

29. Having determined the density gradient for the soils when compacted statically according to the standard procedure for the above method, an attempt was made to obtain more uniform density distribution by applying the load once to each end of the specimen. Under static compaction (2000 psi) when the load was applied once to each end, the results were as follows: on the sand, the density was brought to a very uniform condition, the variation from top to bottom being negligible.

However, the average density obtained by this method of static compaction was 4 to 5 pounds less than that obtained by the modified AASHO procedure. No static tests of this type were conducted on the silt. On the lean clay the average density was increased approximately two pounds over that obtained by the standard static procedure, the ends being brought to approximately the same density and the middle being the least dense by approximately one pound (see plates 62 and 63).

30. Breakdown of particles under compaction. Tests were performed on Eglin Field sand to determine the amount of breakdown of cohesionless particles under static compaction. Plates 65 through 67 show that for this sand under static compaction, the grain size distribution is altered in the range of medium to very fine particles. The breakdown caused by this compaction method, although not excessive, is undesirable.

Dynamic compaction

31. Shape of compaction curves. Reference to plates 5 through 23 indicates that for the compactive efforts used, cohesionless soils and soils of low plasticity show maximum densities and minimum optimum water contents when the modified AASHO procedure is used. Under dynamic compaction practically all soils develop compaction curves with definite optimum water contents, which agrees with field experience. Poorly graded sands and gravels develop very erratic test points under dynamic compaction and determination of a definite optimum water content is usually very difficult, if not impossible. Plates 5 and 7 show that for a sand and a silt the maximum AASHO densities obtained in the CBR mold are one to two pounds per cubic foot less than those obtained in the standard Proctor mold. As shown later, small variations in density and molding water content greatly affect the CBR values for some soils. Thus it is necessary to conduct all compaction control tests for preparation of samples for the CBR test in the 6-inch diameter CBR mold.

32. Density gradient. In order to obtain data on the density gradient of remolded specimens compacted dynamically, a series of tests was performed on a sand, a silt and a lean clay, using modified AASHO compactive effort. All specimens were compacted at the optimum water content for this effort according to the modified procedure. Under modified AASHO compaction, the sand and the silt showed the bottom of the specimen as compacted to be the most dense and the top to be the least dense, the difference in density being in the order of four to five pounds per cubic foot for the silt and two to three pounds per cubic foot for the sand. Dynamic compaction on the lean clay resulted in a tendency for the middle of the specimen to be slightly more dense than the ends, the difference in density being approximately one to two pounds per cubic foot. These data are shown on plates 68 through 71, inclusive.

33. Having determined the density gradient for the soils when compacted dynamically according to the standard procedure for the above method, an attempt was then made to obtain more uniform density

distribution by staggering the number of blows per layer. Under this method, which used a total number of staggered blows equal to modified AASHO effort, the sand and the silt had the highest density shifted toward the center, and the ends tended to approach equal densities (see plates 68, 69 and 70). On the lean clay no advantage seemed to be gained by staggering the blows (see plate 71).

34. It is desired to emphasize the fact that the above tests were performed at only one molding water content for each soil. These tests show that the density gradient is markedly different for different soils. Further tests on a clayey sand and a silt showed that the density gradient changed with variation in the molding water content (see plates 72 and 73). Note on these plates that the trend is for the density to become more uniform throughout the specimen with increase in molding water content under a given compactive effort. This may account in part for the high dry-of-optimum and low wet-of-optimum CBR values which are usually obtained at equal densities and which will be discussed later.

35. Breakdown of particles under compaction. Tests were performed on California pumice and Eglin Field sand to determine the breakdown of cohesionless particles under dynamic compaction. Plate 74 shows the breakdown of pumice under modified AASHO compaction for molding water contents of 6 and 30 percent. As can be seen from these curves, the breakdown of the material at 6 percent water content is appreciable more than at 30 percent, showing that in studying the breakdown of particles under compaction the molding water content is a factor to be considered. This material would probably also break down under field compaction, because of the presence of unsound particles.

36. Plate 75 shows that under dynamic compaction the grain size distribution of Eglin Field sand is altered in the range of the coarse particles and the range of fine to very fine, but that the distribution remains practically unchanged for the medium particles. The breakdown under this method of compaction, like that under static, although not excessive is undesirable.

37. As has been pointed out by other investigators, material should not be used more than once in establishing the water content-density relationship, or in remolding specimens for any soil test. Beside the breakdown of some materials containing friable particles, other soils having enough cohesion to do so will retain the density in small lumps from the previous compaction, even though the soil mass is thoroughly kneaded. It is possible that this latter characteristic may be applied to advantage in field compaction. Plate 18 shows what happened to a lean clay when the material was recompacted. The third time the material was used, the compaction characteristics had altered as follows:

	Modified AASHO		Standard AASHO		15-Blow AASHO	
	Opt. w %	Max. γ_d Lb/cu ft	Opt. w %	Max. γ_d Lb/cu ft	Opt. w %	Max. γ_d Lb/cu ft
Initial compaction	15	113	18	107	19	105
Third time used	12	119	15	110	17	108
Change	-3	+6	-3	+3	-2	+3

Static versus
dynamic compaction

38. Summarizing, it appears that soils of medium to high plasticity and pumice obtain greater densities at lower optimum water contents when the Porter static (2000 psi) procedure is used, whereas cohesionless sands and soils of low plasticity obtain greater densities at lower optimum water contents when the modified AASHO procedure is used. The range of results are shown in table 3. Under dynamic compaction, practically all soils develop compaction curves with definite optimum water contents, which agrees with field experience. Under static compaction, the soils of medium and high plasticity are the only ones for which a compaction curve with a definite optimum can be developed. In addition, it was found that static compaction on soils of low plasticity did not produce compaction curves with a definite optimum water content. This is contrary to field experience.

39. It appears that it is possible to obtain a uniform density gradient under static compaction. However, to accomplish this uniformity, the load must be applied once to each end of the specimen, or an apparatus must be used which would allow a movable piston at each end of the specimen during compaction. Under dynamic compaction it appears practically impossible to establish a standard procedure of a constant number of blows or of staggering blows to give uniform density, because of the fact that a change in the type of soil and the molding water content causes a change in density gradient. Therefore, if dynamic compaction is used for the preparation of CBR test specimens, some means should be developed of accurately and rapidly determining the density of approximately the top two inches of the specimen as tested. This problem deserves more study than time has allowed during this investigation.

40. Breakdown of cohesionless particles under both methods of compaction is undesirable but may not exceed that experienced under field compaction. Because of this breakdown and because other soils having enough cohesion to do so may retain the density in small lumps from the previous compaction, material should not be used more than once in establishing each point of the water content density curve. Neither should material be reused in remolding specimens for any soil test.

Constant number of blows to obtain 95% of modified AASHO density

41. In order to save time and simplify methods for preparing specimens at specification density, it was decided during the course of the investigation to determine whether there was any one dynamic compactive effort which would obtain 95 percent of modified AASHO density at the optimum water content for all plastic soils. The results of this study, which was a co-operative project between various Engineer Districts and the Experiment Station, are covered in Appendix B. These tests showed that the dynamic compactive effort required to obtain 95 percent of modified AASHO maximum density was a function of the plasticity of the soil; the required compactive effort increasing with increase in plasticity. For 26 blows per layer, using the modified AASHO method, a variation from 93 to 98 percent of modified AASHO density was obtained. This variation was considered to be too much and therefore a constant number of blows was not recommended.

Effects of Molding Water Content, Density and Method of Compaction on CBR

General

42. The effects of water content and density on remolded samples are so closely related that it became necessary to use a method of plotting test data which would allow a ready comparison of the effects of both of these variables on the CBR. Plate 79, as do numerous other plates with this appendix, illustrates the manner in which it was decided to present test data when it was available in sufficient quantity. The left-hand plot shows the basic compaction and CBR test data. The penetration test was performed on all specimens used in the development of each compaction curve. The center plot of CBR versus molded dry density for different molding water contents was obtained from data shown on the left-hand plot. This was accomplished by plotting at a constant molding water content the molded dry density and corresponding CBR value for each compactive effort. Each pair of these values gives a point on one of the curves in the center plot. In like manner, the right-hand plot was obtained from the center plot by showing the various combinations of CBR and molding water content for a constant molded dry density. In the case of soaked specimens, as illustrated on plate 80, the procedure is repeated except that the soaked instead of the as-molded CBR value is plotted versus molded dry density and molding water content. It was impracticable to plot soaked CBR values versus soaked densities and soaked water contents. In addition, the soaked plots as shown are of more value, because a designing engineer is interested in knowing what the ultimate soaked CBR value of a given material will be after it has been placed in the field at a certain water content and compacted dry density. These plots show that relationship and give a complete picture of the behavior of a soil for any desired range of water content and density in which the soil would be tested.

Static compaction

43. Density effect. Plates 76 (silt), 77 (sand), 78 (clay), 79, 80 and 81 (clay-sands) show the effect of variation of density and molding water content on the CBR for several different soils under static compaction. Reference to these plates shows that the CBR is extremely sensitive to change in density, and the higher the density range the more sensitive the CBR becomes. This condition exists for both the as-molded and soaked conditions, as shown on plates 79 and 80, the difference being of course a lower range of CBR values in the soaked condition. Typical stress penetration curves for these tests are shown on plates 41 (silt), 45 (sand), 47 (clay) and 49 through 51 (clay-sands). Reference to these plates shows that all the soils, except soaked silt at 2000 psi, soaked Vicksburg clay-sand at 3900 psi, and soaked Marietta clay-sand at 2000 psi, exhibit concave-downward stress-penetration curves.

44. Water content effect. Reference to the families of curves shown on the compaction-CBR plates mentioned in paragraph 43 shows that the CBR is extremely sensitive to change in molding water content on all except the sand and the Marietta clay-sand at low densities, either in the as-molded or soaked condition. Typical of the test results on specimens tested as molded are those of the clay-sand shown on plate 79 which illustrate that, for a constant density, the higher the molding water content the lower the CBR. The general trend of these curves is normally expected for as-molded specimens of any soil (except clean sands) which for a constant density are known to decrease in shearing resistance as the water content or degree of saturation increases.

45. Except for sands, the data for the clay-sand shown on plate 80 are typical of the results obtained on samples tested in the soaked condition. These data show that for a constant molded dry density the higher the molding water content the higher the CBR. This is a reversal of the behavior of specimens tested as-molded and is caused by the fact that the soaked specimens experienced considerable swell on the dry side of optimum, the drier the molding condition the more the swell.

Dynamic compaction

46. Density effect. Plates 78 and 82 through 100, inclusive, show the effect of variation of density and molding water content for most of the soils tested under dynamic compaction for both the as-molded and soaked conditions. Plate 101 shows the relationship of CBR to molded dry density at optimum water content and maximum density for three compactive efforts for most of the soils tested. Reference to these plates shows that the CBR is extremely sensitive to changes in density, and the higher the density range the more sensitive the CBR becomes. This sensitivity exists for specimens tested in both the as-molded and soaked conditions, the difference being of course a lower range of CBR values in the soaked condition. For a given soil, the CBR versus density curve usually has an extremely abrupt increase in slope for the range of density between standard AASHO and modified AASHO. It can also be noted

that, except for cohesionless, free-draining materials, where sufficient tests have been performed to develop families of curves, the CBR at constant molding moisture content increases with increase in density. This increase continues up to the density for which that water content is optimum, but thereafter the CBR decreases with increase in density. This relationship holds for both the as-molded and soaked specimens and is due to differences in the structural arrangement of the components of the compacted soil system formed at the time of compaction.

47. Water content effect. Reference to the same compaction-CBR plates mentioned in paragraph 46, above, shows that the CBR is extremely sensitive to change in molding water content on all except free-draining sands and that the CBR values obtained on soaked samples are much less than those on samples tested as molded. Test results obtained on specimens tested as-molded followed a trend normally expected, i.e., for a constant density the higher the molding water content the lower the CBR. Plate 102 shows the relationship of CBR to molding water content at 95 percent of modified AASHO maximum density for a large number of the soils tested in the soaked condition. Plate 103 gives, for most of the soils tested, the relationship of soaked CBR to optimum water content for any dynamic compactive effort and for 95 and 100 percent maximum density for the effort used.

48. For soils other than free-draining and high-swelling, CBR stress-penetration curves (plates 39, 43, 46, 49, 50, 51 and 52) analyzed in conjunction with compaction and CBR data on both soaked and unsoaked specimens (plates 82, 87, 92, 96, 97, 98 and 99) show that under dynamic compaction with a given effort the rigidity of the soil mass appears to decrease as dry density increases and as optimum water content is approached from the dry side. This decrease in rigidity is offset by increase in density; hence the CBR increases up to optimum water content, at which point the best combination of density and water is obtained. An exception to this relationship was exhibited by sample 5, a sand-clay in which the CBR was found to be higher on the dry side of optimum than at optimum water content (see plates 43 and 87). However, for all soils in this group, once optimum water content is passed, both rigidity and density decrease rapidly and the CBR decreases very rapidly. On this intermediate group of soils for a given density under dynamic compaction the higher the molding water content the lower the CBR value, even though all specimens were soaked to the same degree of saturation prior to penetration. The fact that this general relationship persists for soaked specimens which showed little or no swell is especially significant, since it is generally believed that specimens of a given soil at equal densities with equal degrees of saturation have equal shearing resistances. The variations shown are caused by basic differences in the structural arrangement of the components of the compacted soil system formed at the time of compaction. This arrangement in turn is controlled by the molding water content, density and method of compaction used in preparing test specimens.

49. On soils exhibiting excessive swell, such as adobe clay, the effects of molding water content are not reflected in the test results on soaked specimens. This is shown by reference to plates 47 and 78, which show that, due to excessive swell, the spread of soaked CBR values is small, regardless of molding water content or type of compaction. However, it can also be seen that, although the spread of values is small, the dry-side specimens had the lowest CBR values and consequently compaction on the dry side of optimum must be avoided if swell is to be kept to a minimum.

50. It is of special interest to note that on California pumice, a material composed of very porous particles, the difference between the initial water content and the molding water content of dynamically compacted test specimens has a pronounced effect on the soaked CBR. Initial water content refers to the water content of the material immediately before more water is added in the performance of a soil test, such as the compaction test. This distinguishes it from the molding water content, which is the actual water content at the time of compaction. These data are shown on plate 104. It is believed the variations shown are explained by the fact that the particles, being porous, were capable of absorbing water, and the water initially in the material had time to be largely absorbed. Thus the amount of water around the particles (not absorbed) during compaction was a function of the difference between initial and molding water content. Therefore, for the range of density and water content shown on plate 104, the larger this difference (molding minus initial water content) the more water was present between particles during compaction and the less stable the soil mass became.

Comparison of dynamic and static compaction

51. Table 3 shows a comparison of static and dynamic compaction and soaked CBR data for the eight bracketing soils. It should be kept in mind that densities shown on this table and the various plates, except where noted, are average densities and that density gradient will affect the test results to some extent. It will be noted by reference to plate 105 that for clean sand the range of densities for the statically compacted material is somewhat lower than that for the dynamically compacted material. It was impossible with the equipment available to obtain modified AASHO density statically. However, it appears that whereas density has a decided effect, water content has no appreciable effect on the CBR for this material for the two methods of compaction. Reference to plate 106 shows that for a silt of low plasticity, exhibiting enough swell to alter the initial compacted density, there is no appreciable difference in the soaked CBR values obtained by the two methods of compaction, until the wet side of optimum is reached. Once the wet side is reached, for a given molding water content, the dynamically compacted material shows a decrease in CBR with an increase in density, whereas the statically compacted material shows an increase with increase in density. Plates 107, 108 and 109 show CBR data on two

clay-sands for both dynamic and static compaction. The physical properties obtained by the two methods of compaction on these materials are markedly different. On the dry side of optimum, soaked CBR values are in most instances approximately two to two and one-half times as great under dynamic compaction as they are under static compaction for the same density and molding water content. On the wet side of optimum (for a given water content) the shearing resistance for soaked specimens under dynamic compaction decreases with increase in density, while under static compaction it generally shows an increase with increase in density.

52. Inasmuch as the molding water content is a prime factor in controlling the physical properties of all except free-draining soils when subjected to the CBR test, it follows that, in remolded soil, duplicate laboratory specimens can not be prepared unless molding water contents are duplicated, even though water contents and densities attained subsequent to molding are duplicated. In other words, if a soil is molded at some given water content and then this water content is allowed to increase or decrease by a given amount, another identical soil specimen can not be reproduced unless the whole cycle is reproduced, starting at the same molding water content (see table 4).

53. Summarizing, it can be said that for all soils except clean cohesionless sands, which are chiefly affected by density, the CBR is extremely sensitive to the molding water content, density and method of compaction. For soils of low plasticity, the CBR is more sensitive to molding moisture and density under dynamic than under static compaction. Of particular interest is the shape of the CBR stress-penetration curves which, under static compaction, are practically always concave-downward, regardless of the initial molding water content. However, under dynamic compaction, except for clean sands and high-swelling clays, concave-upward shaped curves are practically always obtained on specimens compacted near optimum and on the wet side of optimum, both for the as-molded and soaked conditions. Under dynamic compaction, soils compacted on the dry side of optimum usually obtain concave-downward shaped curves. Also of particular interest is the fact that under static compaction these intermediate soils between clean sands and impervious high-swelling clays always swell when soaked, whereas under dynamic compaction they do not. This difference and the other variations obtained by the two methods of compaction with varying water content and density are caused by differences in the structural arrangement of the components in the compacted soil system formed at the time of compaction. All laboratory test results are considered qualitatively correct, but may not be quantitatively correct due to the confining effect of the 6-inch CBR mold.

Drying back (curing) from wet side compared with molding on dry side

54. Table 5 will give some indication as to how much the soaked bearing value may be increased by curing sand-clay materials before soaking for testing and how the cured materials compare with the materials

molded on the dry side of optimum. It appears from these data that, although the soaked bearing value of these materials molded on the wet side of optimum is improved slightly by curing, better bearing values are obtained by initially molding the materials on the dry side.

Effects of Molding Water Content, Density and Method of Compaction on Tests Other than CBR

General

55. In order to determine whether the same trends in shearing resistance as indicated by CBR tests would also be indicated by other types of physical tests, triaxial and unconfined compression tests were conducted on a few typical soil types ranging from sands to clays. A few consolidation tests were also performed, in order to compare the compressibility characteristics of soils compacted dynamically at different water contents.

Triaxial compression tests

56. Specimens for these tests were 1.4 inches in diameter and approximately 3 inches in height. All specimens except sands were cut from samples which had been previously compacted in the CBR mold under dynamic compaction. Sand specimens were compacted directly in the standard type of forming jacket. No soils were compacted statically for these tests. Triaxial tests were performed on soaked specimens molded initially on the dry side of optimum, at optimum, and on the wet side of optimum as for the CBR test. Modified AASHO compaction was used in the majority of cases. In general, the tests were of the consolidated quick type, specimens being saturated in the triaxial apparatus prior to loading. Lateral pressures of 0.1, 0.3 and 0.9 ton per square foot were used for consolidating specimens. In some instances, quick type tests were conducted under lateral pressures of zero and approximately one ton per square foot. A constant stress type of machine was used to apply the axial loads, and tests usually took from 5 to 10 minutes to reach failure.

57. The results of these tests are shown on plates 110 through 122. It can be noted by reference to plate 110 that the high-swelling adobe clay showed, as did the CBR for this material, that the stability on the dry side of optimum after soaking is low, due to excessive swell. Data on plate 110 for the two free-draining sands show, as did the CBR values for these materials, that the shearing resistance is practically independent of molding water content.

58. It can be noted by reference to plates 110, 111, 112, 114, 117 and 120 that the stress-strain curves for consolidated quick, saturated triaxial specimens show the same characteristics as the CBR penetration curves, i.e., higher stress per unit of strain on the dry side than on

the wet side of optimum. In other words, at a given density, when the stress is taken at a low strain, the lower the molding water content the higher the shearing resistance, even though the materials were saturated. However, for equal as-molded densities, the results of saturated consolidated quick tests for wet-side specimens show very nearly the same maximum shearing resistance as the dry-side specimens. It must be remembered that these specimens were consolidated before testing and that the more plastic wet-side specimen consolidated more than the dry-side specimen; hence at the time of testing the wet-side specimen was more dense than the dry-side specimen. Thus the difference in density probably compensates for the difference in physical behavior in this instance.

Unconfined compression tests

59. General. A comprehensive series of unconfined compression tests was performed on dynamically compacted Vicksburg loess in the as-molded condition. In addition, a full series of tests was performed on the Vicksburg clay-sand under dynamic and static compaction in the as-molded and soaked conditions. The as-molded specimens for the unconfined compression tests on loess were compacted in a mold 2 inches in diameter by 4 inches in height and the whole specimen tested. The clay-sand was compacted in the CBR mold and a test specimen 2.8 inches in diameter by 5 inches in height was cut from the compacted material. Specimens for soaked tests were soaked in the CBR mold after compaction prior to cutting them out for the test. All density computations were made before removing the specimen from the CBR mold.

60. Dynamic compaction. Compaction and unconfined compression test data including stress-strain curves for dynamically compacted Vicksburg loess and Vicksburg clay-sand are shown on plates 123 to 127, inclusive. It can be noted from these data that it is possible for the maximum unconfined compressive stress at a constant water content to decrease with increase in density, either in the as-molded or soaked condition. If plotted, the stress at a low strain would show the same trend. Note also that plates 124 and 127 show that the yield point increases with increase in molding water content for a given dynamic compactive effort. The behavior of either the as-molded or the soaked clay-sand at low strains (see right-hand plots on plates 125 and 126) is similar to that obtained on this material by the CBR tests. That is, for a given density the shearing resistance of the as-molded or soaked specimens molded initially on the dry side of optimum is greater than that of the specimen molded initially to the same density on the wet side of optimum with the soaked specimen having a lower range of shearing resistances. It can be seen that for a given density the soaked maximum unconfined compressive stress increases with increase in molding water content up to optimum and shows a decrease thereafter. This variation in maximum compressive stress for the soaked soil is not contradictory to the variations obtained by CBR tests, because the CBR test measures the shearing resistance at low strain. As just pointed out,

unconfined compressive stresses at low strains show results similar to those of CBR tests.

61. Static compaction. Compaction and unconfined compression test data including stress-strain curves for statically compacted Vicksburg clay-sand are shown on plates 128 through 130. Plate 130 shows that the strain at the yield point is practically constant for a given static compactive effort, regardless of change in water content and density. A comparison of plate 127 with plate 130 shows that the curves for dynamically compacted specimens are generally concave-upward on the wet side of optimum only, and those for statically compacted specimens are generally concave-downward throughout the range of molding water contents. It is interesting to note on plate 128 that the variation of the as-molded unconfined compressive stress (maximum or low strain) with molding water content for constant densities follows a trend very similar to that obtained for CBR. In like manner, plate 129 shows that the soaked unconfined compressive stress (maximum or low strain) follows the same trends and are in turn similar to the trends obtained with this material in the soaked CBR test. The variations just described and those mentioned in paragraph 60 are further evidence of the differences in the structural arrangement of the components of the compacted soil systems formed by these two methods of compaction.

Comparison and correlation of CBR and unconfined compression tests

62. Plates 131 and 132 show a comparison of unconfined compressive strengths on the Vicksburg clay-sand, compacted by both the dynamic and static methods in the as-molded and soaked conditions. Data on these plates were taken from data on plates 125, 126, 128 and 129. Reference to these plates and plates 107 and 108 for comparable CBR data shows that on the dry side of optimum and near optimum, soaked CBR values and maximum unconfined compressive strengths are in some cases approximately two and one-half times as great under dynamic compaction as they are under static compaction for the same molded dry density and molding water content. On the wet side of optimum (for a given water content) the shearing resistance under dynamic compaction decreases with increase in density, while under static compaction it generally shows an increase with increase in density.

63. The data on plate 133, which are for the unsoaked Vicksburg clay-sand, are taken from plates 107 and 131, and show for a density range of 92 to 99 percent of modified AASHO maximum the following:

a. For a molding water content dry of modified AASHO optimum the CBR increases more rapidly with increase in density under dynamic compaction than under static compaction, the difference in rate of increase becoming more pronounced after 96 percent modified AASHO maximum density is passed.

b. For the same molding water content as in a, above, the maximum unconfined compressive stress increases more rapidly with increase in density under dynamic than under static compaction, the difference in rate of increase being fairly uniform from 92 to 99 percent of modified AASHO maximum density.

c. For a molding water content wet of modified AASHO optimum, the CBR and maximum unconfined compressive stress for both static and dynamic compaction show slight but practically equal increases with increase in density up to 96 percent of modified AASHO maximum; thereafter the static continues to increase, whereas the dynamic decreases.

A similar set of curves for dynamic compaction on loess (plate 134) tested in the as-molded condition show variations in CBR and maximum unconfined compressive stress to be parallel up to modified AASHO maximum density. Modified AASHO maximum density on the loess is approximately 111 pounds per cubic foot as compared to 130 pounds per cubic foot on the clay-sand.

64. The CBR penetration test measures the resistance of the soil mass to deform within itself and measures the combined influence of cohesion and internal friction. Resistance to deformation due to the confining effect of the surrounding material increases as the penetration proceeds to failure. Hence, a single-curve correlation between CBR and unconfined compression or triaxial compression for all soils, independent of normal load, cohesion and internal friction, is not expected. However, it is believed that if all three tests are basically sound, they should give a family of curves for given conditions of normal load, cohesion and internal friction under given conditions of water content and density. Reference to plates 135, 136 and 137 shows that the CBR and unconfined compression tests do correlate under these conditions, giving a family of curves for a given material. However, these families of curves, although similar for different materials, do not have numerical agreement. Sufficient data are not available at present to arrive at similar curves for triaxial test specimens. However, it is reasonable to believe that, although the curves would shift, the test data would correlate for any given normal load.

65. It is pointed out that a correlation of the type given herein in which a constant minor principal stress for the shear tests is maintained, is not a correlation which allows the CBR and unconfined compressive stress values to be used interchangeably in design. It is recognized that the ultimate correlation desired is that of CBR versus shear strength in which the shear strength is that obtained from a shear test performed under a minor principal stress equivalent to that induced by the design wheel load at the required depth indicated by the CBR. It is not known whether this shear strength should be the ultimate, or that at some lower percent strain. It is believed that efforts should be continued to arrive at such a correlation.

Consolidation tests

66. Plates 138 and 139 show the difference in the consolidation characteristics of soaked specimens of clay-sand and loess which were molded on the wet and dry sides of optimum moisture. It can be seen that for a given soil, even though the specimens were at equal densities and were all soaked before testing, the wet-side specimen shows considerably more consolidation than the dry-side specimen for the same pressure. This variation is additional evidence of the difference in the physical behavior of remolded soils and should be studied further.

Summary

67. Summarizing, it can be stated that molding water content, density and method of compaction control the shearing resistance of remolded soils as determined by the triaxial or unconfined compression test in a manner similar to the way they controlled the CBR. Thus it is apparent that no matter what method of test is used to determine the shearing resistance of remolded soils, the method of preparation of the test specimen governs the results to be obtained. The same statement holds true for the determination of any other physical property of soils such as compressibility, or consolidation characteristics, or permeability coefficients.

68. Specifically, it is shown that for soils of low to medium plasticity exhibiting little or no swell, triaxial or unconfined compression tests indicated that when the test stresses are chosen at low strains, dynamically compacted specimens showed the same trends in variation in stress as did the CBR for the same materials. That is, for a given molded dry density, the stress or CBR on the dry side of optimum is greater than the stress or CBR on the wet side of optimum for either as-molded or soaked specimens. This difference was not found for statically compacted specimens. Furthermore it was found for these soils that the CBR and maximum unconfined compressive test stress of dynamically compacted and soaked specimens near modified AASHO optimum are approximately two and one-half times as great as those of statically compacted and soaked specimens.

Height of Specimen, Soaking, Processing Gravel

General

69. To study the effect of other factors encountered in the preparation of test specimens for the CBR penetration test, several series of tests were performed on selected soils. These factors included height of specimen, methods and period of soaking, soaking surcharge, drainage time, and method of processing gravelly materials. These tests will be discussed in the paragraphs which follow.

Height of specimen

70. Special tests were conducted on cohesionless soils and soils of low to medium plasticity under (1) modified AASHO compaction and (2) experimental combinations of static and dynamic compaction, in order to study the effect of height of specimens on the CBR and the depth to which each soil type was affected by the penetration test. All tests were performed in the 6-inch diameter CBR mold and a constant height of 4-1/2 inches was used in the experimental tests. In these tests each layer of soil was sprinkled with a layer of chalk dust just prior to the placement of the next layer. After compaction the specimen was removed from the mold, cut in half, photographed, placed back together again, inserted in the mold and then penetrated. The specimens were then removed from the mold again, divided and rephotographed. The results of these tests are shown on plates 53 to 58, inclusive. The series of tests using actual modified AASHO compactive effort per layer was performed by compacting the test specimens to heights of 2, 3 and 5 inches and performing the CBR penetration test. The results of this series of tests are shown on plate 140.

71. It is believed that the depth to which the soil is affected by the penetration test is influenced by the water content, density, particle size and the structural arrangement of the components of the compacted soil system. As can be seen by reference to plates 53 through 58 and 140, it appears inadvisable to use a specimen height of less than 4-1/2 inches for the plastic and medium plastic soils. It is not fully understood why the cohesionless material shows an increase in CBR in going from a specimen height of 2 inches to one of 5 inches. However, this may possibly be due to slippage of the material along the base of the mold in the shorter specimens, as well as to differences in density gradient. It is believed that a minimum specimen height of 4-1/2 inches, as originally recommended by Porter, should be used for all soils.

Methods of soaking

72. In the case of soils of medium to high plasticity it was considered desirable to know prior to penetration what the most expeditious method of soaking these soils was, and to determine the water content gradient under different soaking conditions. In order to obtain data on these factors, specimens of different soils were compacted in the CBR mold under dynamic compaction and allowed to soak from the bottom only and from the top and bottom for various periods of time. The specimens were then removed from the soaking tank, penetrated in some cases and in other cases extracted from the mold and cut into several sections for water content gradient determinations. The results of these tests and other similar tests are shown on plates 85, 86, 91, 92, on plates 141 to 146, inclusive, and on table 6.

73. Effect of method of soaking on water content gradient. In considering the possibility of reducing the height of specimens to expedite compaction and soaking, especially for the more plastic soils, tests on the method of soaking were conducted to determine the water content distribution. Specimens were soaked from the bottom only, from the top only, and from the top and bottom. Preliminary tests on Berry Field lean clay soaked from the bottom only for periods of 4, 11 and 21 days (see plates 143 and 144) showed that the major water content change occurred in the bottom inch. For these conditions the water contents increased as follows:

Method of Soaking	Days Soaked	Molding w - %	Molded γ_d Lb/CuFt	Water Content of Bottom Inch after Soaking	Actual Water Content Increase - %
Bottom only	4	14.8	114.7	16.2	1.4
Bottom only	11	14.0	116.2	17.8	3.8
Bottom only	21	14.5	114.9	17.1	2.6

It can be noted on plate 144 that after 21 days soaking from the bottom only, the top inch of the specimens increased approximately 1.8 percent above the original molding water content, whereas those soaked 4 and 11 days showed practically no change in the top inch. To investigate this difference further, a second series of tests was performed on this material in which specimens were soaked 10 and 21 days from the bottom only, the top only and the top and bottom. Specimens were compacted from material which had already been used in the preliminary tests.

74. The results of the second series of tests are shown on plates 145 and 146 and indicated the following water content increases:

Method of Soaking Specimen	Days Soaked	Mod. AASHO Molded γ_d Lb/Cu Ft	Water Content Increase in Specimen in Percent			Av. γ_d After Soaking Lb/Cu Ft	Percent* Saturation After Soaking
			Top	Middle	Bottom		
Bottom only	10	119.2	2.6	0.8	2.5	118.6	90
Bottom only	21	119.4	3.1	0.6	2.4	118.9	90
Top only	10	118.6	3.8	1.1	3.1	117.9	90
Top only	21	119.1	3.1	0.8	3.9	118.1	89
Top & bottom	10	118.9	4.1	1.1	3.5	117.8	92
Top & bottom	21	119.4	3.1	0.6	2.2	118.3	90

*Computed on basis of total weight and average water content of specimen

It is believed the reason for these variations is that the bottom inch as soaked is usually the least dense as compacted and is therefore susceptible to additional water absorption; the top inch as soaked is allowed to expand and absorb water; the middle portion of the sample is usually the most dense as compacted, is the most confined portion of the sample and contains entrapped air. Although it absorbs a small additional amount of water, it does not absorb as much as the ends of the sample.

75. It appears that for the material tested there is no advantage to be gained, in so far as increasing the percent saturation is concerned by allowing specimens to soak beyond 10 days. However, as shown on plate 141, there is practically no change in the CBR value after 4 days soaking from top and bottom. As can be seen by the tabulation in paragraph 74, regardless of the method of soaking, the middle portion of the specimens absorbs only approximately 30 percent as much water as the top and bottom portions, which absorb approximately the same amount. Variations in water content increases are probably due to variations in density, which are known to be different for the top, middle and bottom portions of the specimen as molded (see plates 68, 69, 71 and 73).

76. Effect of method and period of soaking. It was found impracticable to soak soils of medium to high plasticity from the bottom only, because of the time required. Specimens of Stockton adobe clay compacted at modified AASHO density after soaking 60 days in this manner were not saturated. Some specimens of Berry Field lean clay required as long as 25 days to soak. However, when the specimens were soaked by submergence (top and bottom), four days were found to be sufficient for saturation (see plate 141). It appears that some soils of low plasticity may be satisfactorily saturated in four days by soaking from the bottom of the specimen only (see plate 142). Purely cohesionless soils may be saturated in a few hours by soaking from the bottom. Reference to plates 91 and 92, Berry Field lean clay, and plates 85 and 86, Vicksburg loess, again shows that the CBR for these materials when soaked top and bottom is markedly less than when the materials are soaked from the bottom only, even though specimens soaked from the bottom only had free water on top after soaking. It may also be noted that on loess the swell is approximately twice as much when soaked top and bottom as when soaked from the bottom only. On the lean clay the swell is about the same by either method.

77. Inasmuch as the soil system of the loess specimens soaked from the bottom only was apparently not disturbed by swell, these specimens show, for a given density, that the lower the molding water content the higher the CBR value (plate 85). However, the material soaked top and bottom (plate 86) exhibited enough swell to alter the initial dry-side compacted structure such that its effects are not reflected in the soaked CBR values of these specimens to the extent that they are in the specimens soaked from the bottom only.

78. In table 6, data are shown for soils of medium and high plasticity which give the water content distribution and the CBR after soaking from the bottom and from the top and bottom. Based on these data and that previously presented, it appears that for a standard procedure the most practical method is to soak the specimens by submergence for four days, although some materials will not require this long.

Soaking surcharge

79. The effect of this variable on plastic soils was studied by compacting several soils under dynamic compaction in the CBR mold, soaking under various surcharge loads and then penetrating under zero surcharge. Data obtained from these tests are illustrated graphically on plates 35, 36 and 147. In general the soaking surcharge would affect the CBR only if it prevented swell or caused consolidation during soaking. For the range of water contents and densities for the soils tested, it can be seen by reference to the above plates that with the exception of the loess the effect of the soaking surcharge above a 10-pound minimum causes only a small change in the numerical value of the CBR. However, small changes in CBR value on low bearing materials radically affect the design thickness of pavement and base course and it is therefore important that the soaking surcharge equal the weight of overburden in the field to plus or minus 5 pounds, as recommended in Part V of the main report.

Drainage time

80. The effect of this variable on the CBR was studied by subjecting specimens of dynamically compacted and soaked soils to various drainage periods prior to CBR penetration. The results of these tests, which were conducted on a sand, a silt and a clay, are shown on plate 148, where it can be seen that a drainage time of 15 minutes appears satisfactory for all soils tested.

Processing materials containing gravels

81. Tests were conducted in 6- and 12-inch diameter molds on three natural and processed gravelly materials to study the effect of large-size particles on the CBR. Processing the materials consisted of removing all particles larger than 0.74 inch and replacing them with equal percentages by weight of sizes 0.74 to 0.37 inch and 0.37 to 0.18 inch. The percentage of material finer than 0.18 inch thus remained constant. Modified AASHO compaction tests conducted in the 6-inch mold showed for the three soils tested that the maximum dry density obtained on processed materials was equal to or greater than the maximum dry density obtained on natural materials (see plates 11, 17 and 19). Specimens used in developing the compaction curves in the 6-inch molds were subjected to soaking and CBR penetration. The results of these tests are shown on plates 88, 90 and 93. All test results are summarized in table 2. No static compaction tests were performed on these three base course soils. The following pertinent results can be obtained from a study of the above plates and table:

- a. CBR value of the natural clay-gravel in the 6-inch mold is approximately 25 percent greater than the CBR value of the processed clay-gravel in the 6-inch mold. CBR values of natural sand-gravel and gravel-sand-silt in the 6-inch mold are approximately 50 percent of the CBR values of processed materials in the 6-inch mold.

b. CBR values of processed clay-gravel and sand-gravel in the 12-inch mold are approximately 40 percent less than CBR values of natural materials in the 12-inch mold. CBR value of processed gravel-sand-silt in the 12-inch mold is approximately 50 percent greater than CBR of natural material in the 12-inch mold.

c. CBR values of either natural or processed materials compacted in the 6-inch mold on the dry side of optimum are approximately 60 to 250 percent greater than CBR values at corresponding densities on the wet side of optimum for clay-gravel and gravel-sand-silt. Dry and wet of optimum natural and processed CBR values are estimated to be equal for sand-gravel.

There are insufficient data from these tests to warrant any definite conclusions concerning the effect of processing granular base course soils. Time did not permit further study during this investigation and therefore it is recommended that the present procedure for preparing samples containing gravel be continued. Inasmuch as experience has shown that erratic CBR test results are obtained when testing gravelly materials, several tests should be performed in order to obtain average representative results.

PART IV: POSTULATION ON THE STRUCTURAL ARRANGEMENT OF THE COMPONENTS OF COMPACTED SOIL SYSTEMS

General

82. As previously indicated in Part III of this appendix, molding water content, density and method of compaction control the physical behavior of remolded soils. The combination of these factors results in a certain type of compacted soil system being formed which may or may not retain the same general physical properties in the soaked state as obtained in the as-molded state, depending on the amount of swell. In the case of sands it appears that density is the only controlling factor and that molding water content and method of compaction have little or no effect on the shearing resistance. However, the physical behavior of all other soil types is definitely influenced by the above-mentioned factors and it is the purpose of this part to set forth a possible explanation for the variation in shearing resistance obtained on soils of low to medium plasticity exhibiting little or no swell and impervious high-swelling soils investigated in this study and reported on in Parts II and III of this appendix. The variations are explained from the standpoint of the structural arrangement of the components in the compacted soil system.

Dynamic compaction, soils other than free-draining and impervious high-swelling

83. Typical of this soil group is the Vicksburg clay-sand for which CBR and compaction data are shown on plates 96 and 97 and CBR stress-penetration curves on plates 49 and 50. Comparable unconfined compression test data are shown on plates 125, 126 and 127. For these soils under dynamic compaction, the structural arrangement of the components of the compacted soil system obtained is a function of the pore pressure developed during compaction, and hence a function of the molding water content. Two distinctly different types of arrangement of the soil system are obtained: one is characterized by elastic properties and the other is characterized by plastic properties. It appears that the line of demarcation is the line showing the relationship between optimum water content and density. For a given compactive effort, a material molded on the dry side of optimum fails in shear at low strain, but when molded wet of optimum the same material fails in shear at a much larger strain. On <u>soaked specimens</u> of the same material <u>at the same density</u> on the wet and dry sides of optimum moisture, the CBR and unconfined compressive stress at low strain is higher for the specimen molded on the dry side of optimum. On <u>soaked</u> specimens molded <u>initially</u> at a <u>constant water content</u> on the <u>wet side</u> of the line of optimum water contents under different compactive efforts, the stress per unit of strain decreases with increase in compactive effort, i.e., the CBR or unconfined compressive stress decreases with increase in density. This indicates that the soil mass becomes more plastic with an increase in compactive effort (more blows per layer).

84. A possible explanation of these phenomena is as follows: On the dry side of the compaction curves, the water present in the pores of the soil mass during compaction acts as a lubricant[1], and increasingly so as the optimum is approached. A relatively small amount of the compaction energy is dissipated in developing pore pressure. On the wet side of the compaction curves, the water present in the pores has reached an amount such that it no longer acts as an aid to compaction but is detrimental, in that pore pressure develops. A relatively large amount of the compacting energy is dissipated in the form of instantaneous pore pressure. The amount of energy dissipated increases as the water content increases. Since considerably less effective energy is utilized in obtaining the same range of densities on the wet side of optimum as was obtained on the dry side of optimum, either one or both of two phenomena are taking place:

a. The lubrication of the mass has reached a point such that the energy required to force the particles together is greatly reduced.

b. The particles are arranging themselves in a manner such that the energy required to force them together is greatly reduced.

It seems, on first thought, that increased lubrication would be the explanation, but, if this were true, once the dry-side material was soaked (lubricated) to the same degree of saturation as the wet-side material, then the two, being at the same density, should behave similarly when stressed. The data on plates 50, 97, 126 and 127, however, show they do not. Pertinent comparisons of shearing resistance, density and degree of saturation for this soil are tabulated below.

	As Compacted			After Soaking				
Compactive Effort	w %	γ_d Lb/Ft^3	Degree of Saturation %	w %	γ_d Lb/Ft^3	Degree of Saturation %	Corrected CBR	Unconfined Compressive Stress at 1% Strain Ton/Sq Ft
Mod.	5.7	127.4	48	9.4	127.4	80	66	--
AASHO	9.0	127.4	76	9.5	127.4	80	25	--
Std.	7.1	118.1	46	12.3	118.1	80	15	--
AASHO	12.0	120.0	81	12.2	120.0	82	4	--
Mod.	6.2	127.0	53	8.8	127.0	80	--	0.52
AASHO	9.6	127.0	82	9.9	127.0	84	--	0.20
Std.	7.7	120.0	53	11.8	120.0	81	--	0.23
AASHO	11.6	120.0	78	12.4	120.0	84	--	0.10

(1) R.R. Proctor: "Fundamental Principles of Soil Compaction," Engineering News-Record, vol. 111, no. 9, 31 Aug. 1933, pp 245-248.

Since equal saturation (lubrication) at equal densities does not bring about like stress-strain characteristics, the distribution of the water, air and soil particles of the soil systems is different, and both of the above-mentioned phenomena take place.

85. A possible explanation of the mechanics of the arrangement of the water, air and soil particles in the three-phase soil system may be described as follows:

a. Dry side of optimum. The pore pressure developed on this side of optimum under the impact of the compaction hammer is negligible, if it develops at all. The soil particles are vibrated and wedged together and a certain amount of arching takes place. The coarser grains are not all uniformly coated with matrix (material passing the 200 mesh) and there is considerable grain-to-grain contact, with little, if any, matrix between the points of contact of the larger grains. The pore spaces are considered to be larger than those in a specimen compacted to the same density on the wet side of optimum and, to a large extent, are filled with air. This soil system is considered to be brittle and more or less rigid and it is considered to largely retain these properties even when the saturation is increased, provided the structural arrangement of the coarser grains is not altered. Inasmuch as the pore spaces are relatively large and air-filled and there is a minimum of matrix between the points of contact of the larger particles, this soil system will saturate at a more rapid rate than will a soil system at equal density on the wet side of optimum. (This difference has actually been observed in the laboratory.) Since the soil system is rigid, it behaves elastically at low strain; since it is brittle, it has a yield point at low strain.

b. Wet side of optimum. The amount of water in the soil system on the wet side of optimum is sufficient to allow the development of an appreciable amount of pore pressure under the impact of the compaction hammer. The soil matrix has sufficient water in it to allow plastic flow under the impact of the compaction hammer. Under a given impact of the compaction hammer the water in the voids is instantaneously stressed, the volume occupied by the air is instantaneously reduced and, for an instant, the soil matrix is caused to flow plastically from beneath the hammer. Instantaneous migrations of pore water toward and away from the air voids also occur. This happens under each blow of the hammer and the more blows that are given the soil system the more homogeneous the distribution of the water, air and soil particles becomes and the more plastic the soil system becomes. Possibly the matrix approaches a distribution through the soil system such that practically all larger grains are coated and the matrix is very uniformly distributed between and at the points of contact of the larger particles. In addition, the individual voids filled with air and/or water are considered to be smaller in comparison to the individual voids obtained in a specimen compacted to the same density on the dry side of optimum. Inasmuch as this wet-side soil system is already at a relatively high degree of saturation, further saturation does not cause any marked changes in the stress-strain characteristics. Since the void spaces are relatively small and the points of contact between the larger particles have a substantial amount of matrix between them, this soil system saturates at a slower rate than a dry-side system with the same total volume of voids. When this soil system is stressed, the resistance to deformation at low strain is largely due to the cohesion of the matrix, since nearly all the larger soil grains are separated from each other by the matrix. Entrapped air may also influence the stress-strain characteristics of the soil system at low strain. It is not until after a certain amount of movement and adjustment of the soil system under load that the larger grains begin to make intimate contact and the soil system begins to develop some so-called frictional resistance. This accounts for the concave-upward shape of the stress-strain curves at low strains and for the fact that ultimately approximately the same strength is reached by all samples at equal density. Since the wet-side soil system is of a more plastic nature than the dry-side soil system, it has a yield point at a higher strain than the dry-side system.

Dynamic compaction,
impervious high-swelling soils

86. The behavior of these soils during compaction, which is typified by that of the Stockton adobe clay, is very similar to that of the soils covered in the preceding paragraphs. The difference in the behavior of these two soil groups when saturated and stressed is due to the difference in the swelling properties. In these high-swelling soils the soil system is altered during soaking. Nevertheless, those compacted on the wet side continue to show at a given water content the phenomenon of decrease in the CBR or stress per unit of strain with increase in compactive effort — evidently indicating that the material becomes more plastic with continued compaction. Thus, on plate 78, it can be seen that under standard AASHO compaction with a density of 85 pounds per cubic foot the CBR is 2.5, whereas under modified AASHO effort with a density of 97 pounds per cubic foot the CBR is 2. It can also be seen that on the plot of CBR versus molding water content the standard AASHO effort curve falls below the 15-blow compaction curve.

87. Clay soils in this group with plasticity indices of approximately 30 or greater never develop a concave-upward shape in the CBR stress-penetration curve on the wet side of optimum, regardless of the

method of compaction and for the range of efforts used in this investigation (see plate 47). The stress per unit of strain will become less with increased compaction at a given water content but no reverse curvature develops. This may be explained by the fact that the material gets its strength largely from cohesion and does not contain sufficient sand grains for the transition from cohesive resistance to cohesive plus frictional resistance to take place, as described in paragraph 85 b for the wet side of soils other than free-draining or high-swelling. Other clay soils which do not swell excessively attain compacted soil systems on the wet side similar to those of the soils described in paragraph 85 above.

Static compaction,
general

88. Under static compaction the load is usually applied at a relatively slow rate (see description of method in paragraph 26) and the soil particles are forced together with lateral support without the benefit of vibration and impact received in dynamic compaction. Hence there is very little rearrangement of the particles. Since the particles are allowed to do relatively little shifting or rearranging under any condition of molding water content or load, a different arrangement of the soil system is created under static compaction than is created under dynamic compaction. A non-wedged type of soil system with considerable arching of grains may be obtained for all conditions of static compaction on all soils except clays. The CBR stress-penetration curve seldom develops a concave-upward shape on statically compacted soaked or unsoaked specimens (see plates 47, 49, 50 and 51).

Static compaction,
impervious high-swelling soils

89. The only case where appreciable pore pressure might develop under static compaction at a slow rate of strain (approximately 0.05 in. per min) is probably on the wet side of optimum water content for impervious soils. Because of the development of pore pressure, the impervious soils are the only ones for which a compaction curve with a definite optimum water content, similar to that obtained dynamically, can be developed under a given static compactive effort. Present indications are that static compaction on the wet side of optimum for these impervious soils has a slight tendency to produce a plastic type of soil system.

90. As pointed out previously, clays with plasticity indices of approximately 30 or greater never develop a concave-upward shape in the CBR penetration curve, regardless of whether they are compacted by dynamic methods or static methods. This was explained as being due to the fact that they get their strength largely from cohesion and therefore no transition occurs from cohesive to cohesive plus

frictional resistance, as described in paragraph 85 b for other soils on the wet side of optimum. However, there are quite a number of soils in this group that are outside the range of the above-mentioned clays. In these other soils (for example, see plate 41 for Vicksburg loess) both dynamically and statically compacted specimens develop concave-upward shaped CBR penetration curves. The statically compacted specimens, however, develop these curves to a much less degree than the dynamically compacted specimens. This may be explained by the fact that the material does not receive the kneading under static compaction that it does under the repeated impact of the hammer under dynamic compaction. The arrangement of the soil system obtained on these materials under static compaction may be similar to that obtained on the dry side of other than free-draining and impervious high-swelling soils, with the exception that there is considerably more arching of grains and less wedging.

Static compaction, soils other than free-draining and impervious high-swelling

91. The following is a possible explanation of the behavior of this group of soils under static compaction. Considerable arching of soil grains occurs under static compaction. These soils are pervious to the extent that no appreciable pore pressure can develop at static pressure of 2000 psi or less for the rate of application of static compaction load used. After optimum water content is reached under static compaction, the soil drains under continued increased static load instead of developing pore pressure; the water content decreases and the density increases until the soil skeleton has stabilized itself under the load. Therefore, the wet-side portion of the curve cannot be developed under a given static effort (see plate 79). However, the wet-side portion of the curve may be developed by stopping the load as soon as free water appears. This does not, however, give a plastic type of soil system as obtained on the wet side of optimum under dynamic compaction. This method of obtaining the wet side of the curve under static load is analogous to dynamic compaction, in which the effective effort on the wet side is actually less than the applied effort, due to the development of pore pressure.

92. The data on plate 79 seem to indicate that under very high static efforts it may be possible to develop part of the wet side of the moisture-density curve. The difference in behavior at higher pressures from that at lower pressures is not fully understood. One explanation for this may be that under very high static pressures the soil grains are crushed to the extent that the void spaces have been materially reduced in size.

PART V: CONCLUSIONS

93. The CBR test is considered to be only the actual penetration of the soil and variations in CBR test results are largely due to the method of preparation of the test specimens. The variations, however, are systematic and are caused primarily by the effects of molding water content, density and method of compaction. The variations are probably qualitatively valid but may not be strictly quantitatively valid, partly because of the confining effect of the 6-inch diameter CBR mold. Consistent laboratory results can be obtained only when the above variables are given full consideration. Unconfined compression and triaxial shear tests indicate that when the shearing resistances are chosen at low strains, similar trends are found to occur as found for the CBR. Inasmuch as the molding water content is a prime factor in controlling the physical properties of all except free-draining soils, it follows that, in remolded soil, duplicate laboratory specimens can not be prepared unless molding water contents are duplicated, even though water contents and densities attained subsequent to molding are duplicated. In other words, if a soil is molded at some given water content and then this water content is allowed to increase or decrease by a given amount, another identical soil specimen can not be reproduced unless the whole cycle is reproduced, starting at the same molding water content.

The CBR penetration test

94. The following conclusions can be drawn regarding the CBR penetration test:

a. In general, the effect of the penetration surcharge on the CBR increases with increase in density or decrease in plasticity of the soil. The penetration surcharge greatly affects the CBR of cohesionless soil, moderately affects that of soils of low plasticity, and has practically no effect on that of soils of medium to high plasticity.

b. A penetration rate of 0.05 inch per minute will give satisfactory results for all soils.

c. For consistency and more uniform results, all stress-penetration curves that develop concave-upward shapes should be corrected by extending the portion of the curve with maximum slope over 0.1-inch penetration to zero stress and establishing a new origin.

d. A closely-controlled constant strain type of loading apparatus should be used.

e. Test results on samples containing stones are erratic and modification to the test procedure is needed for these soils. Until such a procedure is developed, several tests should be performed in order to obtain average representative results.

Laboratory compaction of soils

95. The results obtained by a study of the compaction characteristics produced by dynamic and static compaction on soils warrant the following conclusions:

a. The maximum dry densities obtained by the modified AASHO dynamic compaction test do not equal those obtained by the 2000 psi Porter static method. The variation may be as much as 20 pounds per cubic foot, depending on the type of soil.

b. Under laboratory dynamic (dropping hammer) compaction, practically all soils develop compaction curves with definite optimum water contents, which agrees with field experience. The compaction curves for soils of low plasticity developed by the static method of compaction are not characteristic, since a definite optimum water content is usually not obtained.

c. The density gradient of compacted specimens is different for different soils and varies with the molding water content. Under the present dynamic compaction method , it is impossible to obtain uniform density in the test specimens. Fairly uniform density can be obtained by the static method of compaction, provided the load is applied at both ends of the specimen.

d. Material should not be used more than once in establishing the water content-density relationship, or in remolding specimens for any soil test. All control compaction for CBR tests should be performed in the 6-inch diameter CBR mold with the mold placed on a concrete floor or pedestal for firm support.

Preparation of remolded specimens for CBR test

96. As a result of the laboratory studies, the following-listed conclusions appear warranted:

a. Small changes in density greatly affect the CBR, especially at high densities in the order of magnitude of modified AASHO. The effect of density is most pronounced for cohesionless soils and soils of low plasticity.

b. Small changes in molding water content greatly affect the CBR of unsoaked laboratory samples, except on clean sands and gravels. At a constant density in the unsoaked condition the higher the molding water content the lower the CBR.

c. The CBR for all soils compacted statically and soaked showed an increase with increase in molding water content at a constant density. This is a reversal of the behavior of unsoaked specimens stated above and is caused by the fact that the drier the molding condition the greater the swell during soaking.

d. The CBR for impervious high-swelling soils compacted dynamically and soaked showed an increase with increase in molding water content for a constant density.

e. The CBR for specimens of soils of low plasticity with little or no swell when compacted dynamically and soaked showed an appreciable decrease with slight increase in molding water content for a constant density. This is especially significant, since it is generally believed by most engineers that the shearing resistance is not sensitive to the molding moisture within normal laboratory control for soaked specimens compacted to a given density.

f. Specimens compacted statically on the dry side of optimum swell more when soaked than specimens compacted dynamically to the same density at the same moisture.

g. Laboratory soaked specimens of soils of low plasticity compacted at optimum moisture content or dry of optimum moisture content by the dynamic method have greater CBR values than corresponding specimens compacted by the static method. On the clay-sand tested, the CBR for dynamically compacted soaked specimens were approximately two and one-half times greater than statically compacted soaked specimens at equal densities and at approximately modified AASHO optimum.

h. The same trends described for the CBR test in paragraph 96 c, d, e, f and g, above, also occur for maximum strength and stresses at low strain obtained by unconfined compression and quick triaxial tests, except for maximum strength values for soaked specimens compacted dynamically. These latter values showed trends similar to those obtained for soaked specimens compacted statically.

i. The CBR specimen height should not be less than 4-1/2 inches for all soils.

_j_. The most practical method for soaking test specimens is by submergence (soaking from top and bottom) for four days. Less time than this is required for some cohesionless soils.

_k_. In general, the effect of variation of the soaking surcharge on the CBR increases with increase in plasticity of the soil. The soaking surcharge greatly affects the CBR of soils of medium to high plasticity, moderately affects that of soils of low plasticity, and has practically no effect on the CBR of cohesionless soils.

_l_. Fifteen minutes is a satisfactory drainage time for all soils.

_m_. The extreme increase in CBR values for some soils above standard AASHO density may be partly due to the confining effect of the 6-inch diameter mold. Present indications are that the 6-inch diameter mold is not large enough at high densities (modified AASHO) on soils exhibiting less than 3 percent swell during soaking, nor for base materials containing 3/4-inch particles.

APPENDIX B

INVESTIGATION OF COMPACTION CHARACTERISTICS OF PLASTIC SOILS IN RANGE OF 95 PERCENT MODIFIED AASHO DENSITY

APPENDIX B

INVESTIGATION OF COMPACTION CHARACTERISTICS OF PLASTIC SOILS IN RANGE OF 95 PERCENT MODIFIED AASHO DENSITY

Purpose of investigation

1. The purpose of this investigation was to determine if there were any one compactive effort which would obtain 95 percent of modified AASHO maximum density at the optimum water content for all plastic soils. Usually, laboratory samples must be compacted to this density for design tests, since this is the density generally specified for field compaction of subgrade and base materials. To accomplish this study the U. S. Waterways Experiment Station was instructed by the Office, Chief of Engineers in November 1943 to make a study of all its available compaction data on plastic soils, and to contact certain District and Division offices for information and compaction test data on at least five plastic soils. Where complete data were not already available, additional compaction tests were requested to be performed.

Offices furnishing compaction data

2. In addition to the U. S. Waterways Experiment Station, the following-listed U. S. Engineer District and Division offices furnished compaction data for this study:

Boston District
Mobile District
New York District
Portland District
Missouri River Division
Southwestern Division

Method of analyzing the test data

3. The Experiment Station found on all soils studied that a semilog plot of dry density (at optimum water content) versus compactive effort gave for efforts between standard and modified AASHO a straight-line relationship. Dry density in pounds per cubic foot is plotted on the arithmetic scale and compactive effort in foot pounds of energy is plotted on the log scale. The density may also be expressed as a percent of some given density, such as that obtained by modified AASHO compaction. In comparing different soils, it is more convenient to express the density on a percentage basis.

4. In order to study this relationship more thoroughly, tests were performed on Vicksburg loess for six different compactive efforts. These efforts ranged from 15-blow AASHO to an effort three times as great as modified AASHO. Plate 155 shows the data from these tests. It can be seen that a straight line is obtained between standard AASHO effort and the effort three times as great as modified AASHO. However, a slight curvature occurs for efforts below standard AASHO. The Experiment Station has found this method of plotting compaction data to be a useful and reliable means of determining whether the compaction data are valid.

Test results

5. Classification tests. Results of classification tests on the 46 soils used in this study are shown on plates 149 through 154. It can be noted that all except two of these soils show 70 percent or more of material passing the 35-mesh screen. For simplicity and convenience, all samples from the various Districts and Divisions were renumbered, and no attempt is made to distinguish between the samples from different localities.

6. Compaction tests. All data were plotted according to the method outlined in paragraph 3. In cases where the tests were performed in the standard AASHO mold, the energy was converted to the equivalent in the CBR mold, so that all data could be shown for the same size mold for comparative purposes. For all efforts and mold sizes investigated in the past, the amount of energy required for a given density for different size molds has been found to be approximately in the same ratio as the areas of the molds.

7. Upon plotting the data for the 46 plastic soils, it was found that the plasticity index was a very good criterion of the amount of compactive effort required to produce 95 percent of the modified AASHO maximum density. These soils divided themselves into groups as shown on plate 156. Plate 157 is a plot of plasticity index versus number of blows required to give 95 percent of modified AASHO maximum density in the CBR mold. It can be seen that, as the plasticity index increases, the compactive effort required to obtain 95 percent of modified AASHO maximum density also increases. For these soils, in which the plasticity index ranged from 2 through 50, the required compactive effort ranged from 16 to 31 blows per layer (compaction in the CBR mold in 5 layers using the 10-lb hammer dropped 18 in.). These curves are not to be considered as establishing rigid limits, but only to show the trend of compactive effort with increase in plasticity index.

Conclusions

8. As a result of this study, the following conclusions appear warranted:

a. On the soils investigated, one dynamic compactive effort (26 blows per layer) will give 93 to 98 percent of modified AASHO maximum. However, due to the sensitivity of the CBR test to density, this variation from 95 percent is too great.

b. The dynamic compactive effort required to obtain 95 percent of modified AASHO maximum density is a function of the plasticity of the soil -- the required compactive effort increasing with increase in plasticity.

c. Under dynamic compaction (on a semilog plot) a straight-line relationship exists between dry density at optimum water content and compactive effort for efforts between standard and modified AASHO. This plot can be used as a reliable guide in establishing the validity of dynamic compaction data.

APPENDIX C

INSTRUCTIONS FOR USE OF

FIELD IN-PLACE CALIFORNIA BEARING RATIO APPARATUS

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INSTRUCTIONS FOR USE OF
FIELD IN-PLACE CALIFORNIA BEARING RATIO APPARATUS

Introduction

1. Past experience with the present hydraulic jack type field CBR equipment used for conducting field in-place CBR tests has demonstrated it to be generally unreliable and inaccurate, due to the pulsating effect of the jack.

2. In the field tests on the Barksdale Field Pavement Behavior tests, near Shreveport, Louisiana, the Little Rock Engineer District constructed and used a new type of field in-place CBR equipment, utilizing a calibrated proving ring and Walker screw jack which eliminated the inaccuracy of the test due to the pulsations of the hydraulic jack. The Experiment Station, using this basic idea as a guide, has developed and constructed a set of field CBR equipment which has been given extensive field use on several large-scale tests. These field tests have been compared with laboratory undisturbed cylinder tests and, in general, good correlation has been found to exist.

3. In the subsequent paragraphs this newly developed equipment and method of test procedure are described. There are also included photographs and detailed drawings illustrating this equipment.

Field in-place CBR equipment

4. The equipment used to conduct field in-place CBR tests consists of the following:

a. Mechanical screw jack equipped with special swivel head for applying load to penetration piston designed as follows:

Maximum pressure	10,000 lb
Maximum lift	5 in.
Detachable handle	6-in. radius
High-gear ratio	120 rev. per in.
Low-gear ratio	360 rev. per in.

b. Two calibrated proving rings with the following ranges:

0 to 2000 lb
0 to 5000 lb

c. Circular penetration piston (3 sq in. area, 6-in. height) and internally threaded pipe extensions with connectors in the following lengths:

2	1-1/2 in. lengths
2	4-in. lengths
1	1-ft length
1	2-ft length
1	3-ft length

d. Dial gauge (reading to 0.0001 in.) for measuring proving ring deflection.

e. Dial gauge (reading to 0.001 in.) with adjustable dial extension clamp for measuring penetration.

f. Support for penetration dial, made of 2-in. angle iron and approximately 8 ft long.

g. Circular steel plate, 10-in. diameter, weighing 10 lb, with 2-1/32 in. diameter hold cut in center.

h. Several surcharge weights in the following numbers, weights and dimensions:

2 ... 10-lb weights, 8-1/2 in. diameter, slotted
3 ... 20-lb weights, 8-1/2 in. diameter, slotted

More or less of these weights may be required, depending on anticipated overburden pressures.

i. Truck equipped with heavy iron beam mounted across rear end with approximately 2-ft clearance above ground.

j. Two track jacks.

k. Other general equipment such as sample containers for moisture and density determinations, spatula, rod level, straightedge, digging tools, etc.

5. Detailed mechanical drawings of the screw jack and appurtenances are shown on plates 158 and 159. Plate 160 shows a photograph of the unassembled CBR equipment, while plate 161 illustrates a typical field set-up with this new type of equipment.

Field in-place
CBR test procedure

6. The step-by-step procedure usually followed in the conduct of field in-place CBR tests is given in the subparagraphs below:

a. Prepare the surface to be tested by removing loose and dried material, leveling the test area as perfectly as practicable.

b. Locate the truck so that the center of the iron beam on the rear end is directly over the surface to be tested. Place the track jacks beneath the ends of the iron beam and lift the truck so that no weight rests on the springs and so that the truck is approximately level. The truck should be loaded sufficiently to give a counterreaction of about 7000 pounds.

c. Install the swivel head and test jack to underside and center of iron beam, connect proving ring to end of jack, connect penetration piston to bottom of proving ring, using enough pipe extension to bring the piston to within 1 to 2 inches of the surface to be tested. Fasten rod level to pipe extension and adjust swivel head until the penetration piston is plumb. Then lock apparatus in position by tightening the clamping nut in the swivel head.

d. Place the steel plate beneath the penetration piston so that when the piston is lowered it will pass through the center hole.

e. Seat the penetration piston under a load of 3 psi. (For rapid setting use high-gear ratio of jack.)

f. Raise surcharge plate while seating load is on piston and spread clean fine sand to a depth of 1/4 to 3/8 inch over the surface to be covered by the plate. This serves to distribute the weight of the surcharge uniformly.

g. Apply surcharge weights to the steel plate equivalent to the load intensity of material and/or pavement which will overlie the subgrade or base, except that the minimum weight applied should be made up of the 10-pound circular steel plate plus one 20-pound surcharge weight. This minimum weight creates an intensity of loading equal to that created by the 10-pound weight used in the 6-inch diameter CBR mold in the laboratory.

h. Attach the penetration dial clamp to the piston so that the dial rests upon the dial support.

i. Set the dial gauges to zero.

j. Apply load to penetration piston so that the rate of penetration is approximately 0.05 inch per minute. By using the low-gear ratio of jack during test, a uniform rate of penetration can be maintained by the operator. Record the deflection of the proving ring at penetration depths of 0.025, 0.05, 0.075, 0.1, 0.15, 0.2, 0.3, 0.4 and 0.5 inch. Compute the bearing value in pounds per square inch and the CBR in percent.

k. At the completion of the test, obtain a sample at the point of penetration for water content determination. A sample should also be obtained about 4 to 6 inches away from the point of penetration for density determination.

TABLES

TABLE I

SOILS INVESTIGATED

LOCATION - CLASSIFICATION - COMPACTION - CBR - VARIABLES INVESTIGATED - LABORATORY CORRELATION

Sample	LOCATION: Vicinity from which Soil came	CLASSIFICATION: Visual Classification	Casagrande Classification	Specific Gravity	Liquid Limit	Plastic Limit	COMPACTION: Modified Proctor: Optimum Water Content	Modified Proctor: Maximum Dry Density	Modified Proctor: Soaked CBR at Optimum	Standard Proctor: Optimum Water Content	Standard Proctor: Maximum Dry Density	Standard Proctor: Soaked CBR at Optimum	Porter Static: Optimum Water Content	Porter Static: Maximum Dry Density	Porter Static: Soaked CBR at Optimum
*1	DeRidder, La.	Clay sand	SC	2.67	17	16	7.5	130.0	105	10.0	123.5	27	10.0	121.7	24
2	Mounds, La.	Clay (buckshot)	CH	2.75	88	28	23.0	100.5	2	29.0	88.0	2	--	--	--
*3	Vicksburg, Miss.	Loess	ML	2.74	26	24	14.0	110.5	18	18.0	103.5	7	18.0	107.5	11
4	DeRidder, La.	Clay sand (loam)	SF	2.68	18	15	11.4	115.5	50	14.5	110.0	24	15.0	105.0	4
*5	DeRidder, La.	Sand clay (loam)	SF	2.66	25	19	10.5	126.5	35	12.5	117.0	14	12.0	122.5	30
6	Barksdale Field, La.	Sand	SP	2.65	Nonplastic		9.0	121.3	28	11.0	117.5	25	--	--	--
7	Barksdale Field, La.	Clay gravel	GC	2.64	13	12	6.0	133.5	74	--	--	--	--	--	--
8	Natchitoches, La.	Sand clay	--	2.69	23	23	--	--	--	--	--	--	--	--	--
9	Natchitoches, La.	Clay gravel	--	2.66	26	17	--	--	--	--	--	--	--	--	--
*10	Eglin Field, Fla.	Sand	SP	2.63	Nonplastic		11.0	112.0	36	12.0	110.5	22	15.0	105.5	7
11	Jacksonville, Fla.	Sand	SP	2.64	Nonplastic		15.5	112.0	38	16.5	100.0	20	--	--	--
12	Michigan	Dune sand	SP	2.68	Nonplastic		13.0	107.6	30	13.5	104.5	16	--	--	--
13	Camp Skeel, Mich.	Sand	SP	2.67	Nonplastic		14.0	108.5	37	14.5	106.0	--	--	--	--
*14	Manchester, N.H.	Sand	SP	2.67	Nonplastic		11.5	114.0	53	12.0	110.0	24	12.0	110.5	28
15	Manchester, N.H.	Sand and gravel	GP	2.69	Nonplastic		5.0	121.0	8	--	--	--	--	--	--
16	Vicksburg, Miss.	Silty sand		No tests performed											
17	Lockbourne, Ohio	Clay		No tests performed											
*18	Berry Field, Tenn.	Clay	CL	2.74	32	21	15.0	113.5	14	18.0	107.0	6	13.0	121.0	30
19	Walesboro, Ind.	Gravel-sand-silt	GM	2.76	Nonplastic		6.0	137.0	75	--	--	--	--	--	--
*20	Stockton, Calif.	Clay (adobe)	CH	2.73	68	21	20.0	102.0	2	28.0	87.5	3	13.5	120.0	2
*21	California	Pumice	--	2.49	Nonplastic		28.0	75.0	91	31.0	69.5	35	17.0	79.0	21
22	Mission, Tex.	Caliche	--	2.72	31	17	16.5	107.0	80	20.0	100.5	20	--	--	--
23	Vicksburg, Miss.	Clay sand	SC	2.67	18	16	7.5	130.0	102	9.0	124.5	25	9.0	126.0	32
24	Kellogg Field, Ill.	Clay sand	SC	2.68	17	15	8.0	128.0	75	10.5	120.5	20	--	--	--
25	Georgetown, Tex.	Caliche	—	2.69	35	16	12.0	120.0	50	14.0	113.5	6	—	—	—
Mix	Marietta, Ga. **	Clay sand	SC	2.75	22	17	9.5	129.5	44	12.5	120.5	12	8.0	131.0	50
1	" "	Clay	CL	--	56	27	18.5	104.2	12	25.5	94.3	10	--	--	--
2	" "	Clay	CL	--	51	23	18.5	109.0	18	21.6	100.5	9	--	--	--
4	" "	Clay	CL	--	45	22	14.8	115.5	41	18.0	108.5	14	--	--	--
5	" "	Clay	CL	--	60	27	18.4	106.0	--	24.5	95.8	--	--	--	--
9	" "	Clay	CL	--	56	27	19.1	108.8	30	22.8	99.6	12	--	--	--
11-A	" "	Clay sand	SC	--	34	18	12.2	120.5	35	15.0	112.0	6	--	--	--
11-B	" "	Clay sand	SC	--	24	17	10.0	125.5	88	12.2	118.5	14	10.0	123.0	--
14	" "	Clay gravel	GC	--	47	24	5.0	133.0	46	7.0	122.0	12	--	--	--
21	" "	Clay	CH	--	73	27	19.5	104.5	3	26.0	90.6	2	--	--	--
1	Vicksburg, Miss.***	Clay silt	ML	2.70	41	26	16.0	106.5	30	20.0	100.0	8	21.5	97.5	6
2	" "	Gravelly clay sand	GP	2.67	20	13	7.0	132.0	60	9.3	125.0	12	10.0	123.0	8

Sample	VARIABLES INVESTIGATED: Preparation of Test Specimen: Density Gradient	Water Content Gradient	*Breakdown of Particles	Density	Water Content	Soil Structure	Method of Compaction	Height of Specimen	Method & period of soaking	Soaking Surcharge	Drainage Time	Processing Soil	Penetration (CBR) Test: Mold & Piston Diameter	Rate of Penetration	Penetration Surcharge	Shape of Particles	Correction to Penetr. Curve	CORRELATION: CBR, Triaxial and Unconfined Compression
*1	x			x	x	x	x		x						x		x	x
2				x	x	x											x	
*3	x			x	x	x	x	x	x	x	x		x	x	x		x	x
4				x	x	x	x										x	
*5				x	x	x	x								x		x	x
6				x	x	x									x		x	
7				x	x	x						x	x				x	
8																		
9																		
*10	x		x	x	x	x	x	x					x		x	x	x	x
11				x	x	x									x		x	
12				x	x	x									x		x	
13				x	x	x									x		x	
*14				x	x	x	x				x			x	x	x	x	x
15				x	x	x						x	x				x	
16																		
17																		
*18	x	x		x	x	x	x	x	x	x	x				x		x	x
19				x	x	x						x	x				x	
*20				x	x	x	x		x	x			x	x	x		x	x
*21			x	x	x	x											x	
22				x	x	x											x	
23	x			x	x	x	x										x	x
24				x	x	x											x	
25				x	x	x											x	
Mix				x	x	x	x										x	
1				x	x	x											x	
2				x	x	x											x	
4				x	x	x											x	
5				x	x	x											x	
9				x	x	x											x	
11-A				x	x	x											x	
11-B				x	x	x	x										x	
14				x	x	x											x	
21				x	x	x											x	
1				x	x	x											x	
2				x	x	x											x	

* Bracketing soils of CBR investigation.
** Samples tested in connection with the construction of a pavement behavior test section, Marietta, Ga.
*** Samples tested in connection with the construction of airplane landing mat test sections, Vicksburg, Miss.

TABLE 2

EFFECT OF MOLD AND PISTON DIAMETER ON NATURAL AND PROCESSED BASE MATERIAL

		CBR at Modified Proctor Optimum in 6- and 12-Inch Diameter Molds				CBR at Dry and Wet of Modified Proctor Optimum at 99% Max. Dry Density in 6-Inch Diameter Mold			
		Natural Material		Processed Material		Natural Material		Processed Material	
Sample	Type Material and Location	6-Inch Mold	12-Inch Mold*	6-Inch Mold	12-Inch Mold*	Dry of Optimum	Wet of Optimum	Dry of Optimum	Wet of Optimum
7	Clay-gravel Barksdale, La.	74	37 (15)	58	22	52 [42]	46 [26]	52 [34]	32 [20]
15	Sand-gravel Manchester, N.H.	6**	4 (1)	9**	3	5**	7**	5**	7**
19	Gravel-sand-silt Walesboro, Ind.	73	24 (19)	142	38	80	25	160	45

*Average of two tests
**Estimated

Notes:
Surcharge 10-10 in 6-inch mold.
Surcharge 40-40 in 12-inch mold.
Figures in parentheses obtained using 2-inch piston.
Figures in brackets are for 98% maximum dry density.
Two-inch piston used in tests in 6-inch mold.
Four-inch piston used in tests in 12-inch mold except where noted.

TABLE 3

COMPARISON OF STATIC VS DYNAMIC COMPACTION AND CBR

Sample	Material	Surcharge		PORTER STATIC						MODIFIED PROCTOR						STANDARD PROCTOR						15-BLOW PROCTOR					
		Soaking	Penetration	Maximum Dry Density Lb/Cu Ft	Optimum Water Content Percent	Test Dry Density Lb/Cu Ft	Test Water Content Percent	Soaked CBR Percent Uncorrected	Soaked CBR Percent Corrected	Maximum Dry Density Lb/Cu Ft	Optimum Water Content Percent	Test Dry Density Lb/Cu Ft	Test Water Content Percent	Soaked CBR Percent Uncorrected	Soaked CBR Percent Corrected	Maximum Dry Density Lb/Cu Ft	Optimum Water Content Percent	Test Dry Density Lb/Cu Ft	Test Water Content Percent	Soaked CBR Percent Uncorrected	Soaked CBR Percent Corrected	Maximum Dry Density Lb/Cu Ft	Optimum Water Content Percent	Test Dry Density Lb/Cu Ft	Test Water Content Percent	Soaked CBR Percent Uncorrected	Soaked CBR Percent Corrected
1	DeRidder clay sand	10	10	121.8	10.0	121.8	10.0	24	24	132.2	7.9	129.1	7.8	77	97	125.5	9.8	122.1	9.8	20	23	123.5	10.0	120.5	10.2	11	16
												128.7	7.8	103	104			124.2	9.8	19	26			119.3	10.2	12	15
3	Vicksburg loess	10	0	106.7	8.0	106.7	8.0	10	11	111.0	14.5	111.0	15.5	22	51	105.2	17.4	104.6	17.7	13	16	102.5	17.6	102.4	19.4	11	11
						106.7	8.0	7	7			112.0	15.3	39	52			104.0	17.7	15	15			102.6	19.1	12	12
												[a]111.8	15.5	1	18			[a]103.6	18.0	1	5						
												[a]111.8	15.8	2	16			[a]104.8	18.0	1	8						
5	DeRidder sand clay (loam)	10	0	122.6	12.1	122.6	12.1	32	32	127.5	10.5	127.1	10.5	16	32	119.0	12.5	116.0	12.5	12	12	117.0	13.0	113.8	13.5	5	5
												125.6	10.5	17	34			117.4	12.5	14	14			114.9	13.5	6	6
10	Eglin Field sand	10	10	107.2	9.7	107.2	9.7	9	9	112.5	12.0	112.7	11.0	27	32	109.7	12.2	110.2	12.3	25	26	108.5	12.8	109.5	13.1	14	16
												112.0	11.2	26	36			110.2	12.8	19	20			110.1	12.7	10	15
14	Manchester sand	10	10	110.4	11.9	110.4	11.9	27	28	113.5	12.0	114.1	11.2	21	51	110.0	12.8	109.7	13.1	19	23	108.7	13.0	109.6	12.7	11	14
						110.0	12.0	18	19			114.3	11.2	34	55			110.1	12.8	20	24			109.3	13.1	15	17
18	Berry Field lean clay	10	0	[b]120.8	13.0	120.9	13.0	62	62	113.4	15.0	115.0	14.6	12	14	107.5	17.5	---	--	--	--	105.0	19.0	---	--	--	--
						120.6	13.0	63	63			115.2	14.6	17	19												
										[b]118.5	12.5	119.0	12.3	53	53	[c]110.0	15.0	112.8	15.3	10	10	[c]108.0	17.0	107.8	17.4	5	5
												118.9	12.5	59	59			112.9	15.2	12	12			108.0	17.3	5	5
20	Stockton adobe clay	10	0	119.5	13.5	119.5	13.5	100	100	101.5	20.5	101.4	19.5	40	40	87.5	27.5	87.4	28.0	11	11	83.0	31.5	---	--	--	--
												101.4	21.5	40	40												
21	California pumice	10	10	79.0	15 to 20	---	--	--	--	75.2	15 to 30	75.6	29.2	66	90	69.0	32.0	68.6	30.9	26	31	68.0	34.0	67.2	35.9	9	20
												75.2	29.9	49	58			68.4	31.2	40	40			66.1	36.2	8	21

Note: Unless otherwise indicated, all specimens were soaked from bottom only.

a = Specimens soaked from top and bottom
b = Second time material used
c = Third time material used

TABLE 4

COMPARISON OF CBR OF SOAKED SPECIMENS MOLDED AT OPTIMUM WATER CONTENT WITH CBR OF AS-MOLDED SPECIMENS MOLDED AT WATER CONTENT OF TOP INCH OF SOAKED SPECIMENS

	Tested After Soaking Actual Test Specimens						Tested As-Molded Data From Curves		
		Water Content after Soaking-%							
Compactive Effort	Molding Water Content %	Top Inch	Average	Dry Density as Molded Lb/Cu Ft	Swell %	Corrected CBR %	Molding Water Content %	Dry Density as Molded Lb/Cu Ft	Corrected CBR %
			Sample 23 - Vicksburg Clay-Sand						
M.P.	7.5	8.8	8.8	129.7	0.0	104	8.8	128.0	29
S.P.	9.0	12.0	10.2	124.3	0.0	24	12.0	119.2	2
15-blow	10.1	12.4	11.6	122.9	0.0	16	12.4	118.8	2
			Clay-Silt Subgrade - Vicksburg Landing Mat Investigation						
M.P.	15.8	19.5	18.3	108.1	1.0	38	19.5	102.4	8
S.P.	20.0	26.5	23.1	99.1	0.7	10	26.5	92.5	1
15-blow	21.3	29.7	25.9	95.0	1.2	6	29.7	90.0	0
			*Sample 3 - Vicksburg Loess						
M.P.	15.5	17.7	17.8	111.0	0.1	51	17.7	106.6	11
S.P.	17.7	22.0	20.6	104.6	0.3	16	22.0	99.2	1
15-blow	19.1	22.0	21.3	102.6	0.1	12	22.0	99.0	2
			Sand-Clay Base Course - Vicksburg Landing Mat Investigation						
M.P.	7.4	9.0	9.3	131.4	0.2	30	9.0	127.8	56
S.P.	9.0	11.5	10.5	124.9	0.1	12	11.5	121.8	5
15-blow	9.3	12.2	11.3	122.8	0.2	9	12.2	119.5	2
			Sample 9 - Clay Subgrade - Marietta, Ga. Test Section						
M.P.	19.9	23.0	21.5	108.7	0.5	30	23.0	104.0	9
S.P.	23.4	25.7	25.4	99.5	0.2	13	25.7	97.8	5
15-blow	25.8	28.4	27.6	96.4	0.1	6	28.4	92.6	3
			**Sample 14 - Clay-Gravel, Marietta, Ga. Test Section						
M.P.	5.0	--	6.2	133.4	0.1	46	6.2	130.7	30
S.P.	7.0	--	9.1	122.2	0.1	12	9.1	120.5	3
15-blow	8.0	--	9.9	121.3	0.0	10	9.9	119.0	2

*Soaked from bottom only. All other samples soaked from top and bottom.

**As-molded data. Specimens molded at average water content obtained after soaking.

TABLE 5

COMPARISON OF THE SOAKED CBR OF MATERIAL MOLDED ON DRY SIDE WITH CBR MOLDED ON WET SIDE AND CURED BEFORE SOAKING

Specimens Soaked Immediately After Molding
(Surcharge 10-0)

Material	Molded Average w	Molded Average γ_d	Soaked Average w	Soaked Average γ_d	Swell %	CBR at 0.1 Inch in Percent Uncorrected	CBR at 0.1 Inch in Percent Corrected	
Gravelly clay-sand	6.2	129.2	9.7	129.1	0.7	29	33	(Dry side)
	11.1	124.3	12.3	124.3	-0.2	3	3	(Wet side)
Clay-sand	6.6	123.3	12.5	123.3	0.4	99	99	(Dry side)
	11.6	125.1	12.4	125.1	0.0	5	9	(Wet side)

Specimens Dried Back (Cured) Before Soaking*
(Surcharge 10-0)

Material	Molded Average w	Molded Average γ_d	Dried Back Average w	Dried Back Average γ_d	Soaked Average w	Soaked Average γ_d	Swell %	CBR at 0.1 Inch in Percent Uncorrected	CBR at 0.1 Inch in Percent Corrected
Gravelly clay-sand	10.6	123.7	8.7	126.0	10.7	126.0	0.1	6	7
Clay-sand	11.6	124.5	9.3	127.0	11.4	127.0	0.3	9	16

*Molded on wet side and dried back to approximately modified Proctor optimum.

Note: Modified Proctor compactive effort used.

The gravelly clay-sand is Sample 2 of the Landing Mat Tests at Vicksburg, Mississippi.

The clay-sand is Sample 5 of the CBR investigation.

TABLE 6

COMPARISON OF WATER CONTENT DISTRIBUTION, SWELL, AND CBR OF SPECIMENS SOAKED FROM BOTTOM ONLY WITH SPECIMENS SOAKED TOP AND BOTTOM FOR PERIODS SHOWN

Method of Soaking	Days Soaked	Molded w - %	Molded γ_d	Water Content After Soaking - % Average	Water Content After Soaking - % Top	Water Content After Soaking - % Bottom	Swell %	CBR at 0.1 Inch in Percent Uncorrected	CBR at 0.1 Inch in Percent Corrected
Modified Proctor Compaction - Berry Field Lean Clay									
Bottom only	13	8.6	113.7	17.4	19.4	15.6	1.2	16	16 (Dry)
Top & bottom	4	7.9	113.0	16.0	18.5	16.6	1.5	11	11 (Dry)
Bottom only	13	12.5	118.9	14.6	16.3	15.3	0.3	59	59 (Opt.)
Top & bottom	4	12.6	120.5	14.1	15.2	13.5	0.8	8	14 (Opt.)
Bottom only	13	15.4	113.7	16.0	16.0	16.2	0.1	4	6 (Wet)
Top & bottom	4	15.9	113.0	17.1	15.9	16.5	0.1	2	3 (Wet)
Standard Proctor Compaction - California Adobe Clay									
Bottom only	29	24.2	84.7	32.2	34.8	31.2	2.6	4	4 (Dry)
Top & bottom	4	19.2	81.6	39.9	45.7	36.4	8.2	1	1 (Dry)
Bottom only	25	27.8	88.8	28.8	27.4	28.8	0.4	14	14 (Opt.)
Top & bottom	4	28.0	90.9	30.4	34.4	29.1	1.9	3	3 (Opt.)
Bottom only	27	32.2	85.8	33.6	37.9	34.5	0.4	6	6 (Wet)
Top & bottom	4	34.3	83.8	36.1	36.4	35.1	1.2	2	2 (Wet)

PLATES

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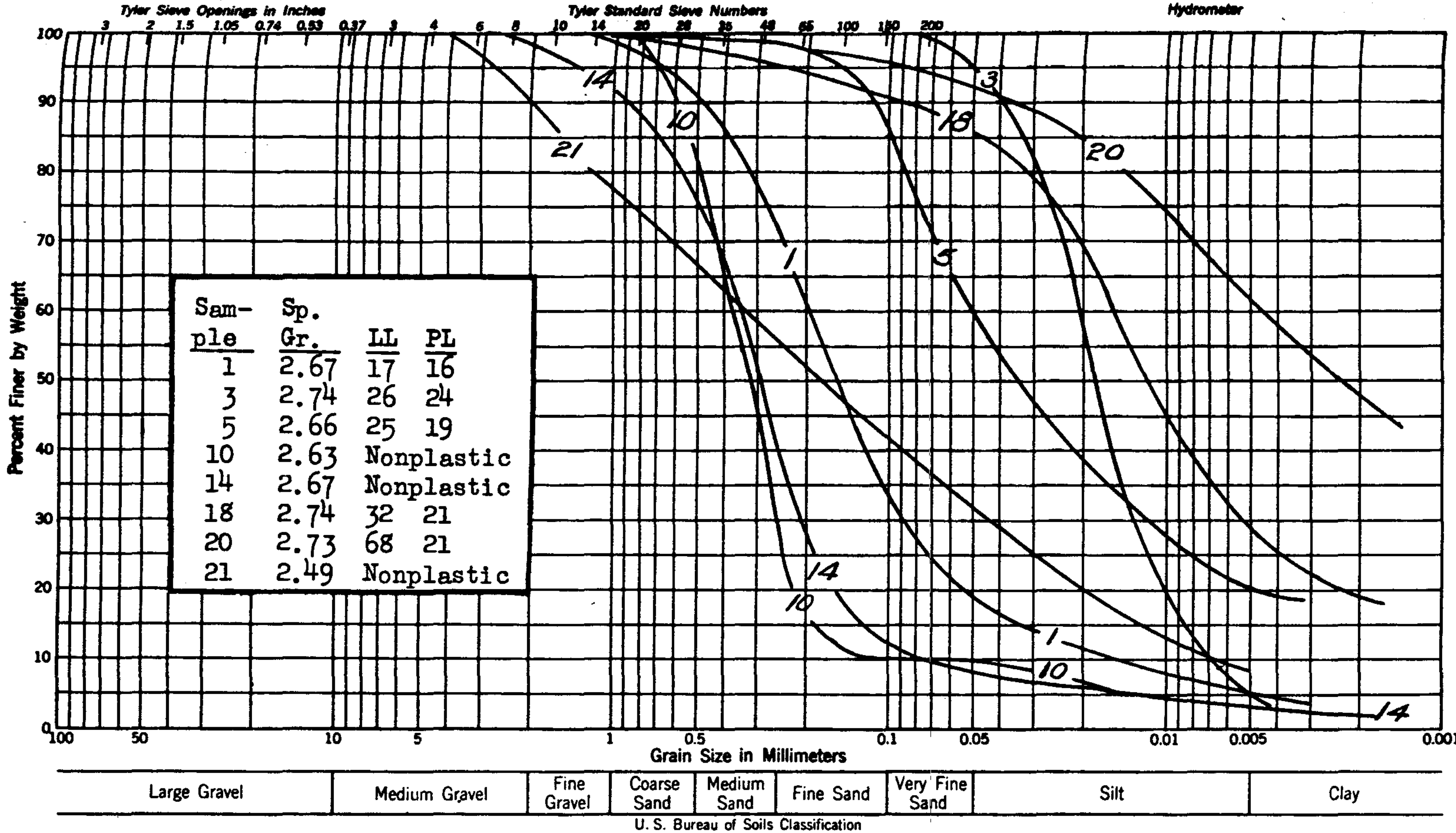

Sample	Sp. Gr.	LL	PL
1	2.67	17	16
3	2.74	26	24
5	2.66	25	19
10	2.63	Nonplastic	
14	2.67	Nonplastic	
18	2.74	32	21
20	2.73	68	21
21	2.49	Nonplastic	

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CLASSIFICATION DATA
(Bracketing Soils)

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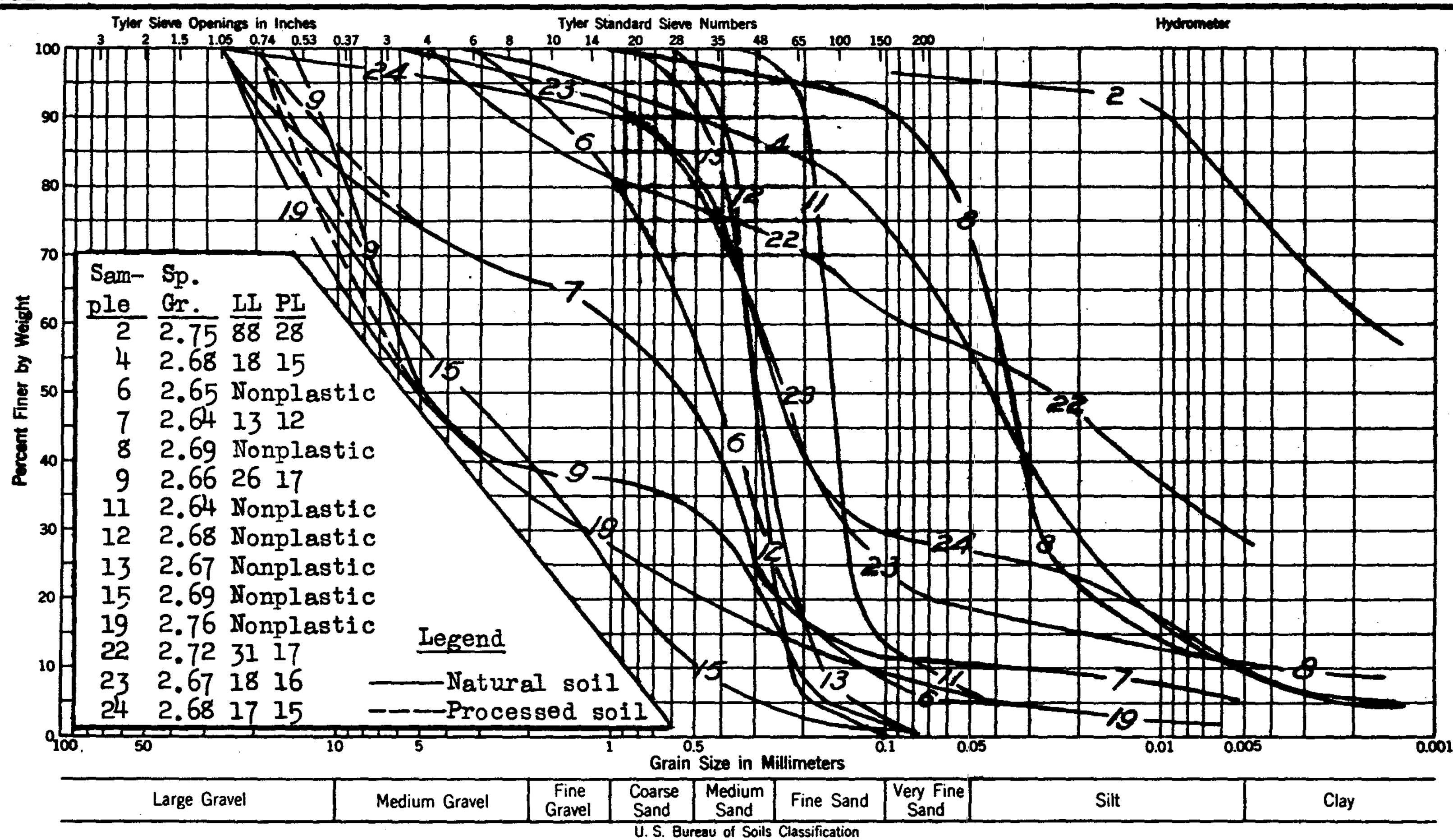

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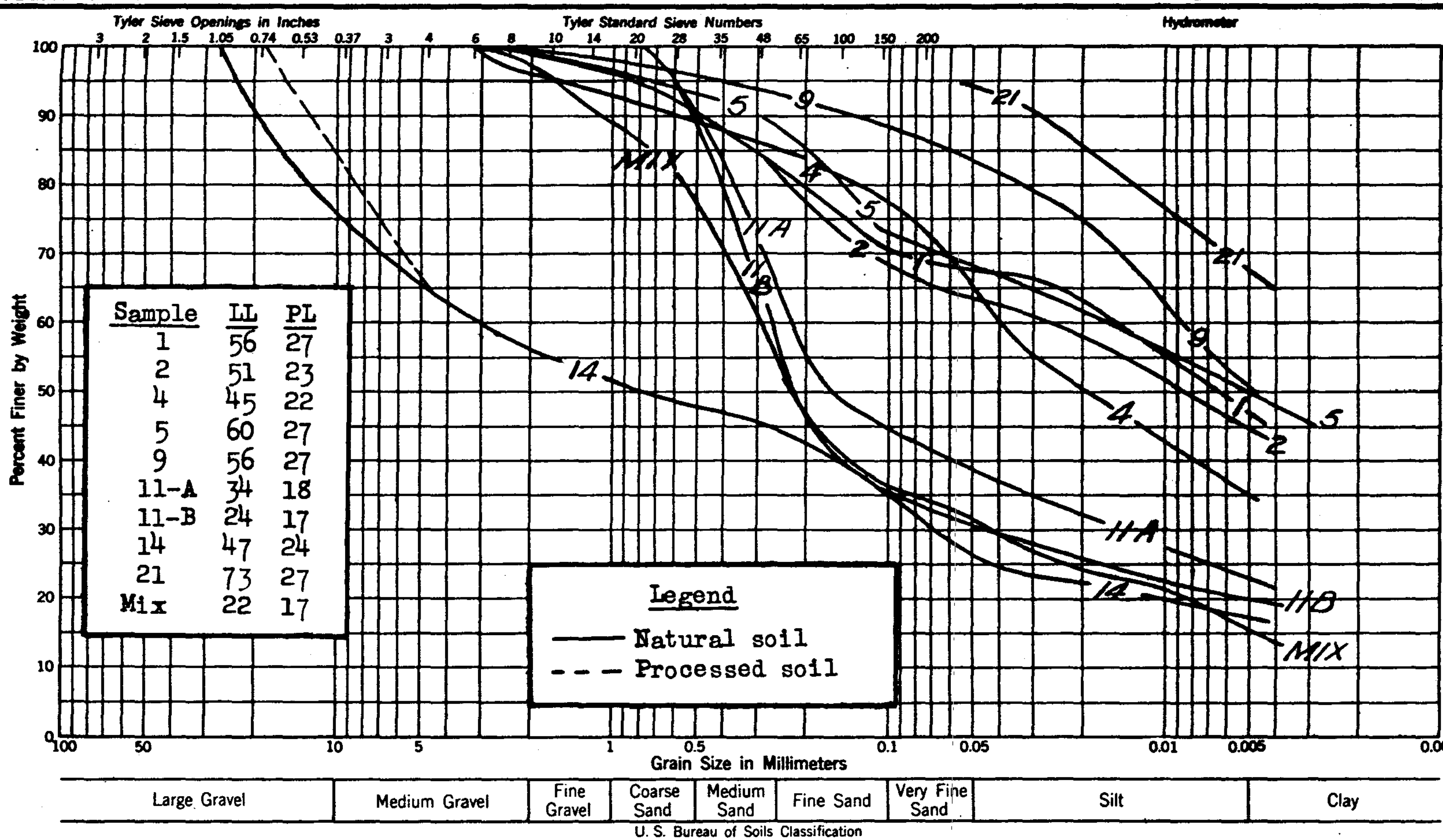
CLASSIFICATION DATA

All Other than Bracketing Soils



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California Bearing Ratio Investigation

CLASSIFICATION DATA

Samples tested in Connection with the Construction of a Pavement Behavior Test Section, Marietta, Ga.

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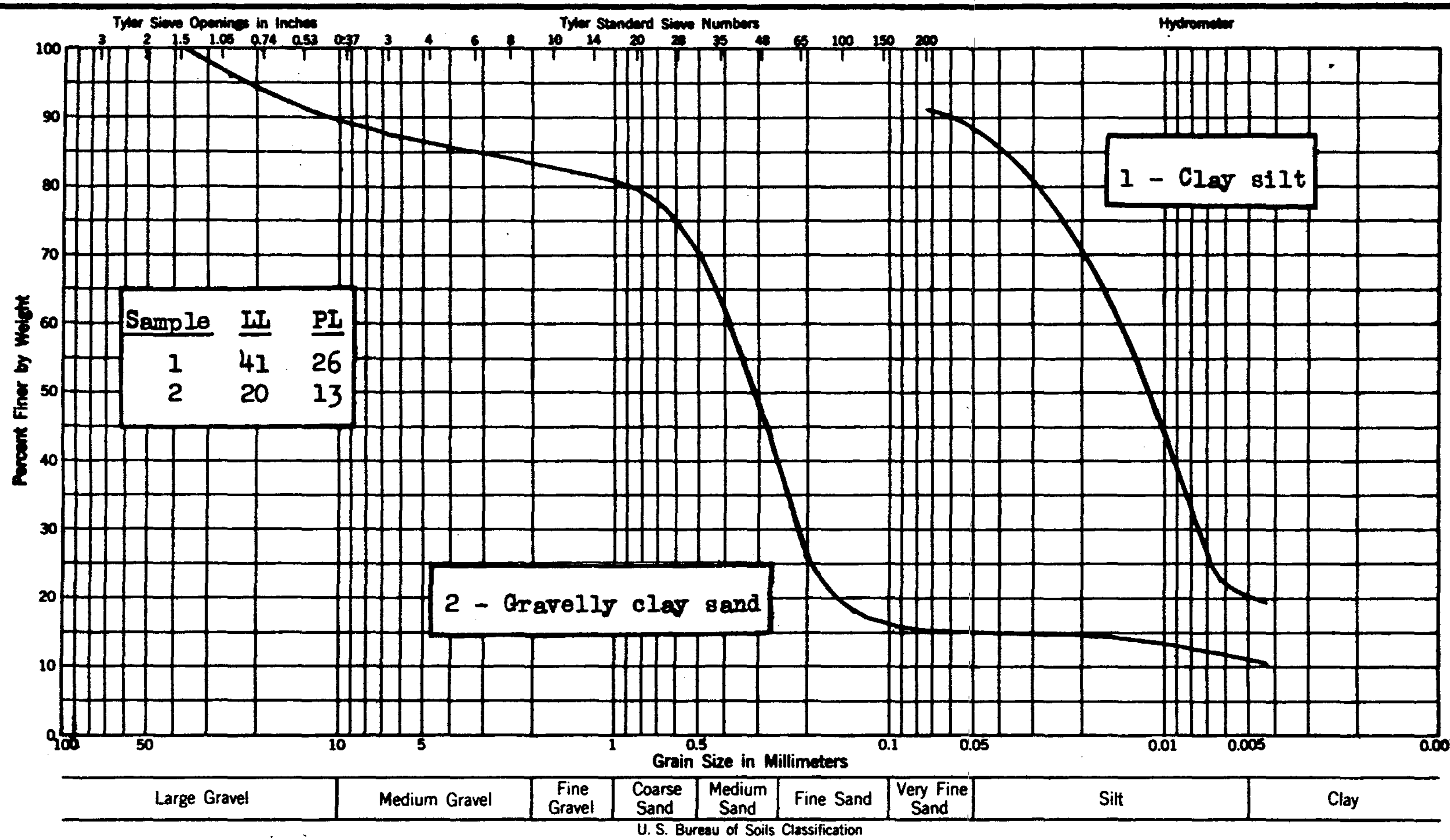

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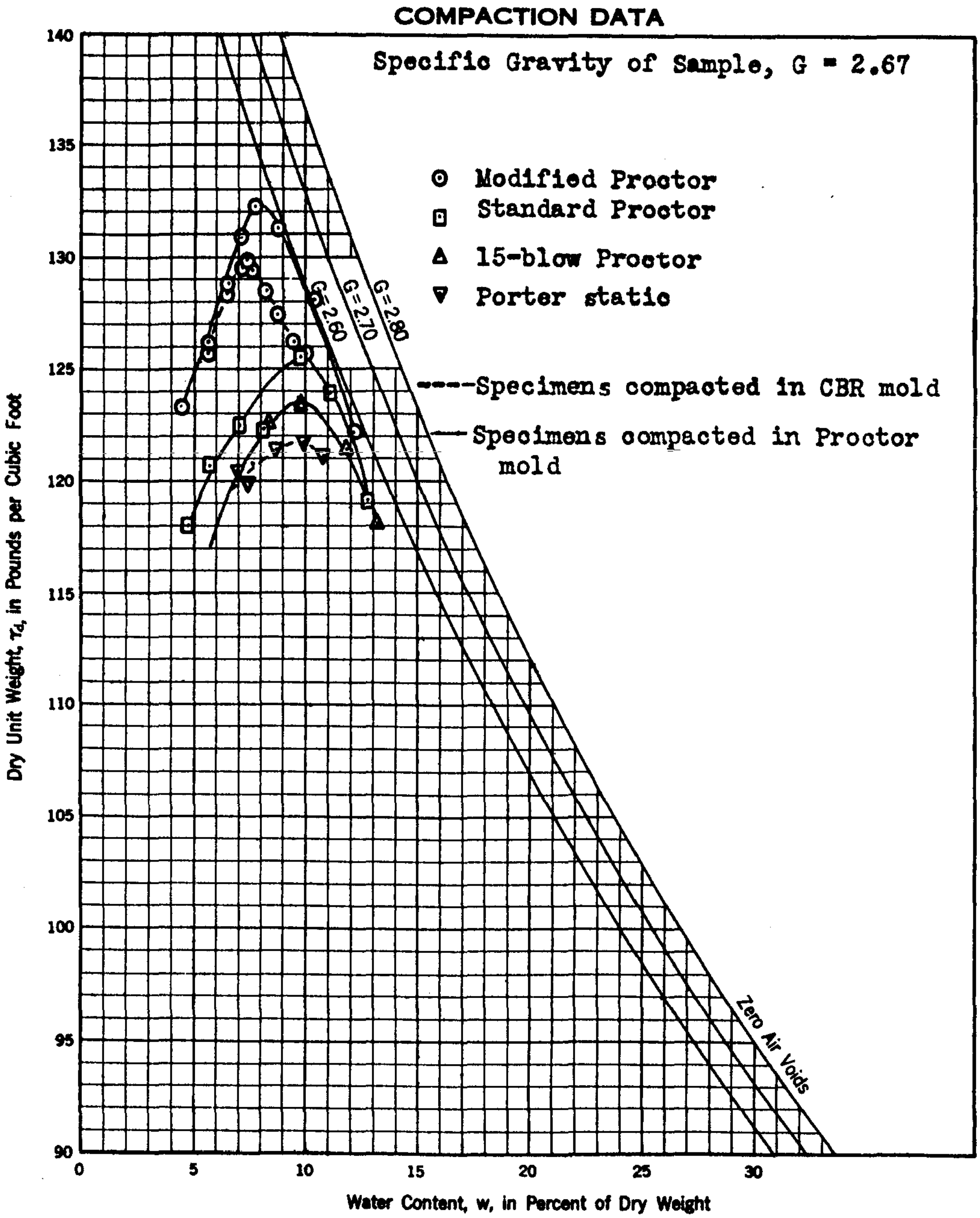
CLASSIFICATION DATA

Samples tested in Connection with the Construction of Airplane Landing Mat Test Sections, Vicksburg, Miss



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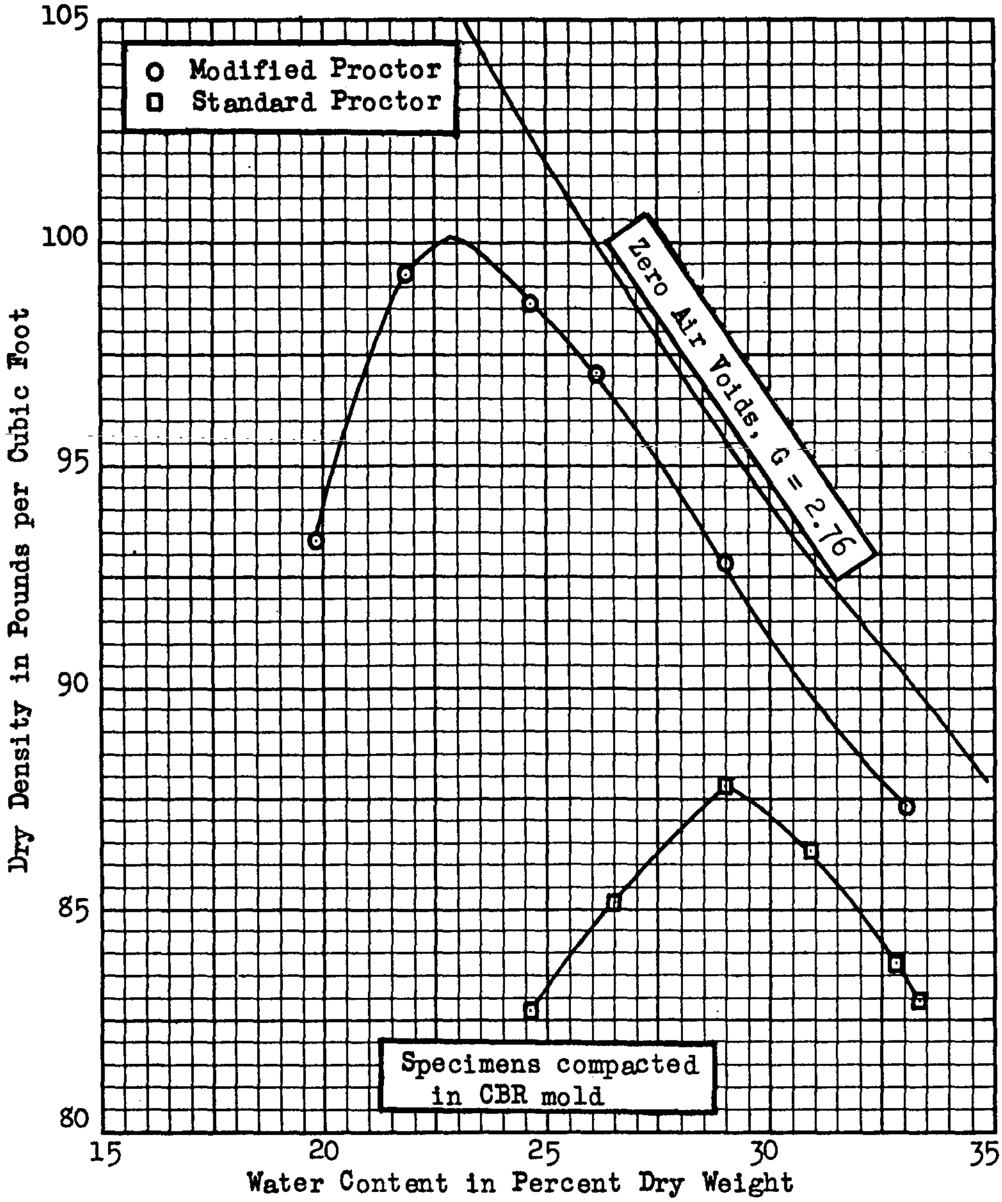
COMPACTION DATA

Sample 1 DeRidder Clay Sand



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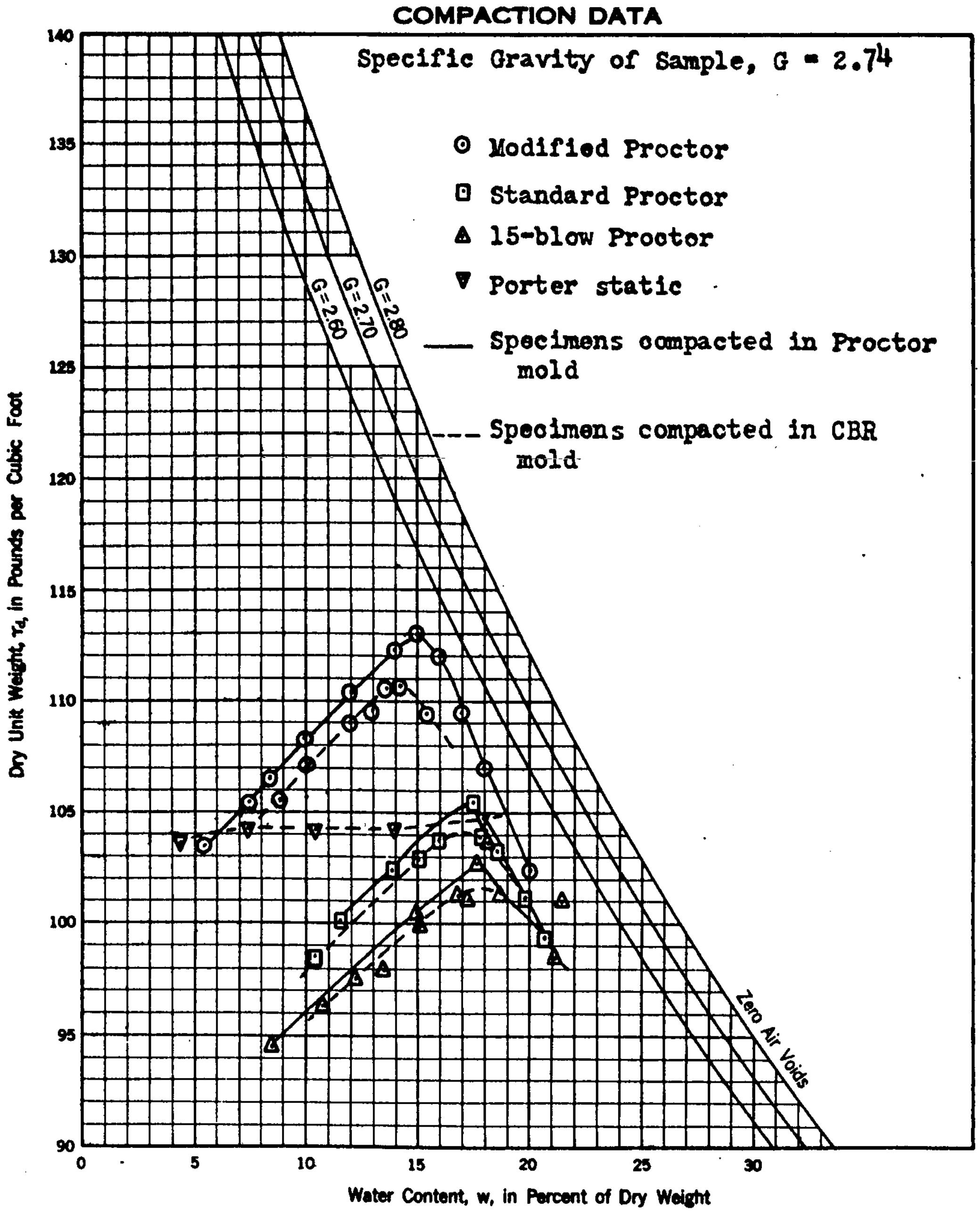
COMPACTION DATA

Sample 2 Louisiana Buckshot



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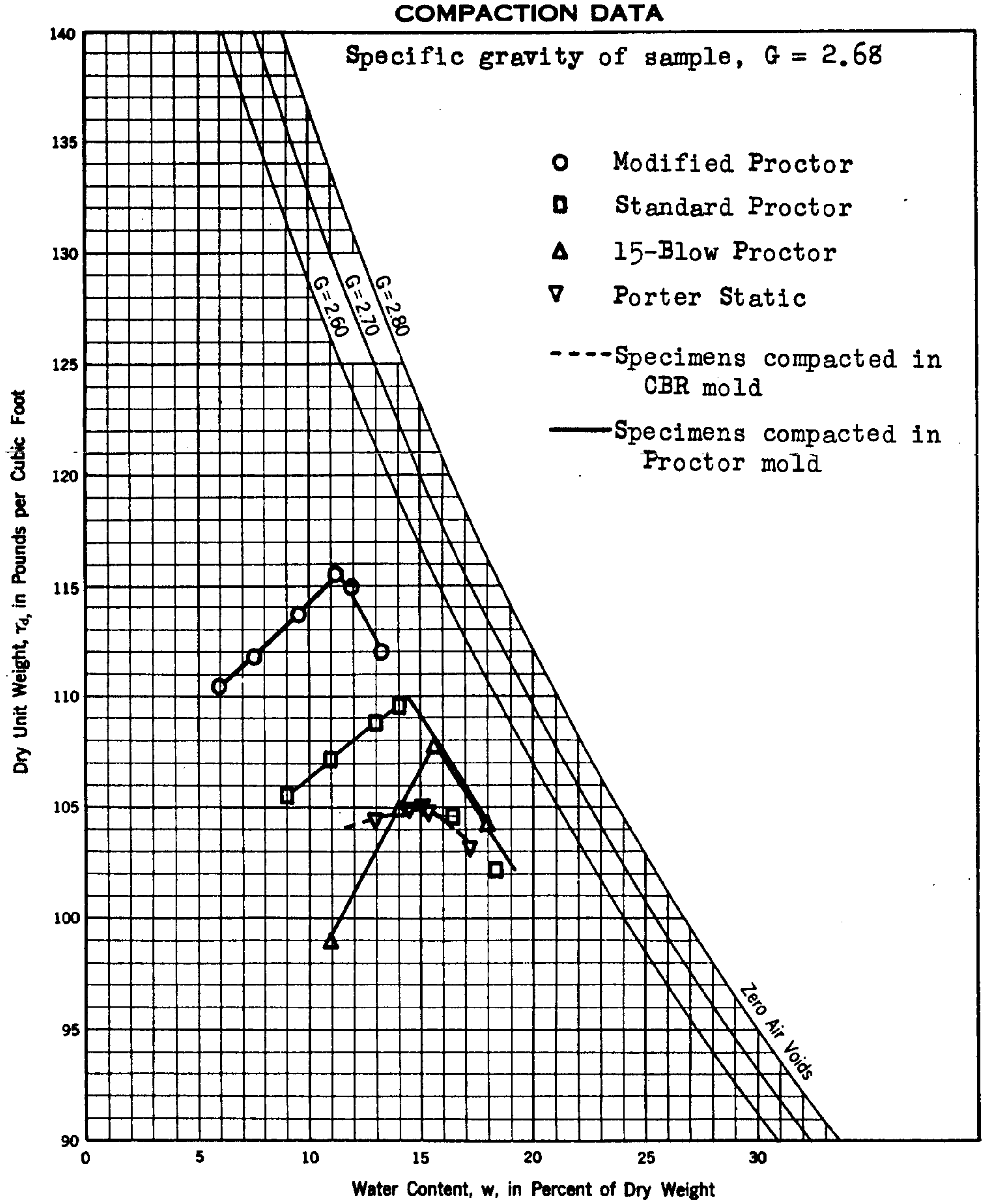
COMPACTION DATA

Sample 3 Vicksburg Loess



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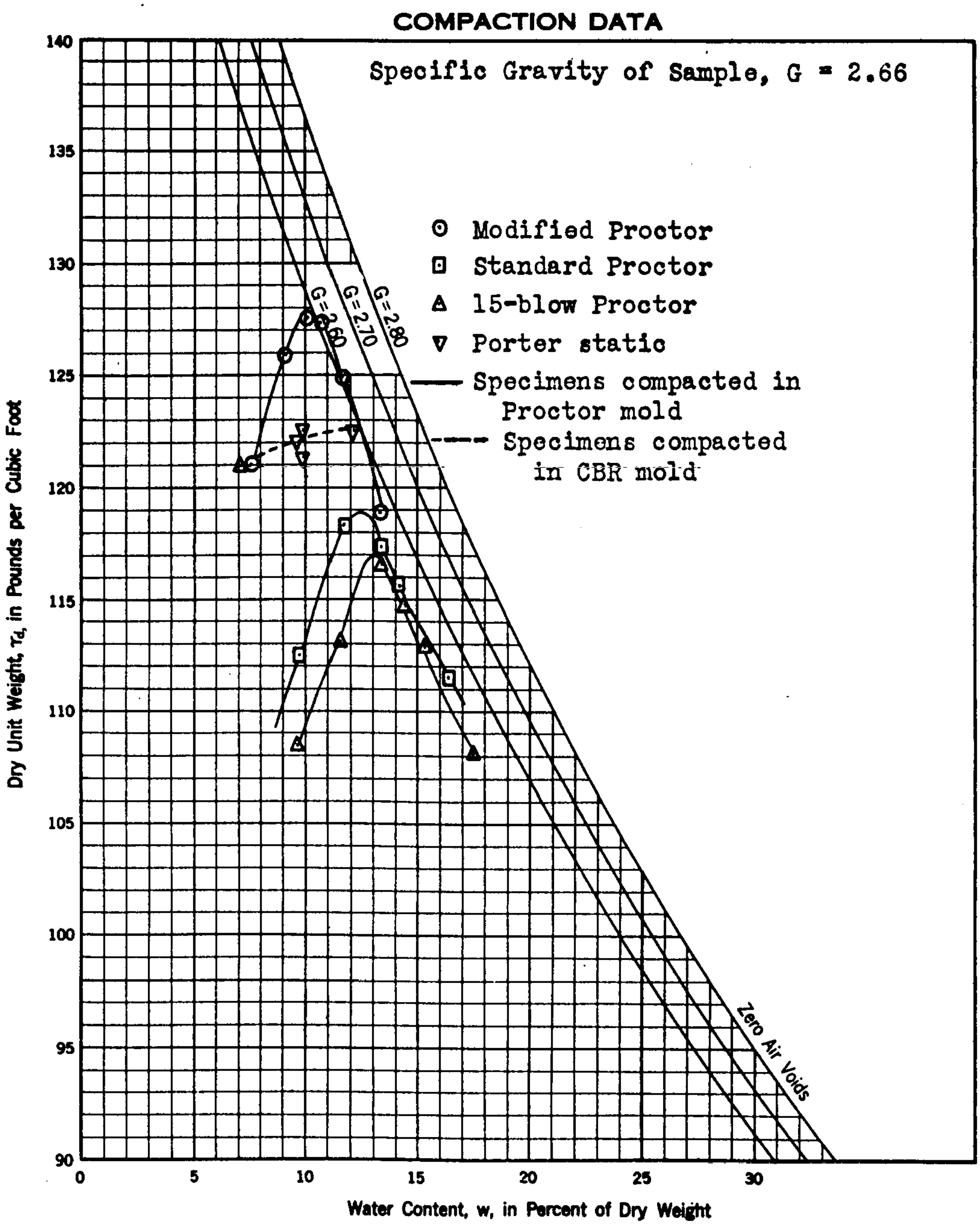
COMPACTION DATA

Sample 4 DeRidder Sandy Loam



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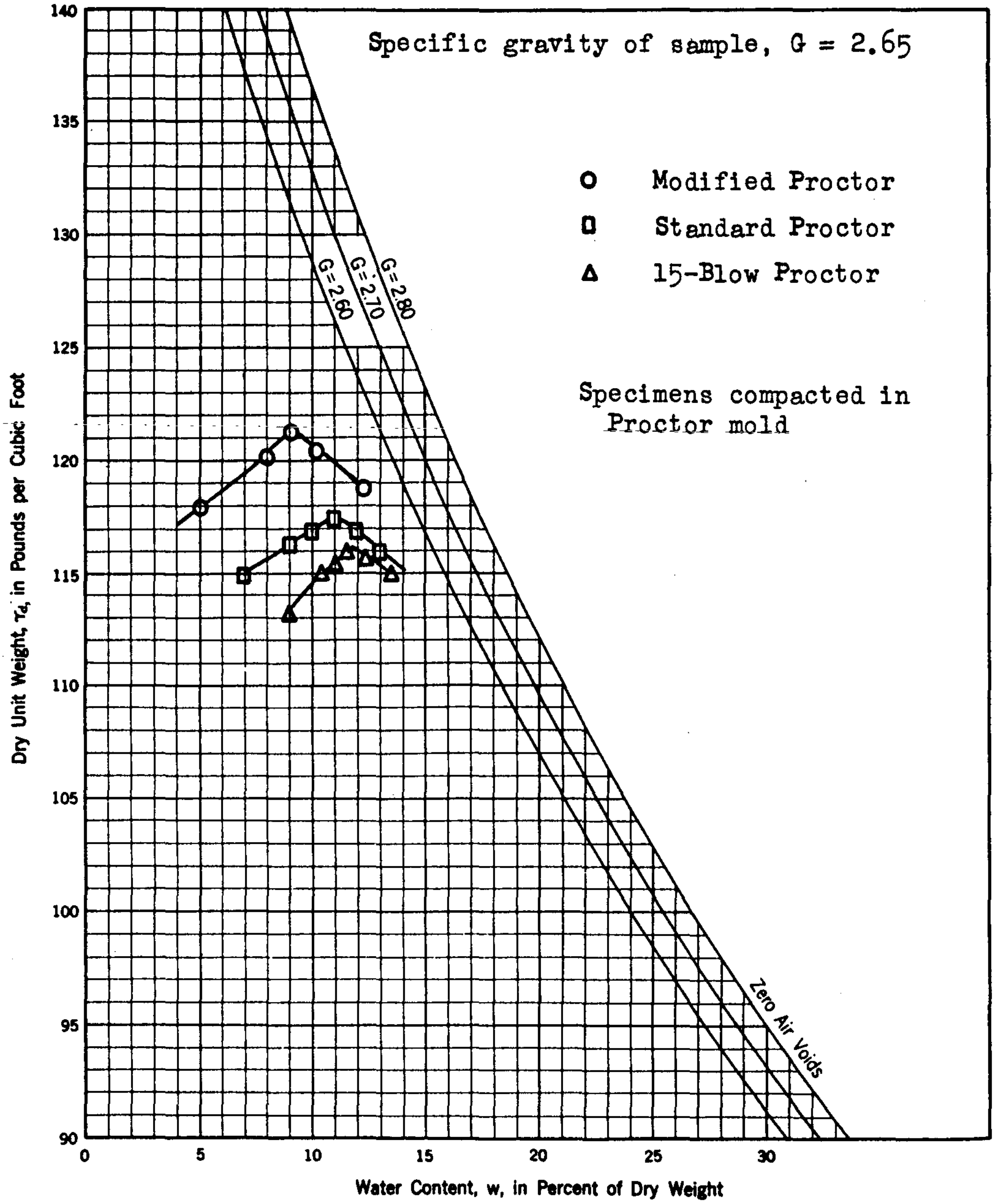
COMPACTION DATA

Sample 5 DeRidder Sand Clay (Loam)

COMPACTION DATA



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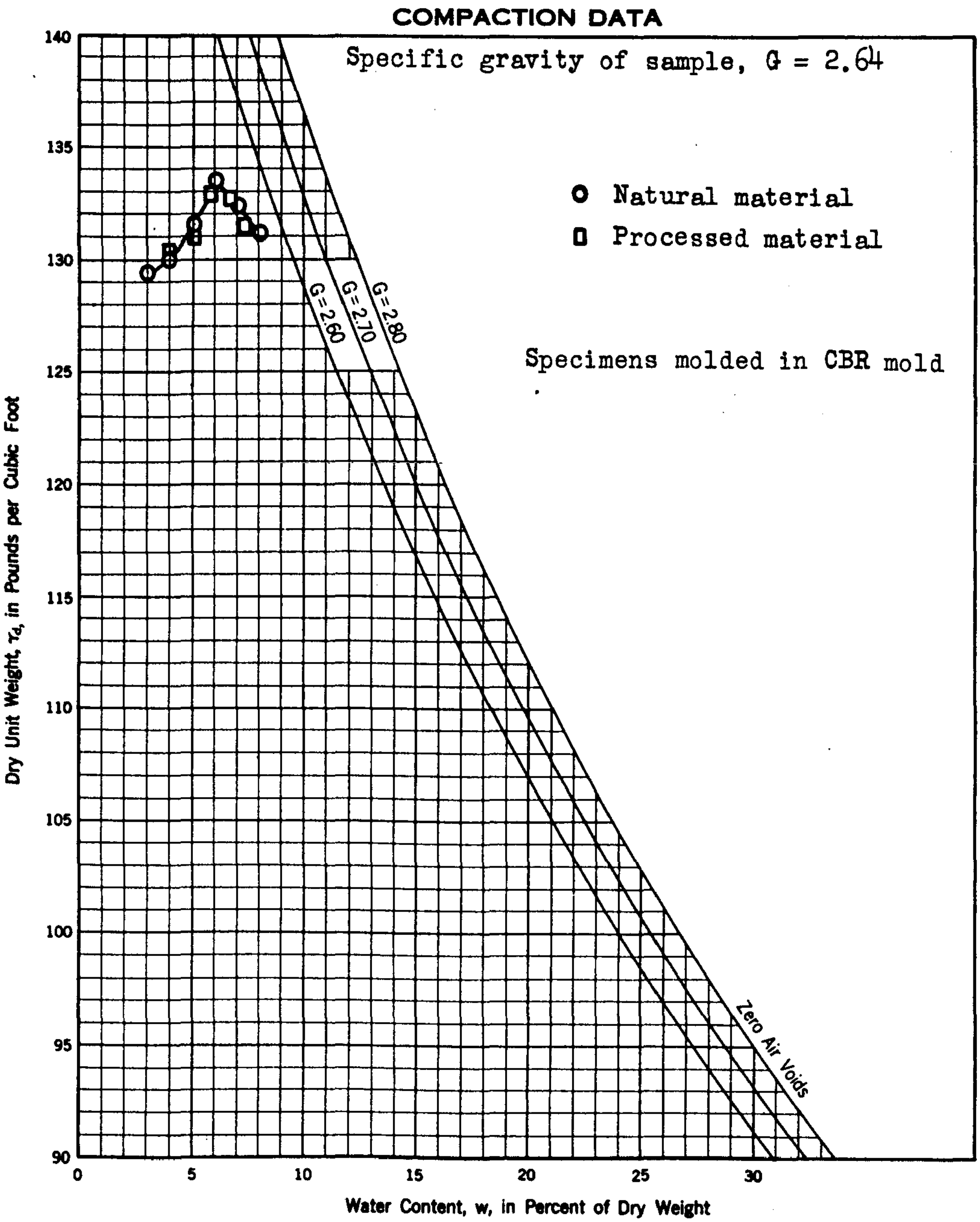
COMPACTION DATA

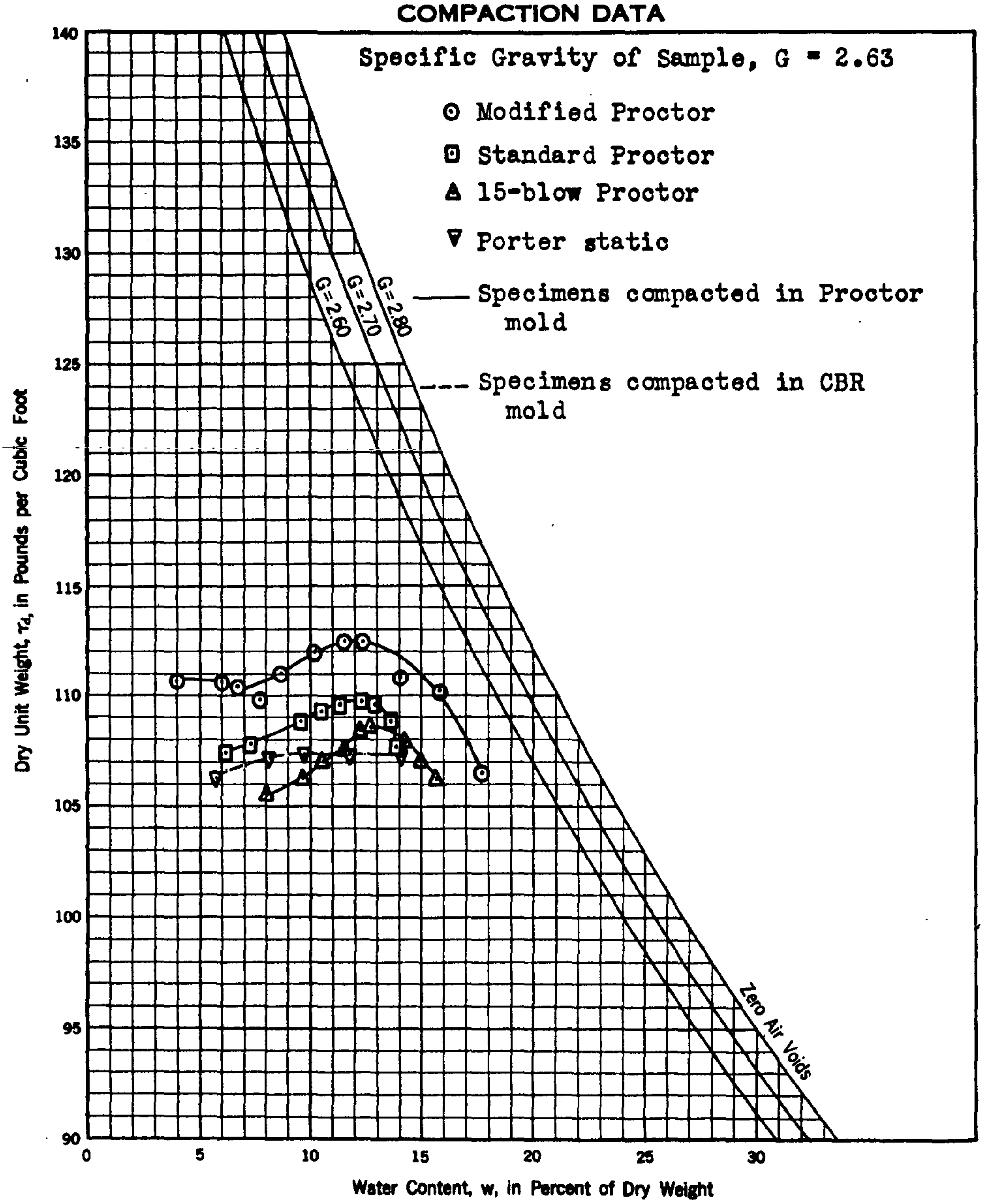
Sample 6 Minden Sand



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MODIFIED PROCTOR COMPACTION

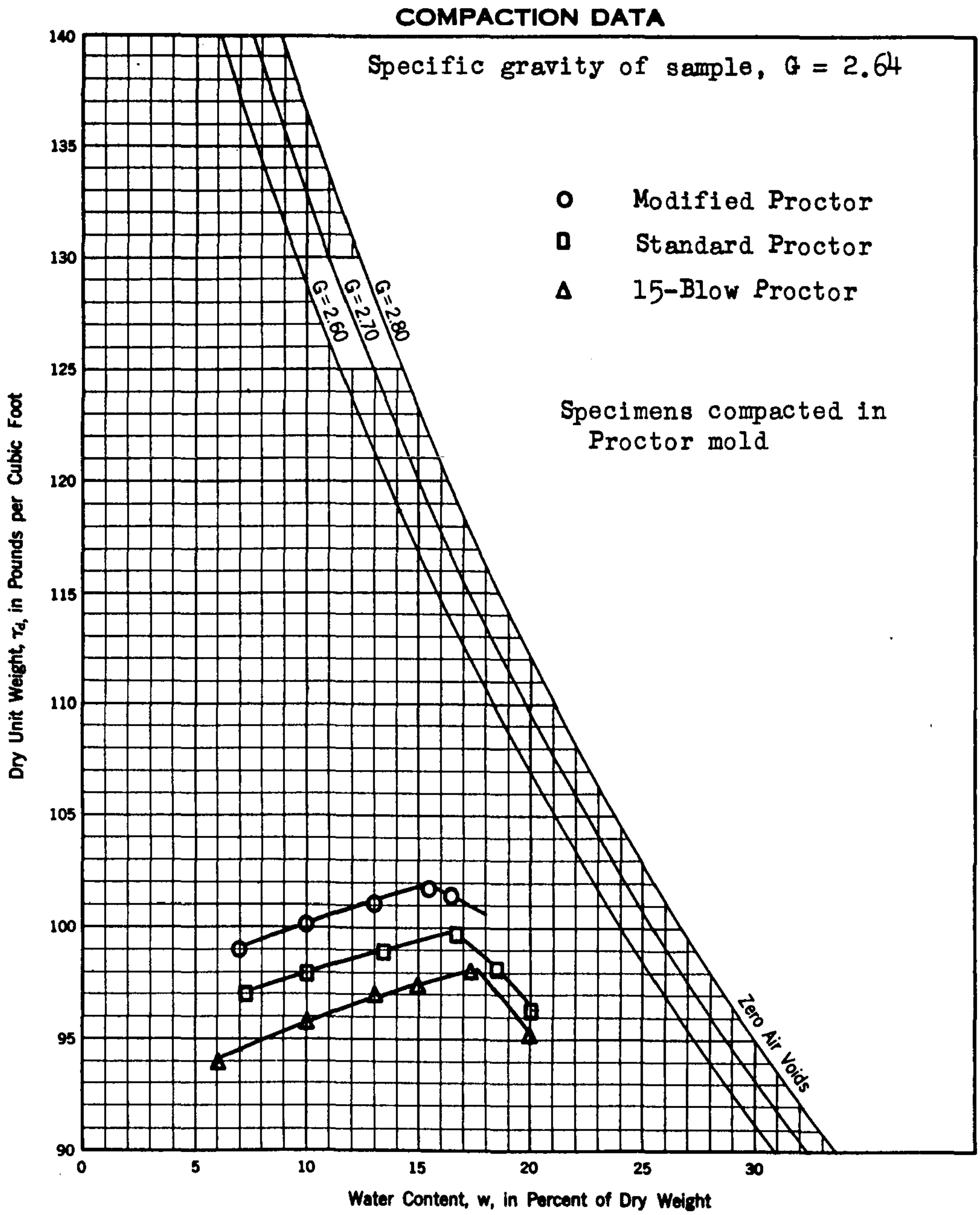
Sample 7 Barksdale Field Clay Gravel



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

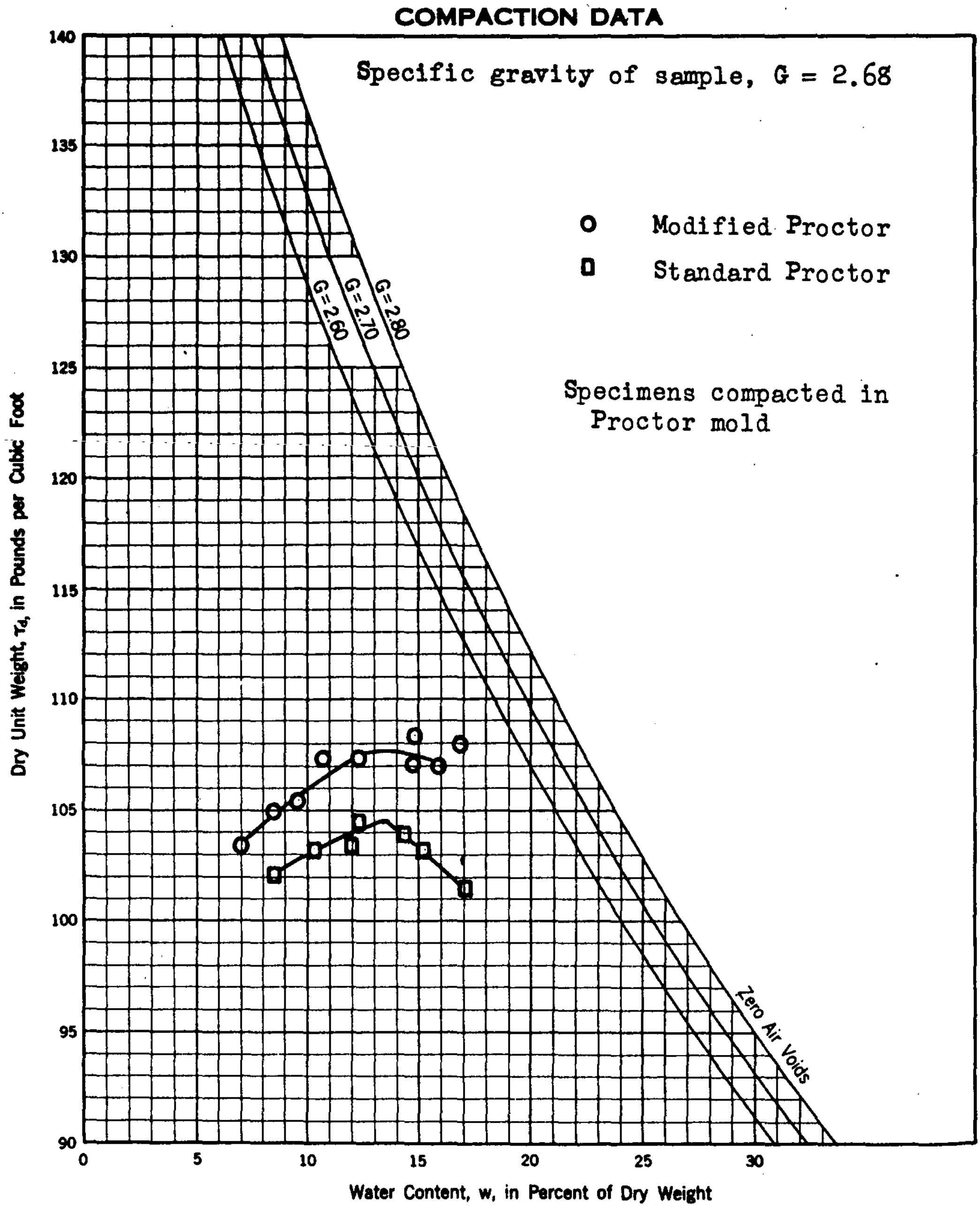
Sample 10 Eglin Sand



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

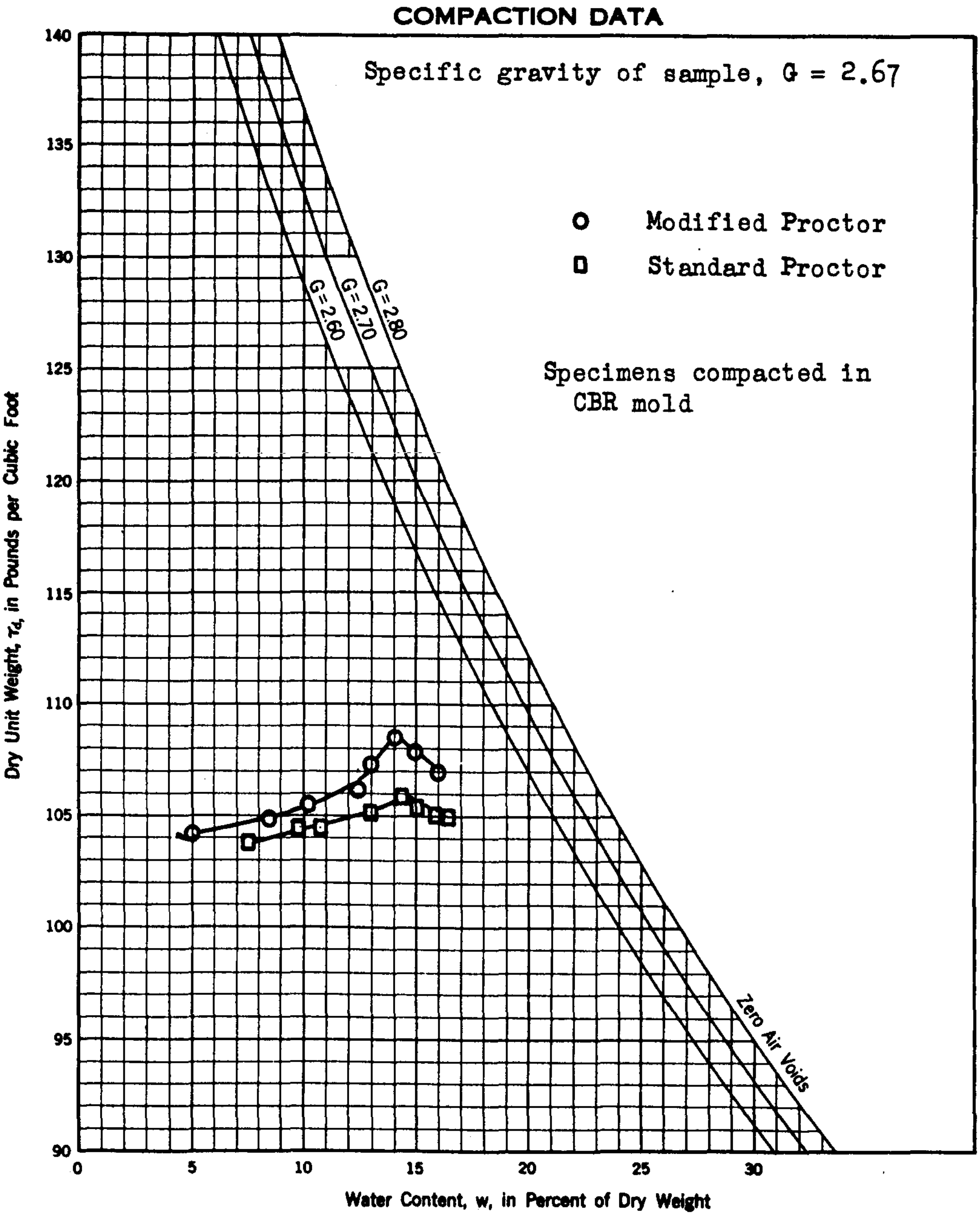
Sample 11 Florida Sand



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

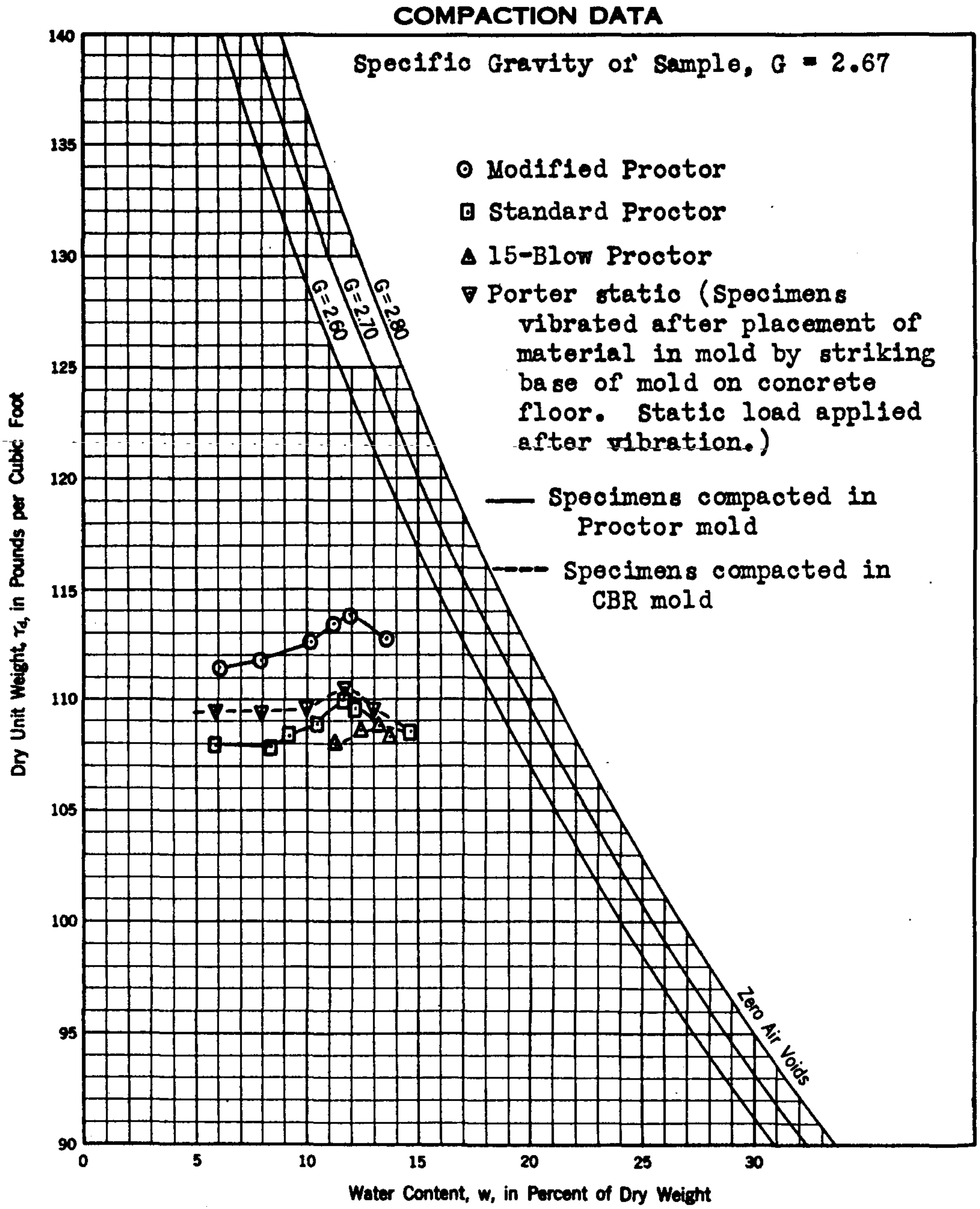
Sample 12 Michigan Dune Sand



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

Sample 13 Camp Skeel Sand

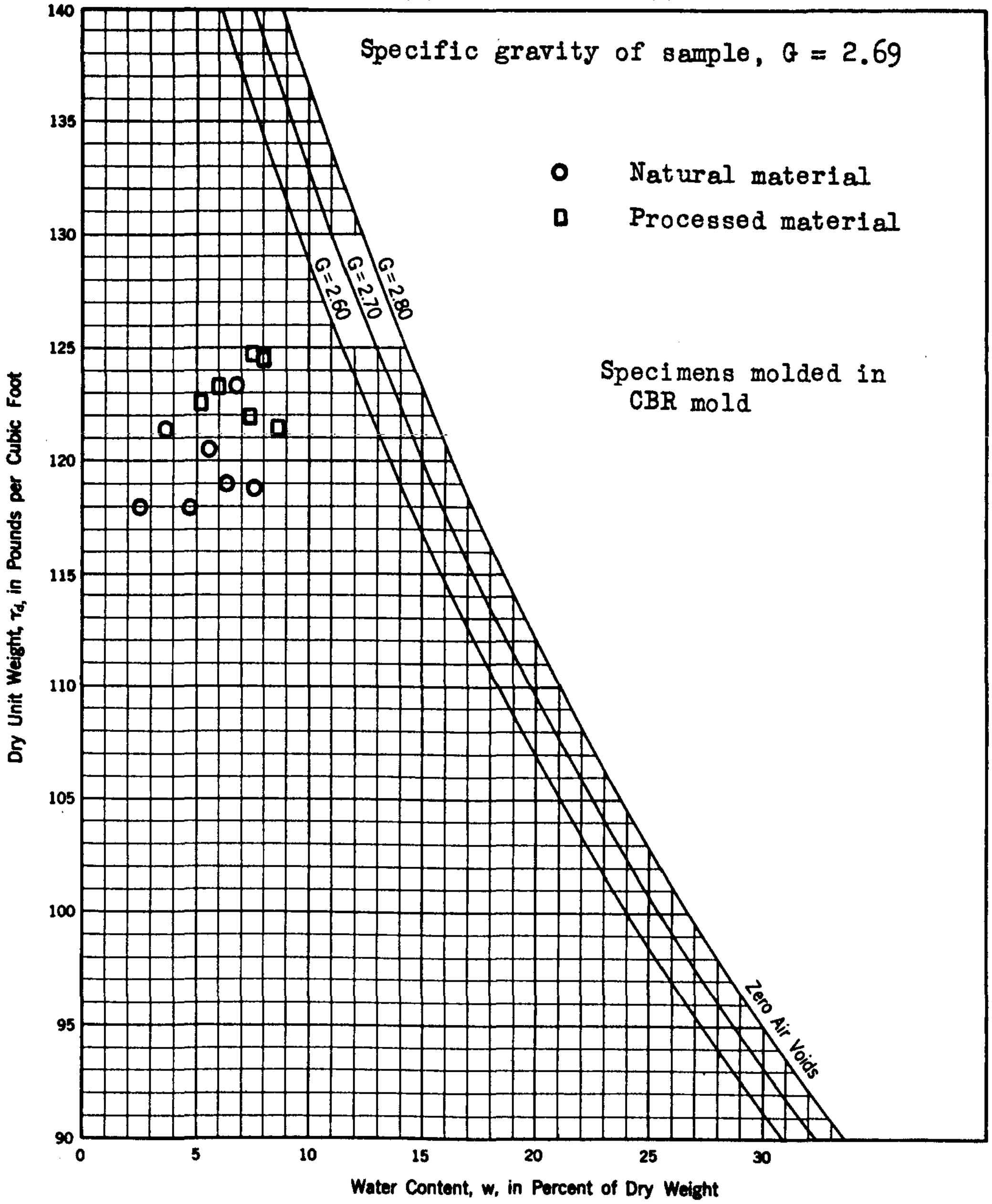


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

Sample 14 Manchester Sand

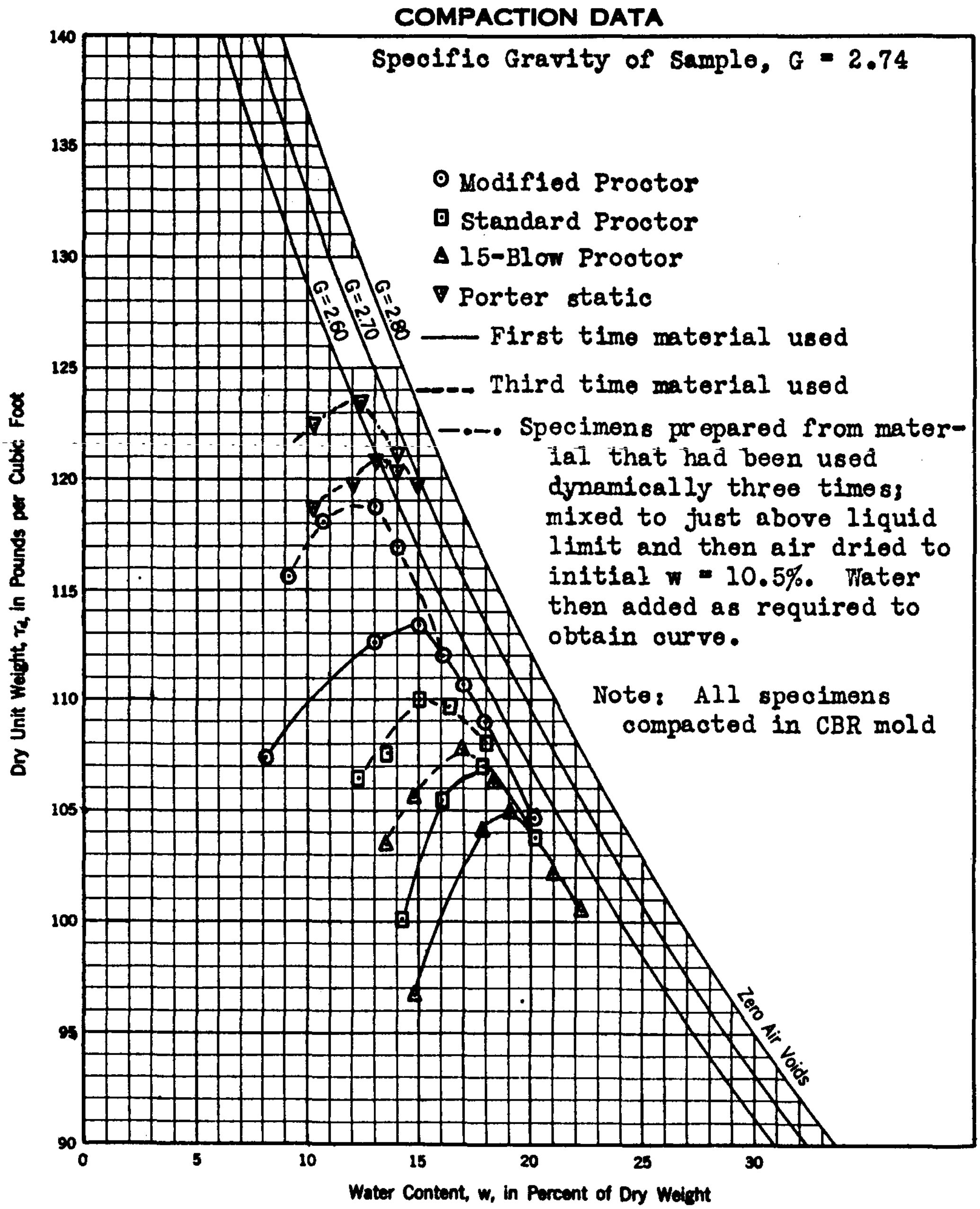
COMPACTION DATA



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MODIFIED PROCTOR COMPACTION

Sample 15 Manchester Sand and Gravel

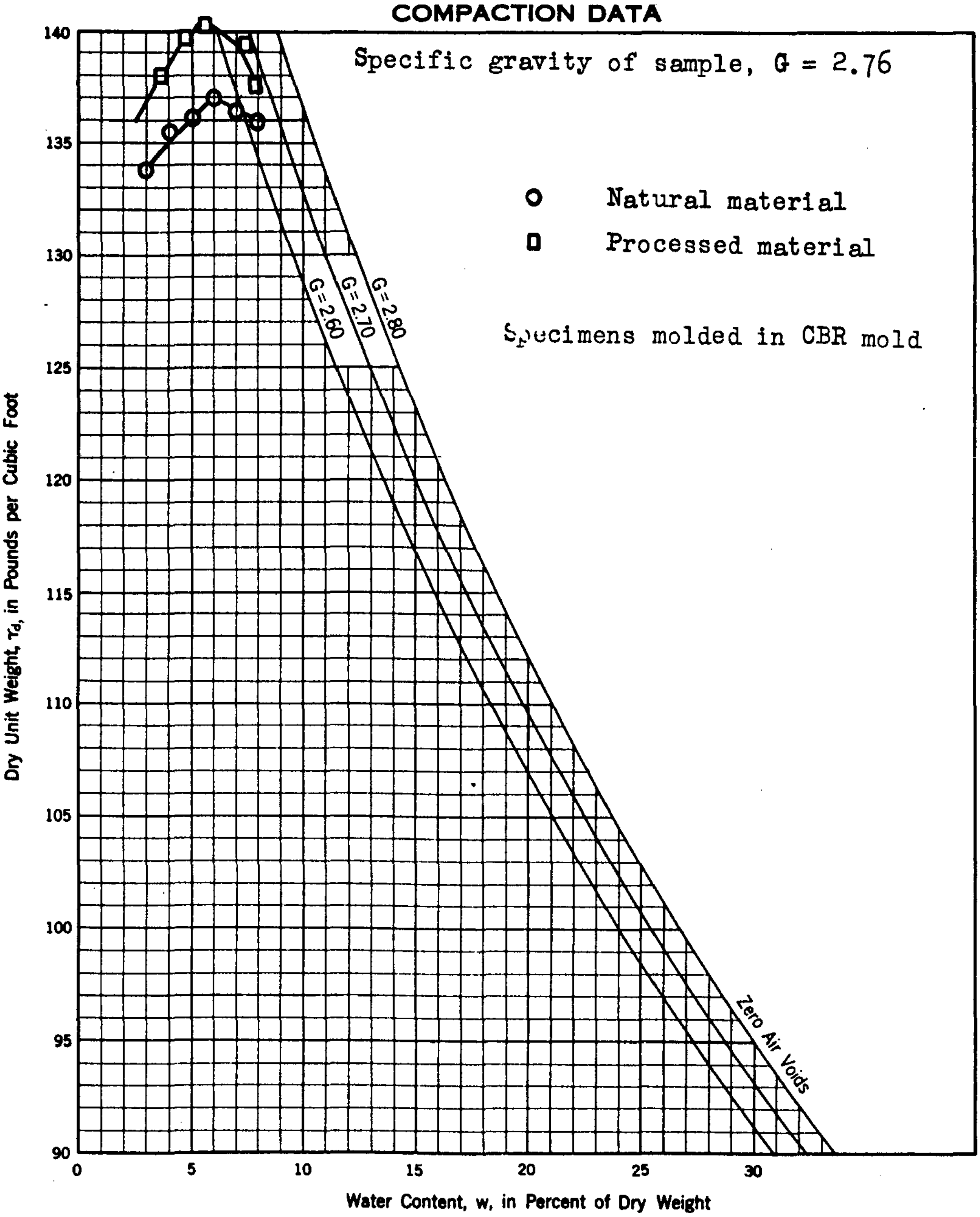


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

Sample 18 Berry Field Lean Clay

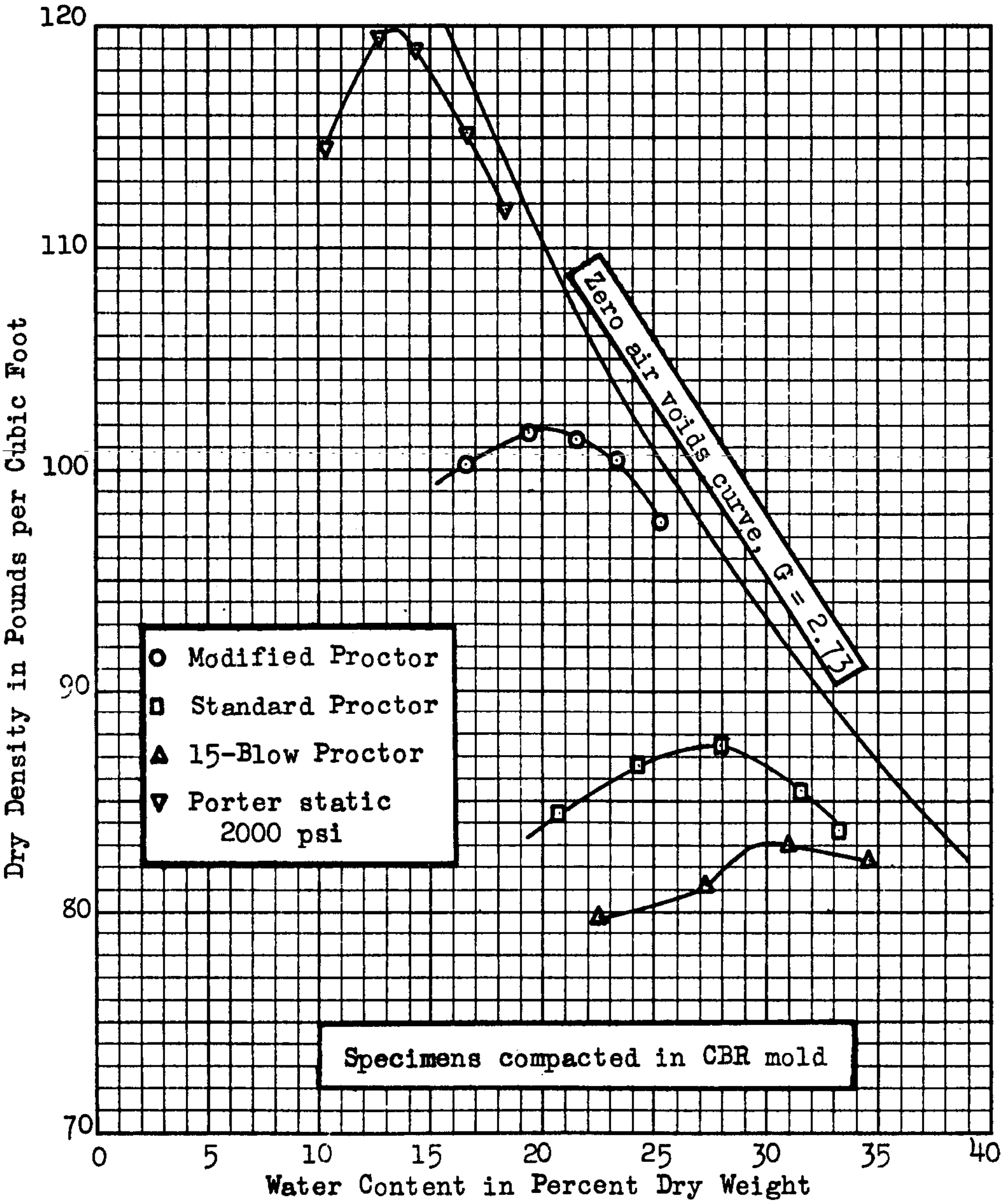
COMPACTION DATA



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MODIFIED PROCTOR COMPACTION

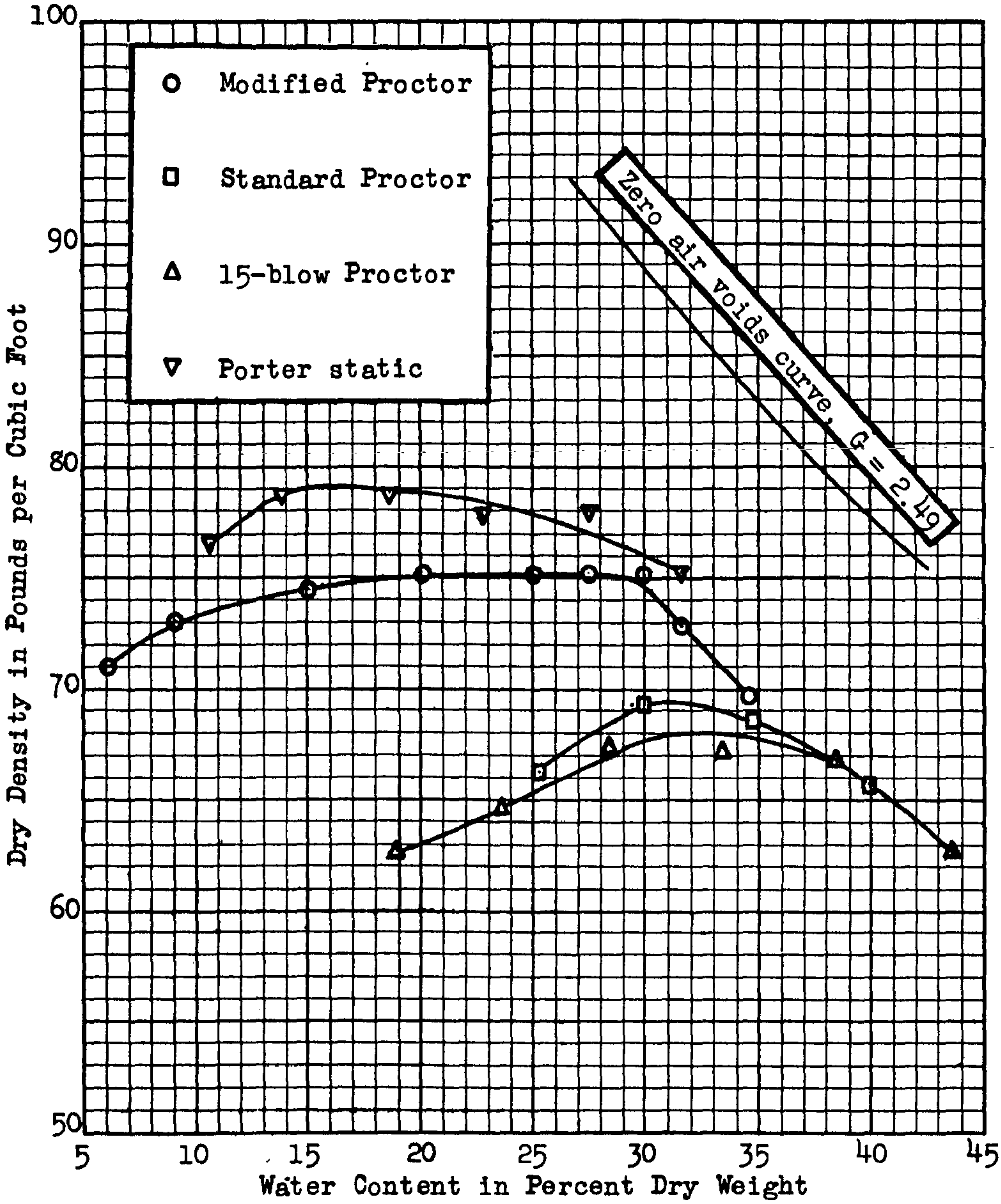
Sample 19 Well Graded Walesboro Gravel Sand Silt



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

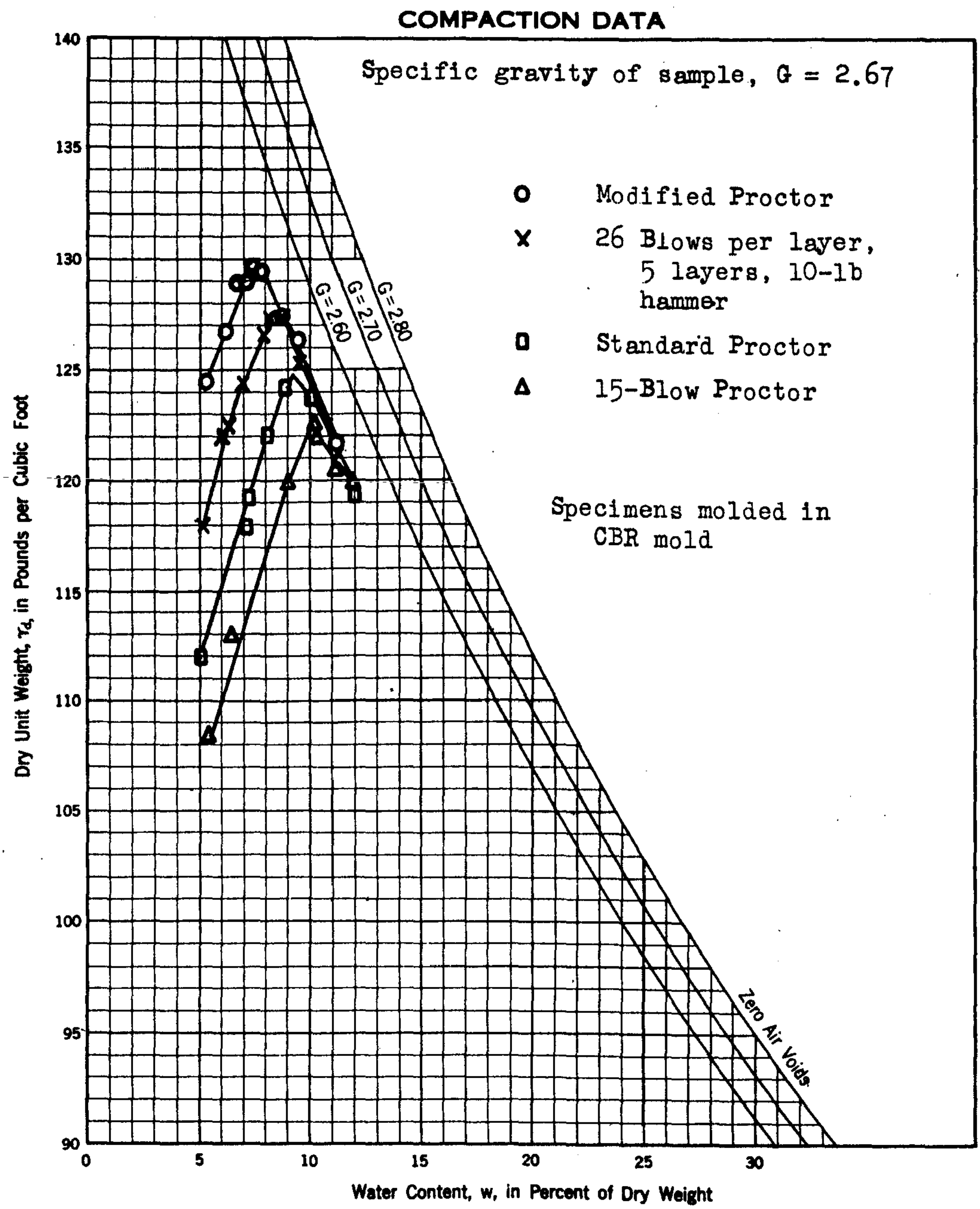
Sample 20 Stockton Adobe Clay



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

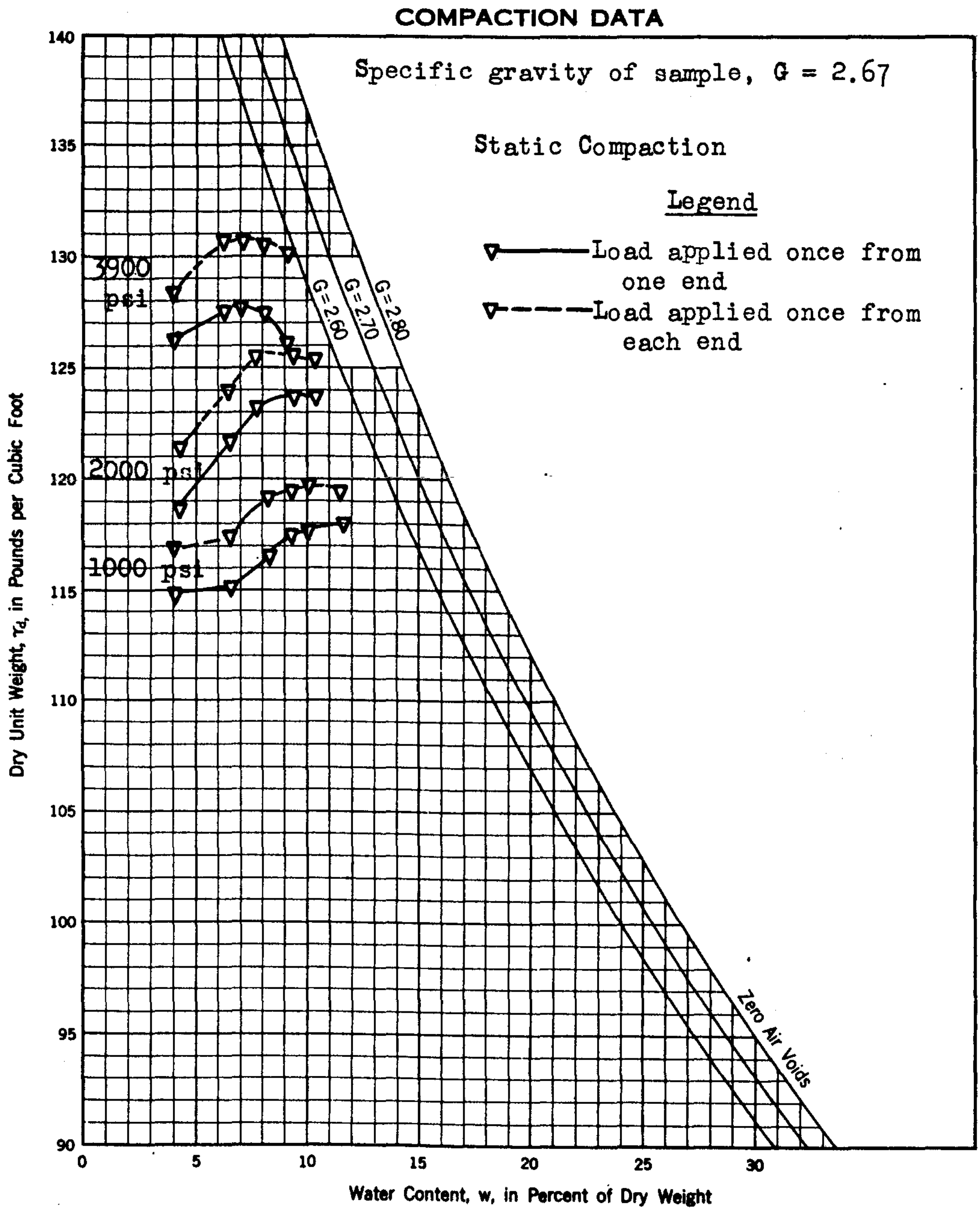
Sample 21 California Pumice



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPACTION DATA

Sample 23 Vicksburg Clay Sand



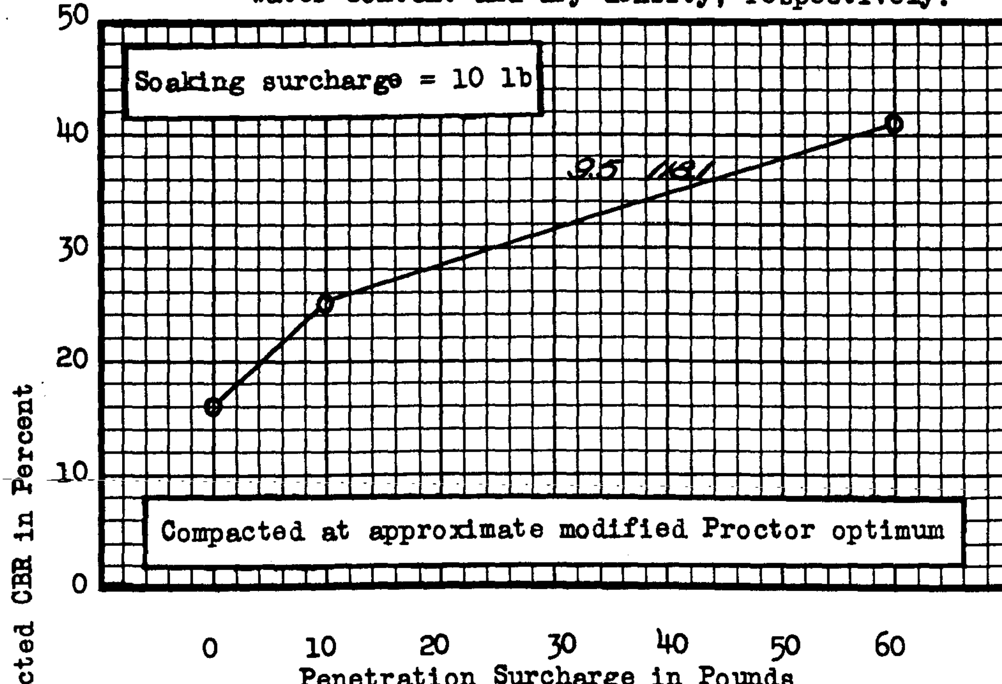
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

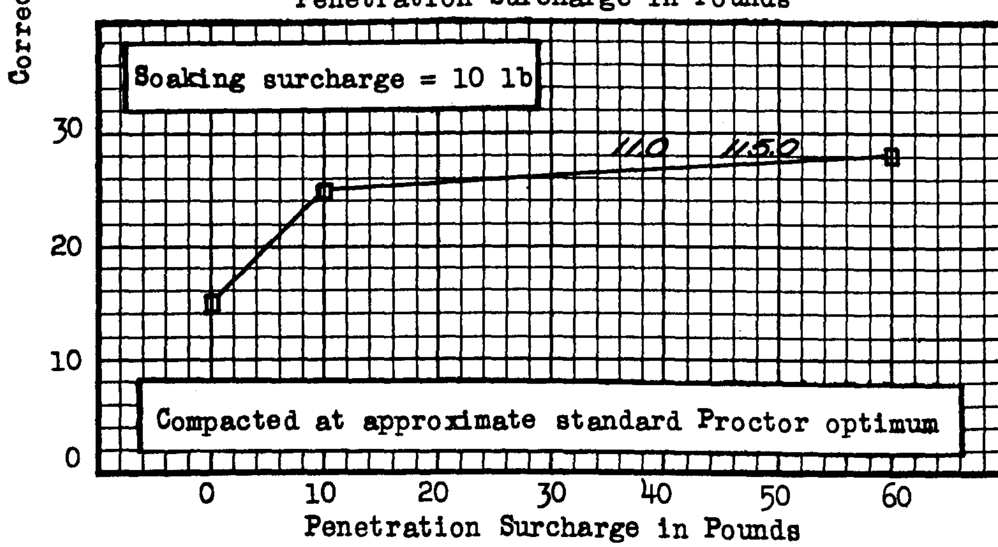
COMPACTION DATA

Sample 23 Vicksburg Clay Sand

Notes: All specimens soaked from bottom only.

Figures beside curves indicate average molding water content and dry density, respectively.





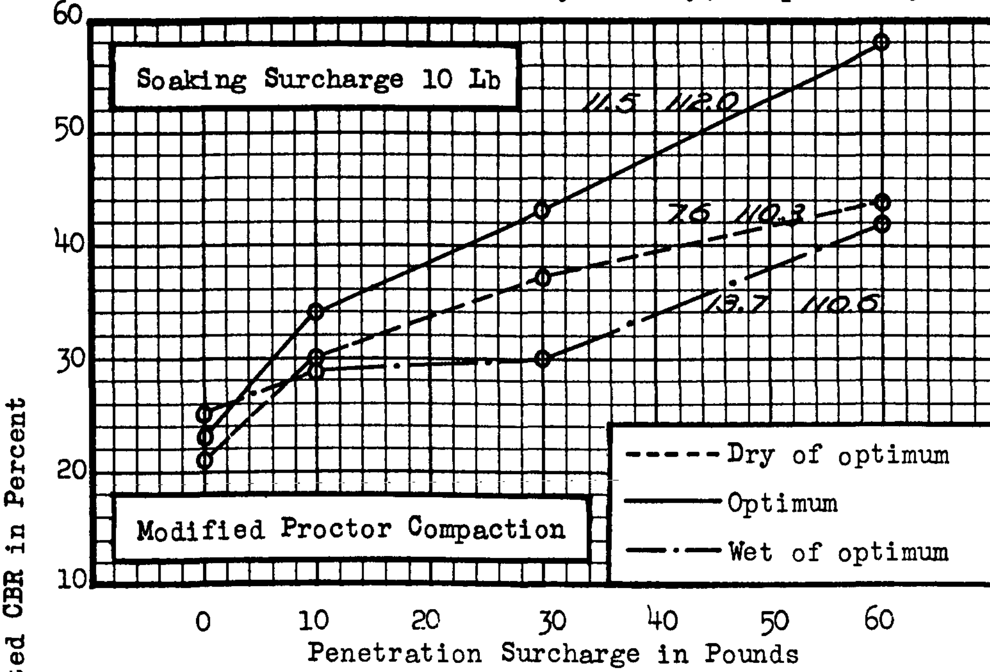
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

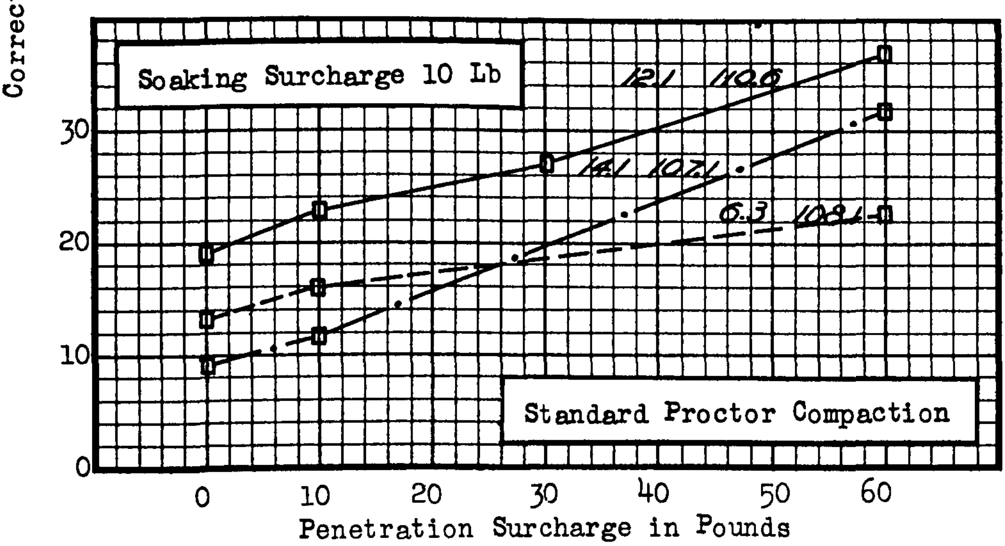
SURCHARGE VS CBR

Sample 6 Minden Sand

Notes: All specimens soaked from bottom only.

Figures beside curves indicate average molded water content and dry density, respectively.





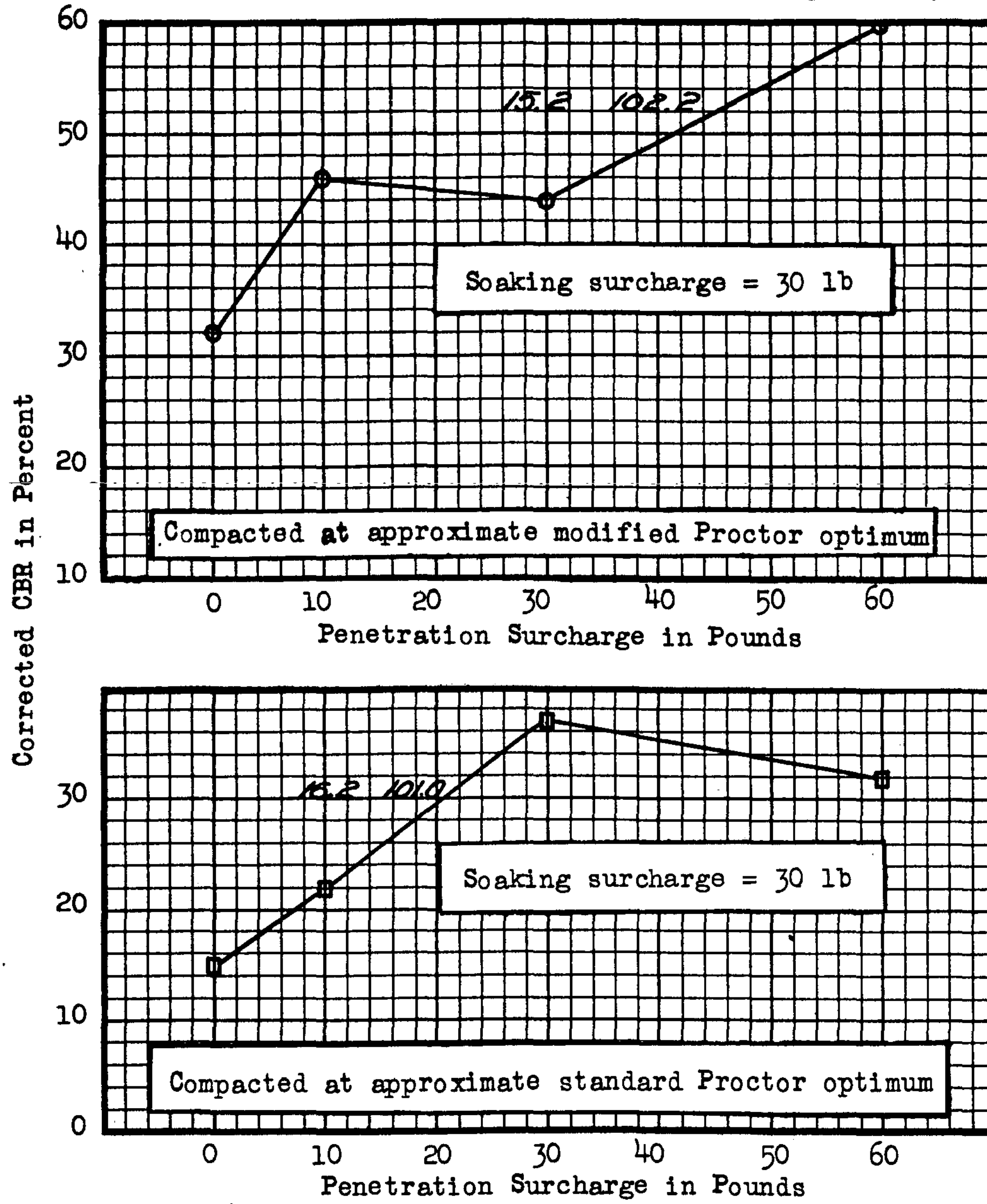
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 10 Eglin Sand

Notes: All specimens soaked from bottom only.

Figures beside curve indicate average molded water content and dry density, respectively.

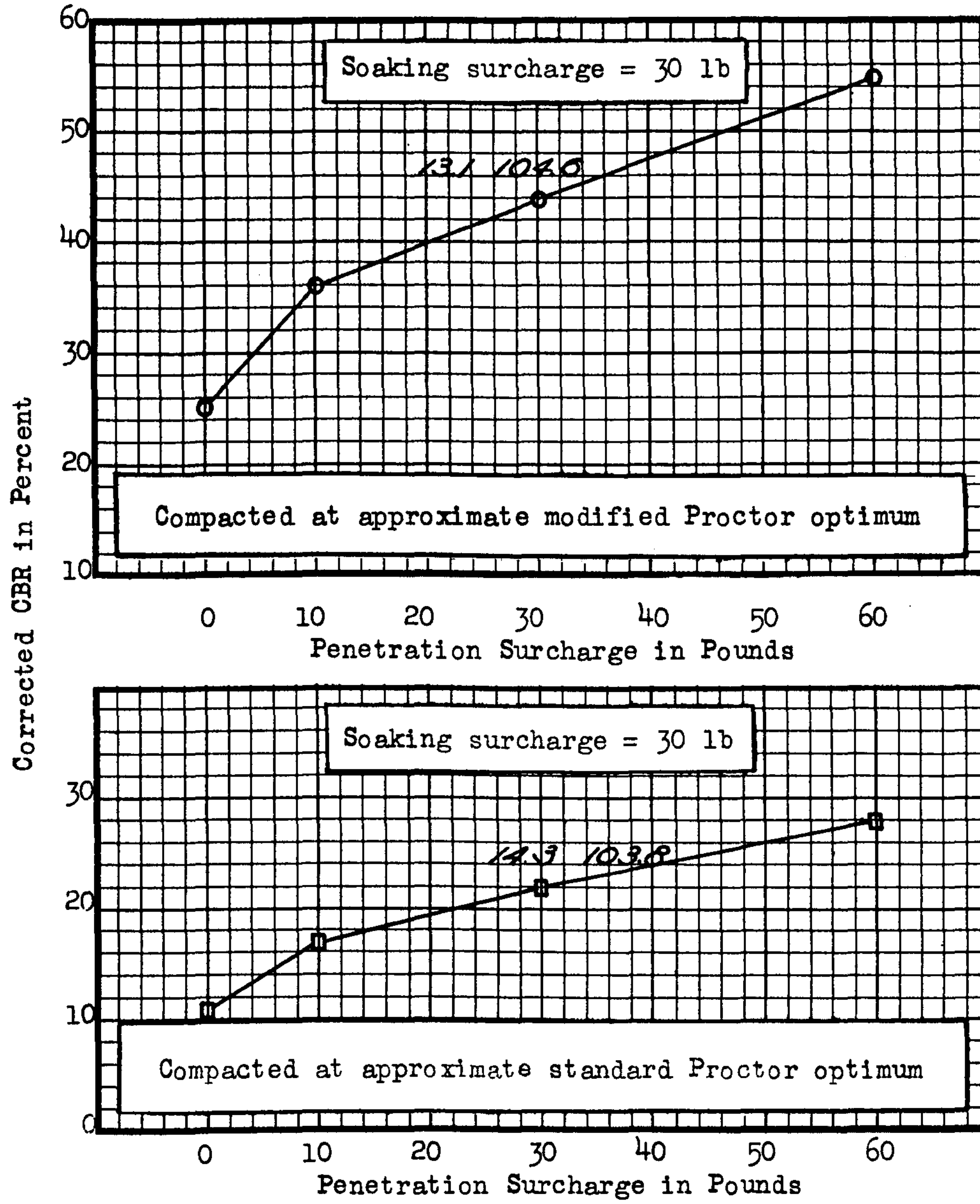


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 11 Florida Sand

Notes:
All specimens soaked from bottom only.
Figures beside curves indicate average molded water content and dry density, respectively.



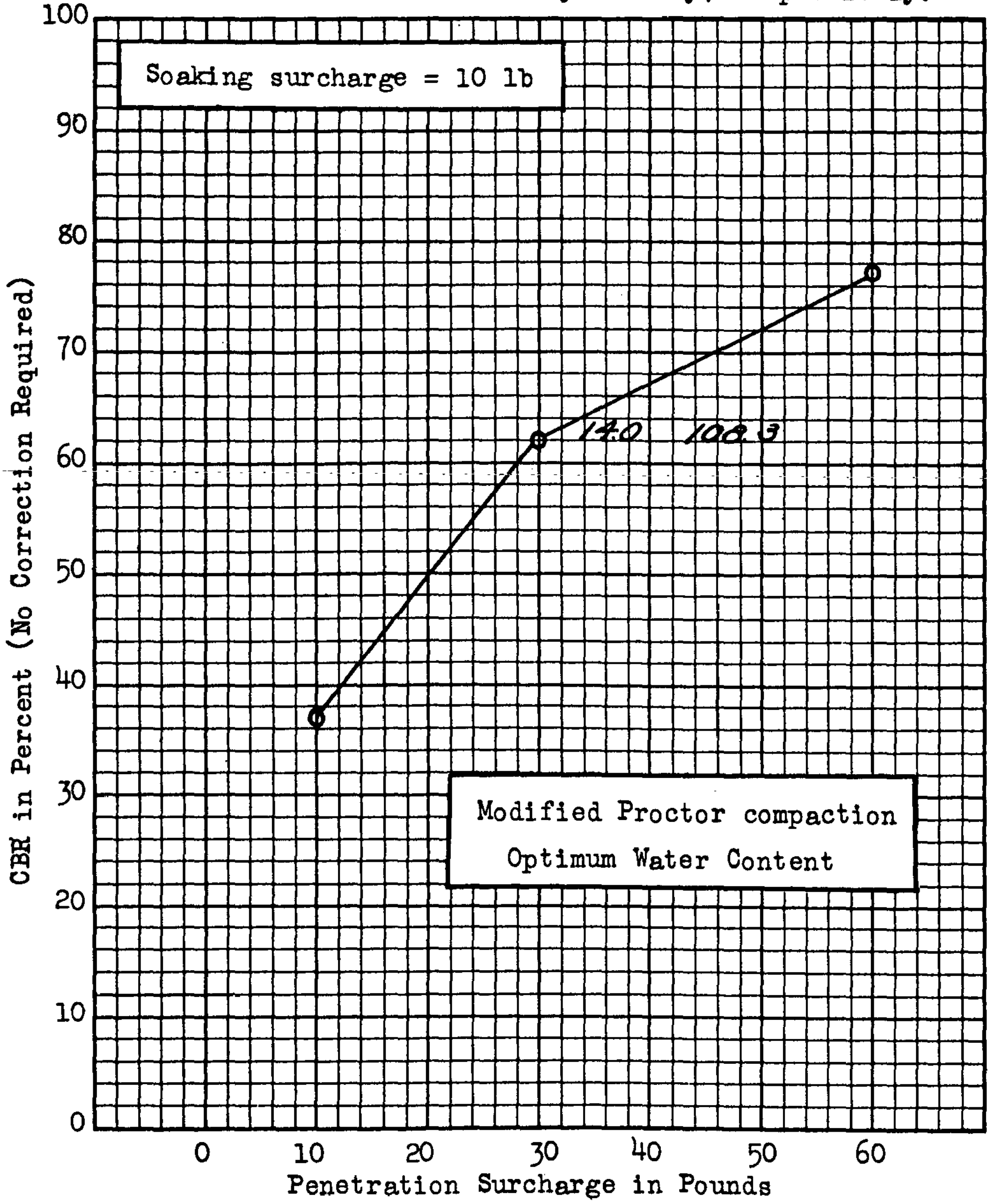
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 12 Michigan Dune Sand

Notes: All specimens soaked from bottom only.

Figures beside curve indicate average molding water content and dry density, respectively.



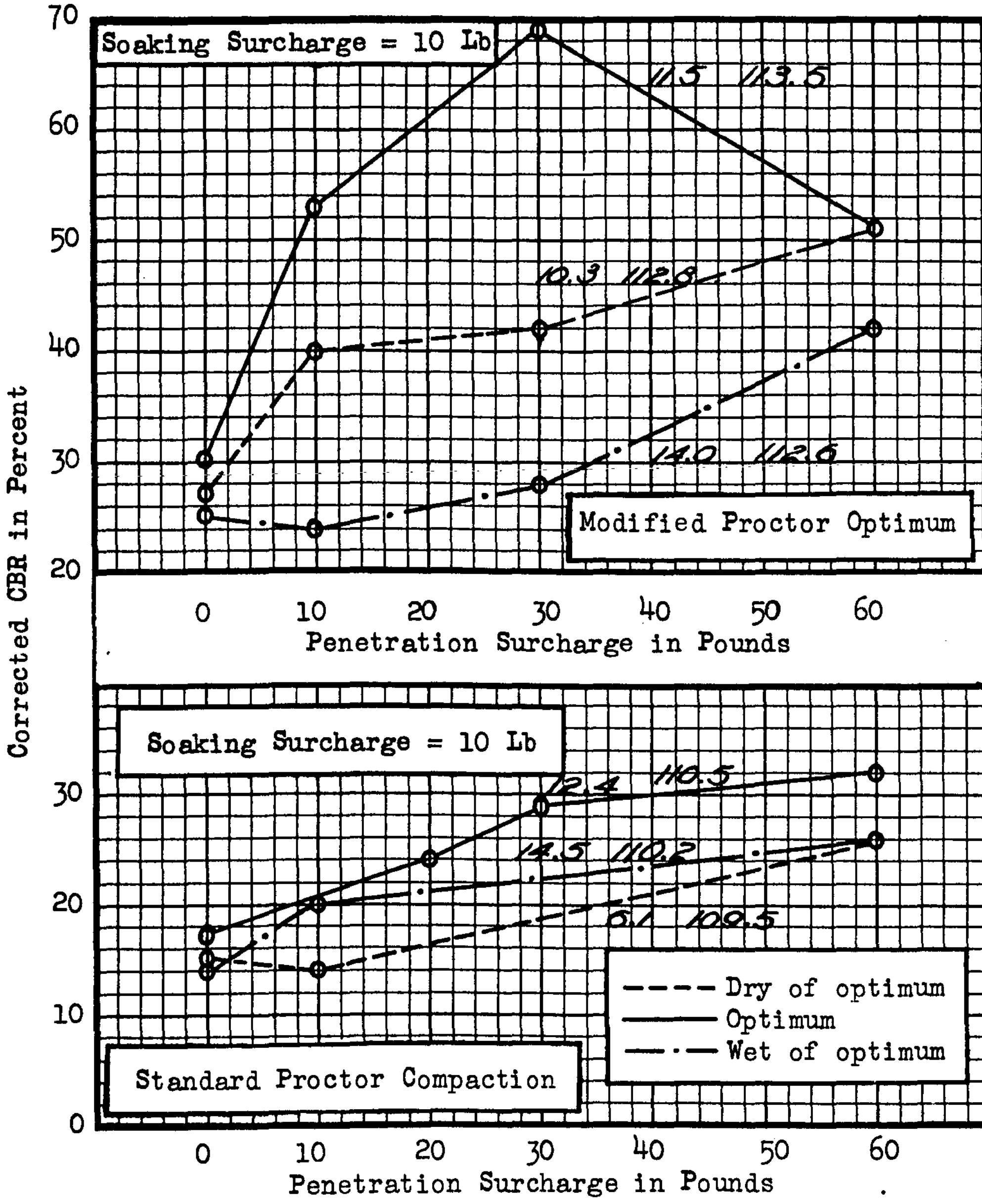
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 13 Camp Skeel Sand

Notes: All specimens soaked from bottom only.

Figures beside curves indicate average molded water content and dry density, respectively.

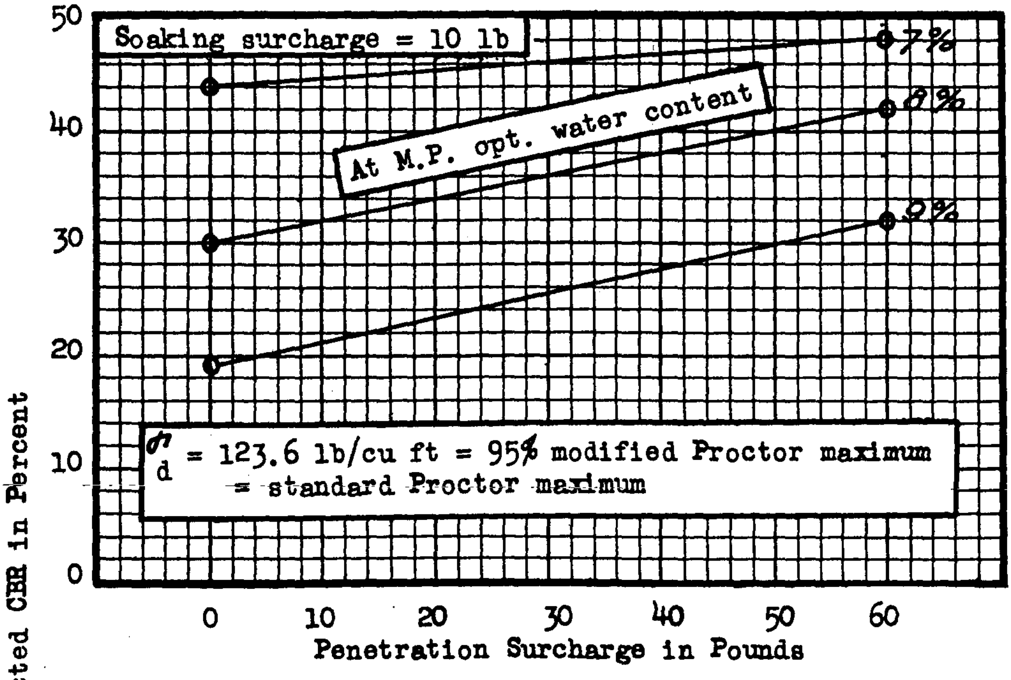


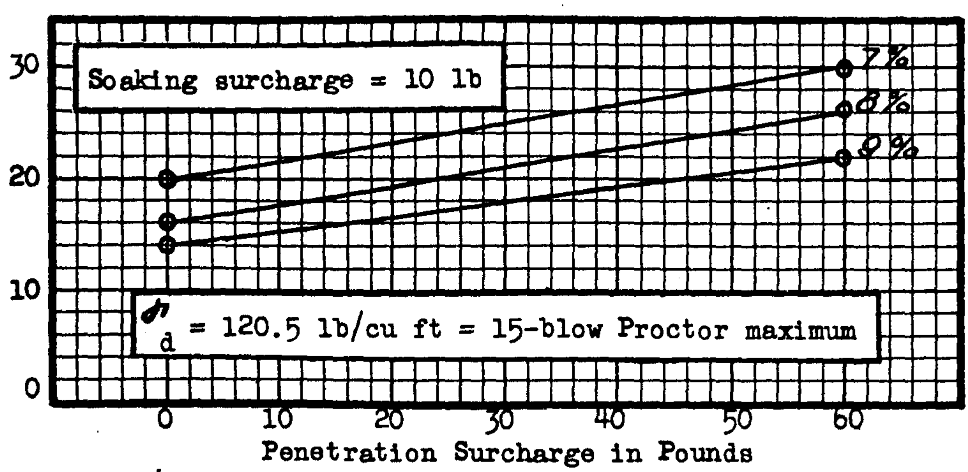
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 14 Manchester Sand

Notes: All specimens soaked top and bottom for four days.
Figure beside curve is molding water content.
Data taken from families of curves of γ_d vs CBR for constant molding water contents.





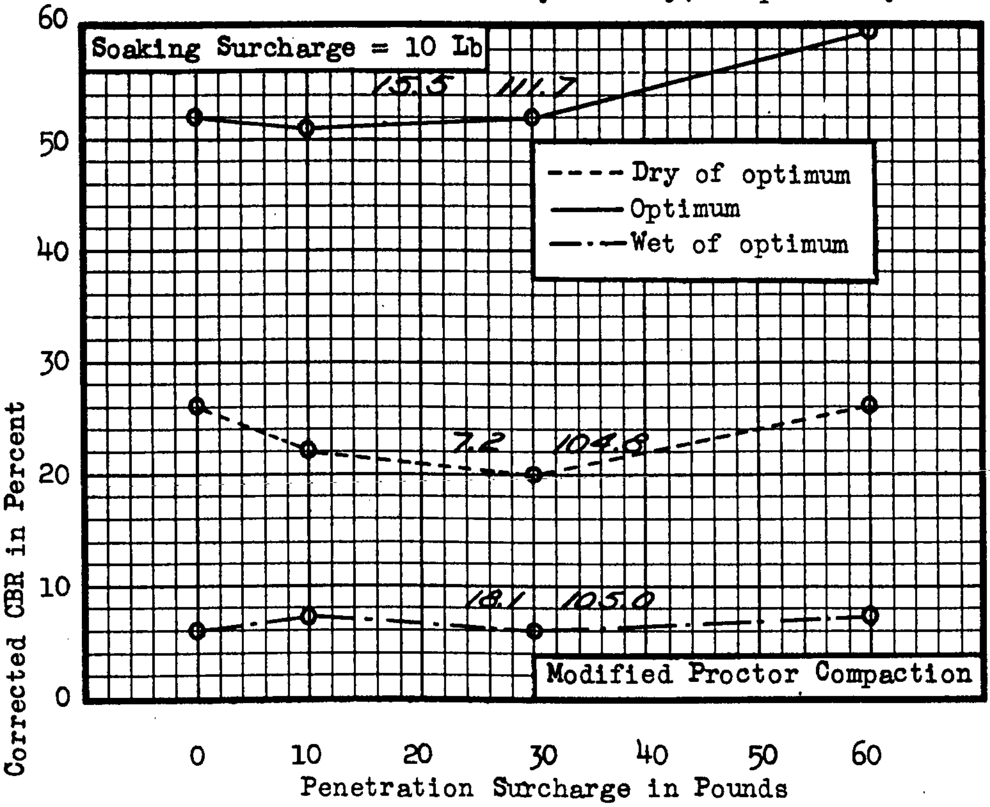
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

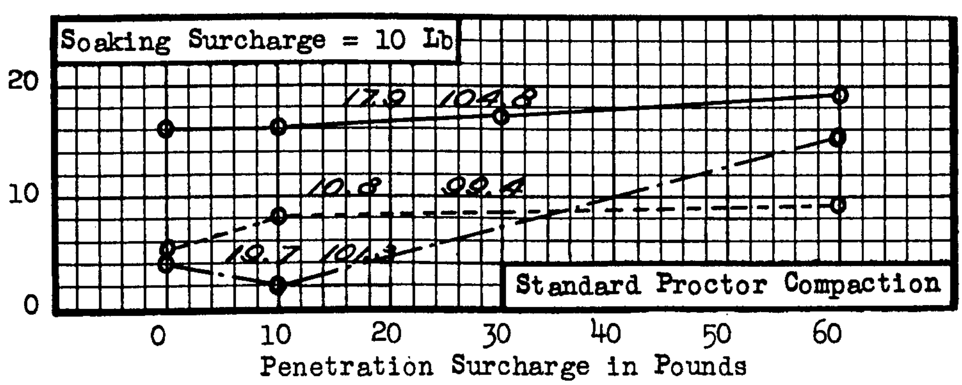
SURCHARGE VS CBR

Sample 1 DeRidder Clay Sand

Notes: All specimens soaked from bottom only.

Figures beside curves indicate average molded water content and dry density, respectively.





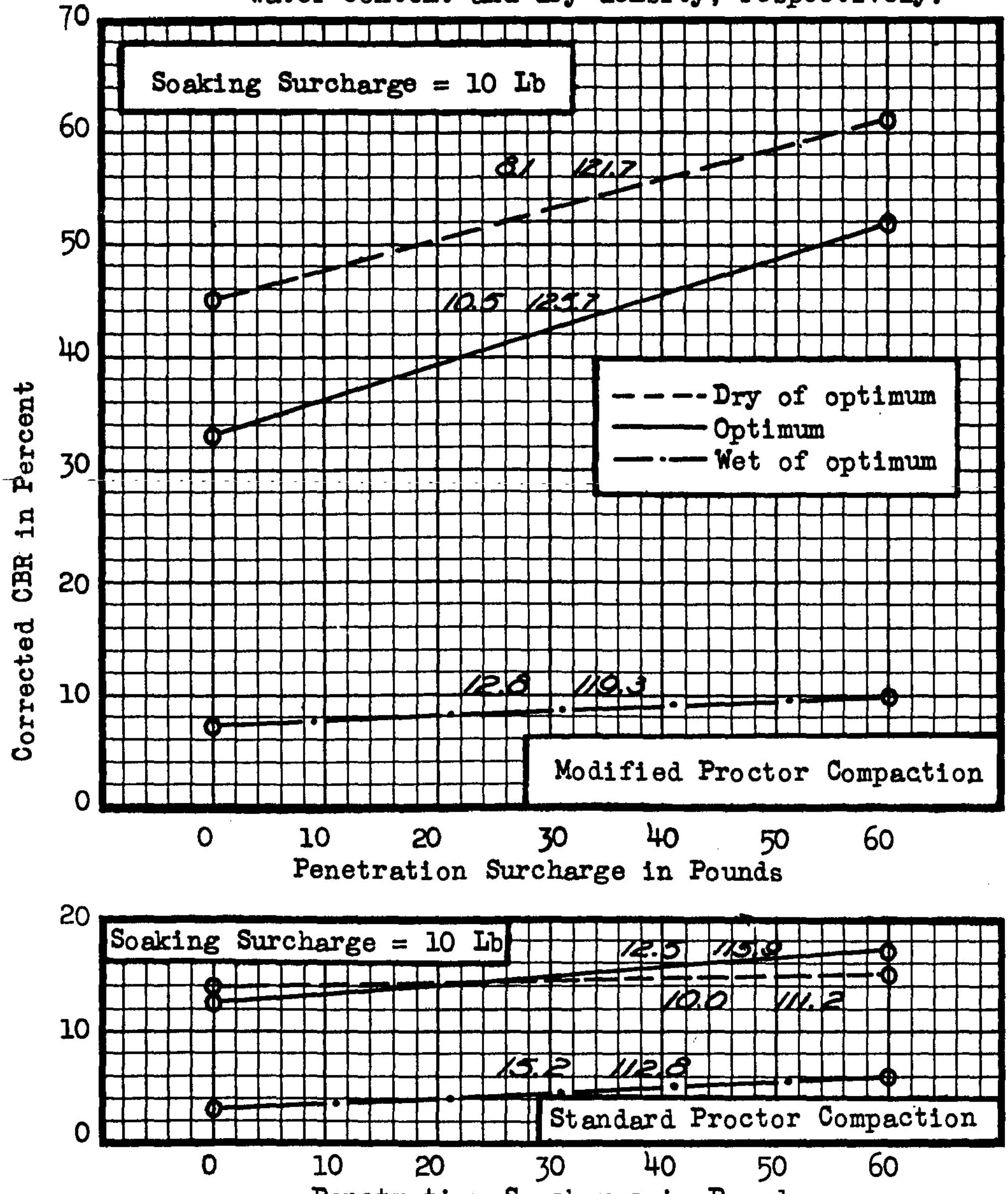
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 3 Vicksburg Loess

Notes: All specimens soaked top and bottom.

Figures beside curves indicate average molded water content and dry density, respectively.

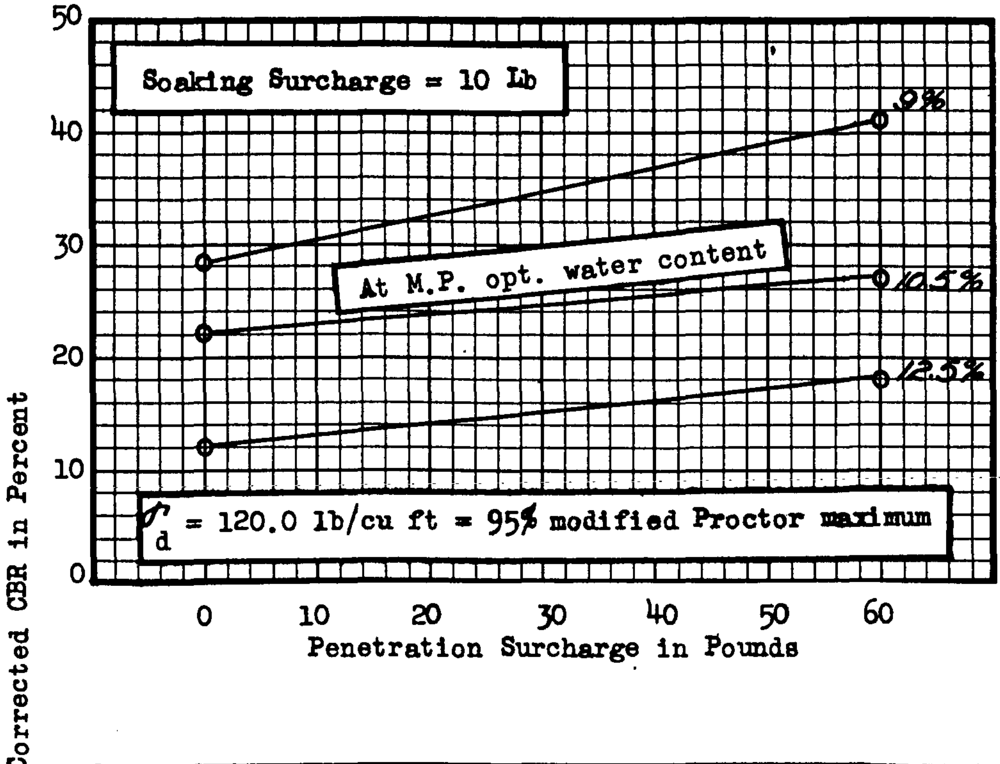


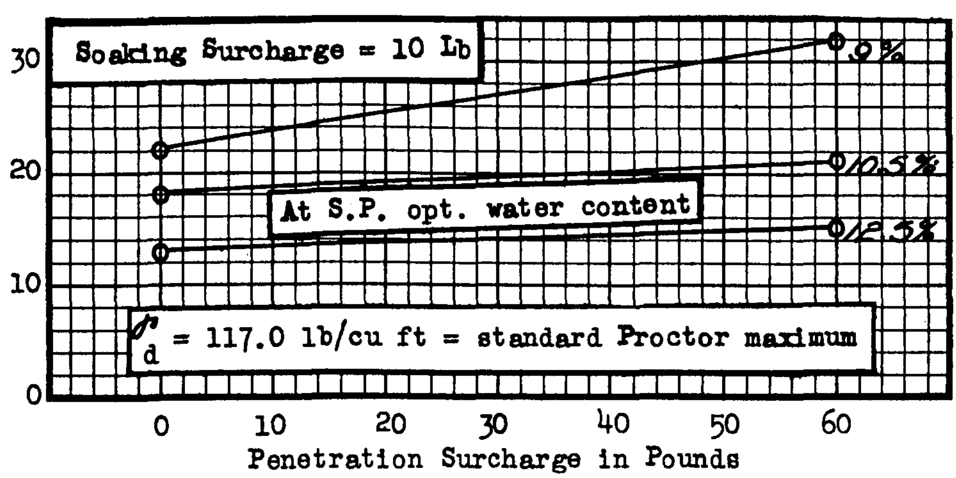
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 5 DeRidder Sand Clay (Loam)

Notes: All specimens soaked top and bottom for four days.
Figure beside curve is molding water content.
Data taken from families of curves of γ_d vs CBR for constant molding water contents.



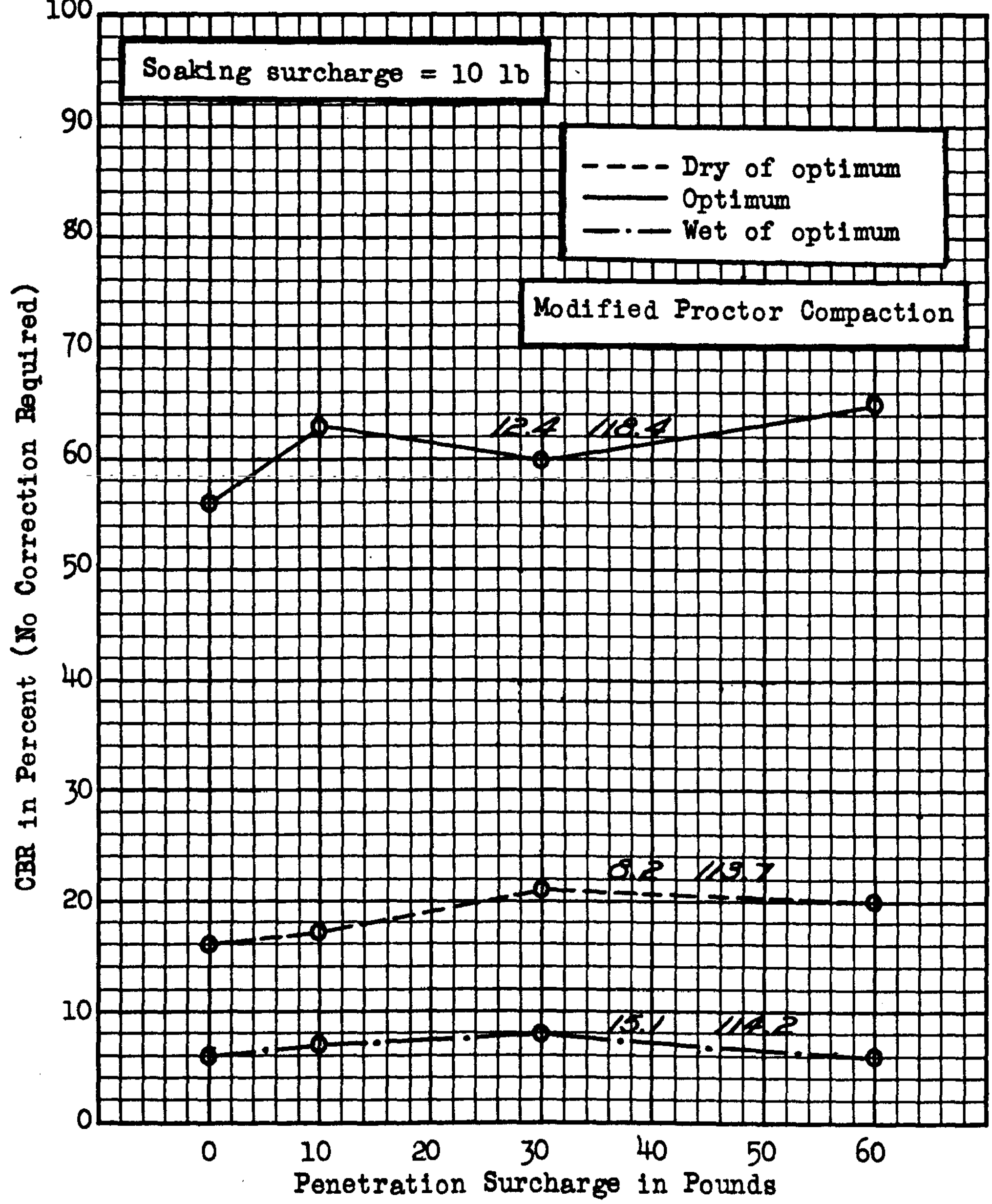


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 5 DeRidder Sand Clay (Loam)

Notes: All specimens soaked from bottom only for 14 days.
Figures beside curves indicate average molded water content and dry density, respectively.
Third time material was used for compaction.

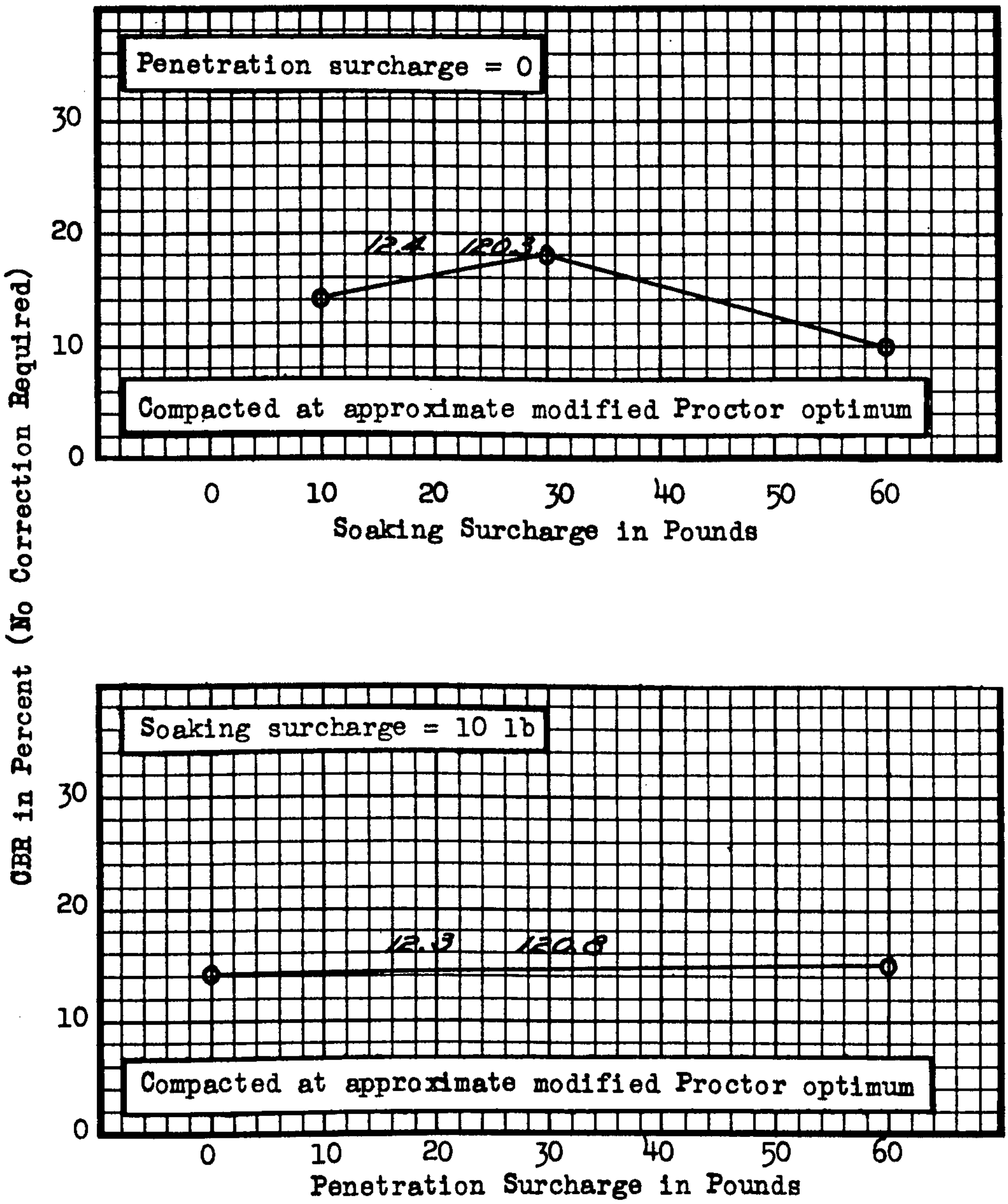


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

SURCHARGE VS CBR

Sample 18 Berry Field Lean Clay

Notes: All specimens soaked top and bottom for four days.
Figures beside curves indicate average molded water content and dry density, respectively.
Third time material was used for compaction.



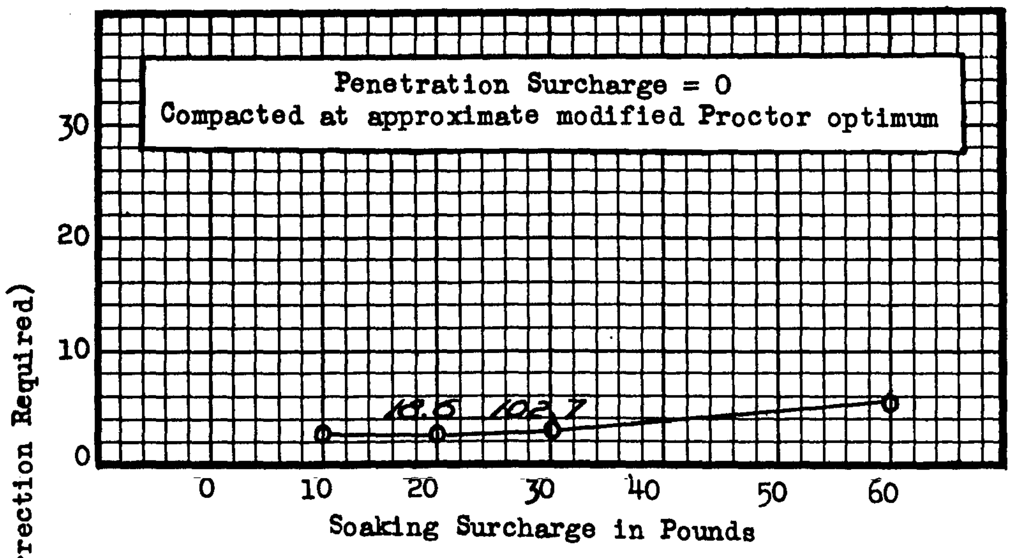
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

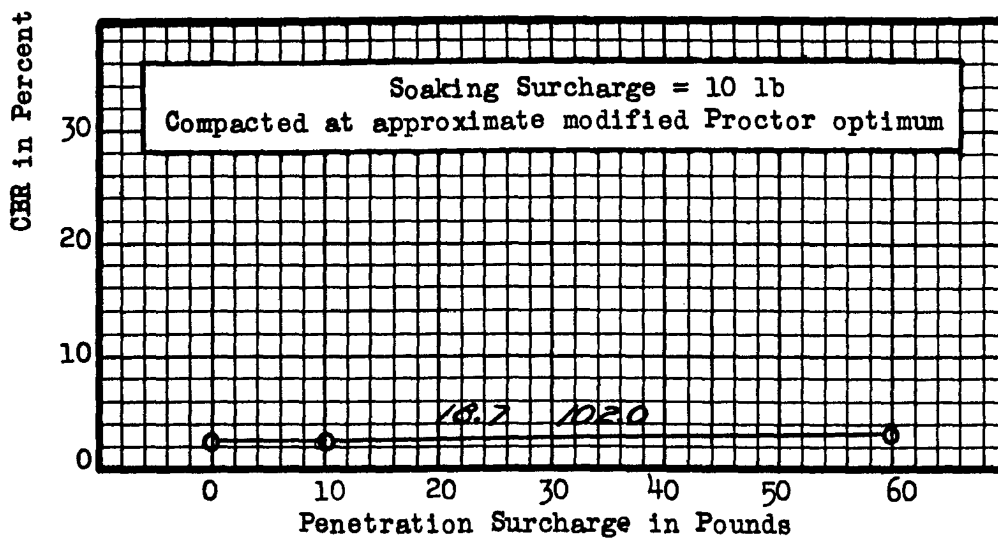
SURCHARGE VS CBR

Sample 18 Berry Field Lean Clay

Notes:

All specimens soaked top and bottom for four days. Figures beside curves indicate average molded water contents and dry density, respectively.

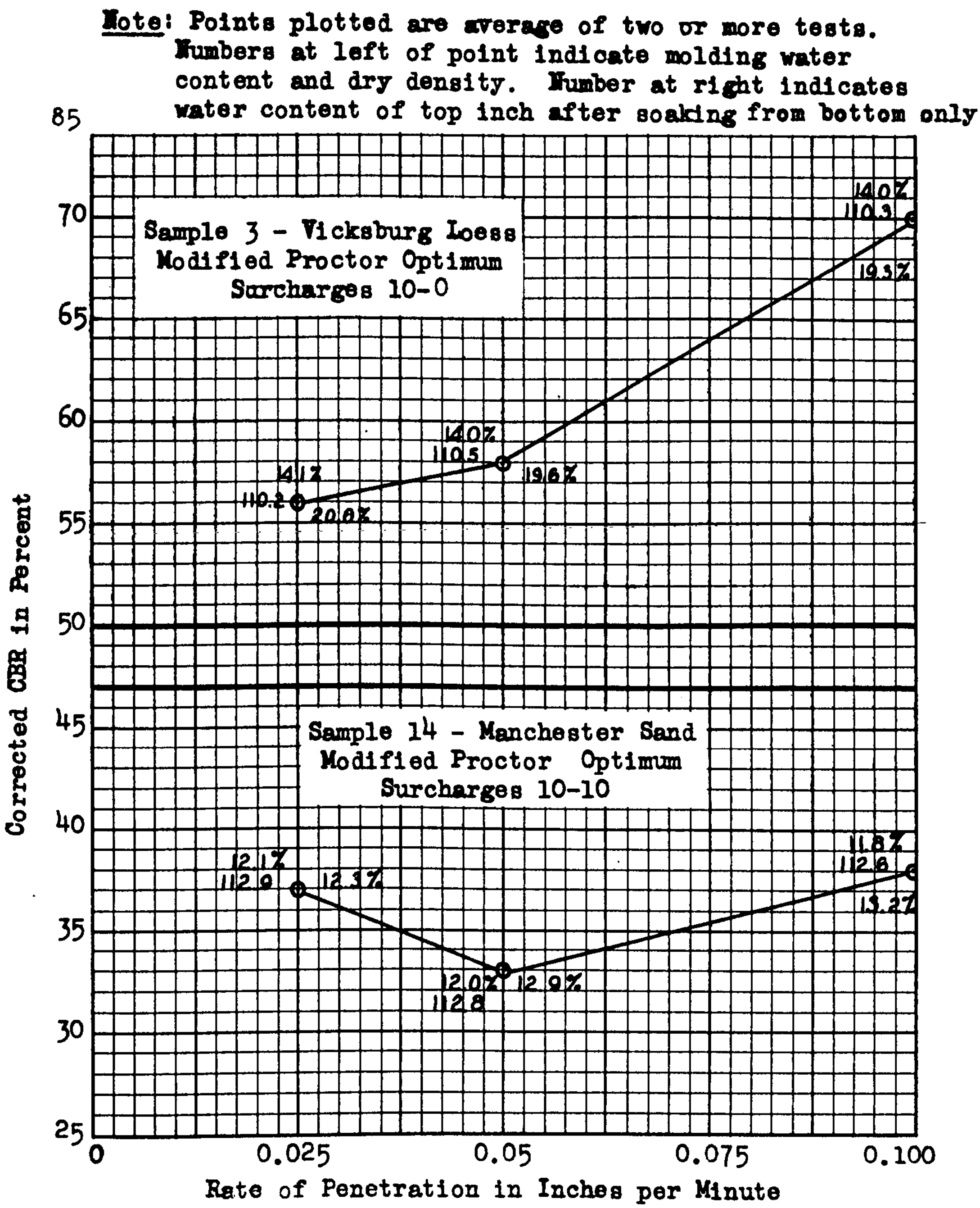




Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

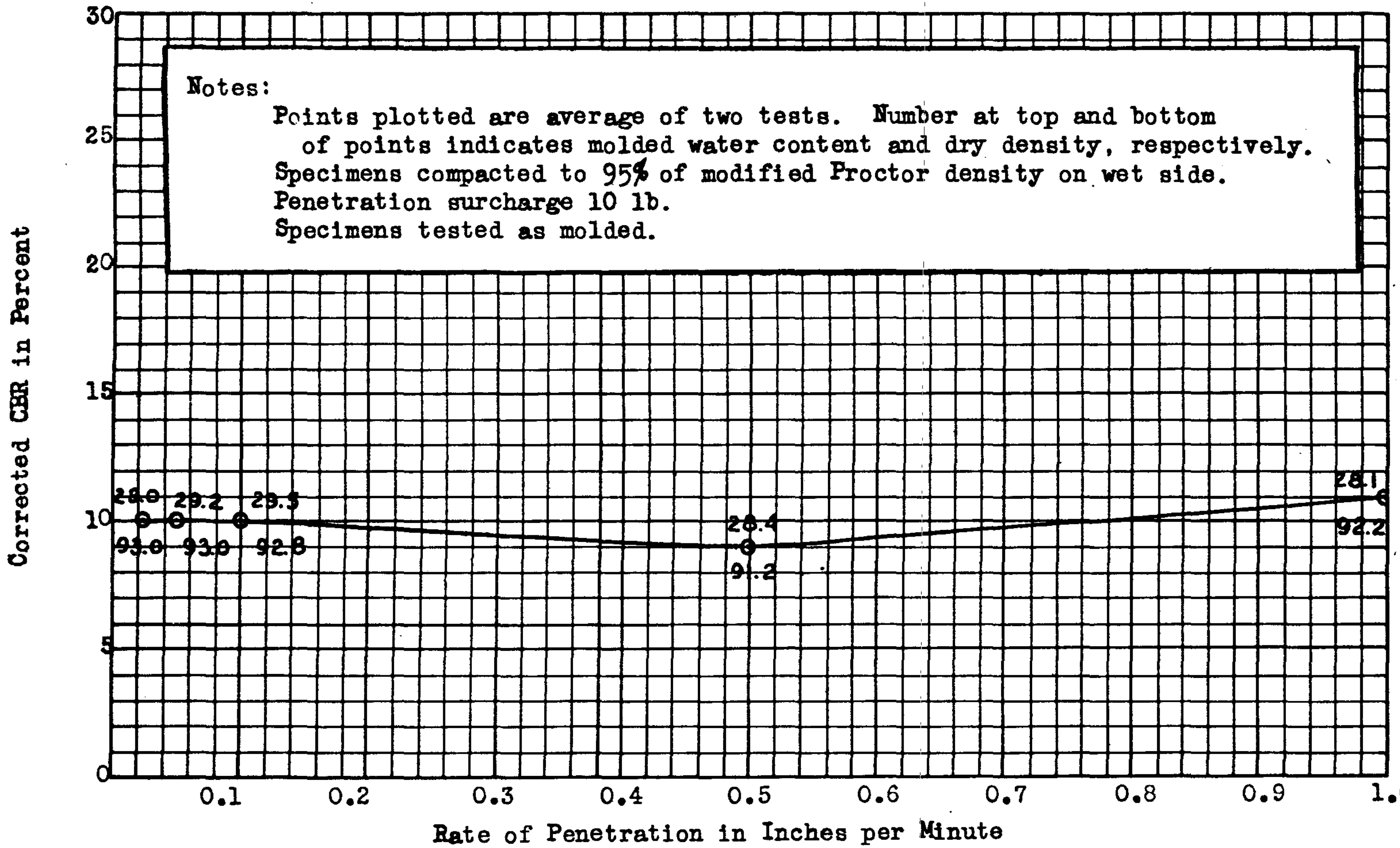
SURCHARGE VS CBR

Sample 20 Stockton Adobe Clay



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

CBR VS RATE OF PENETRATION

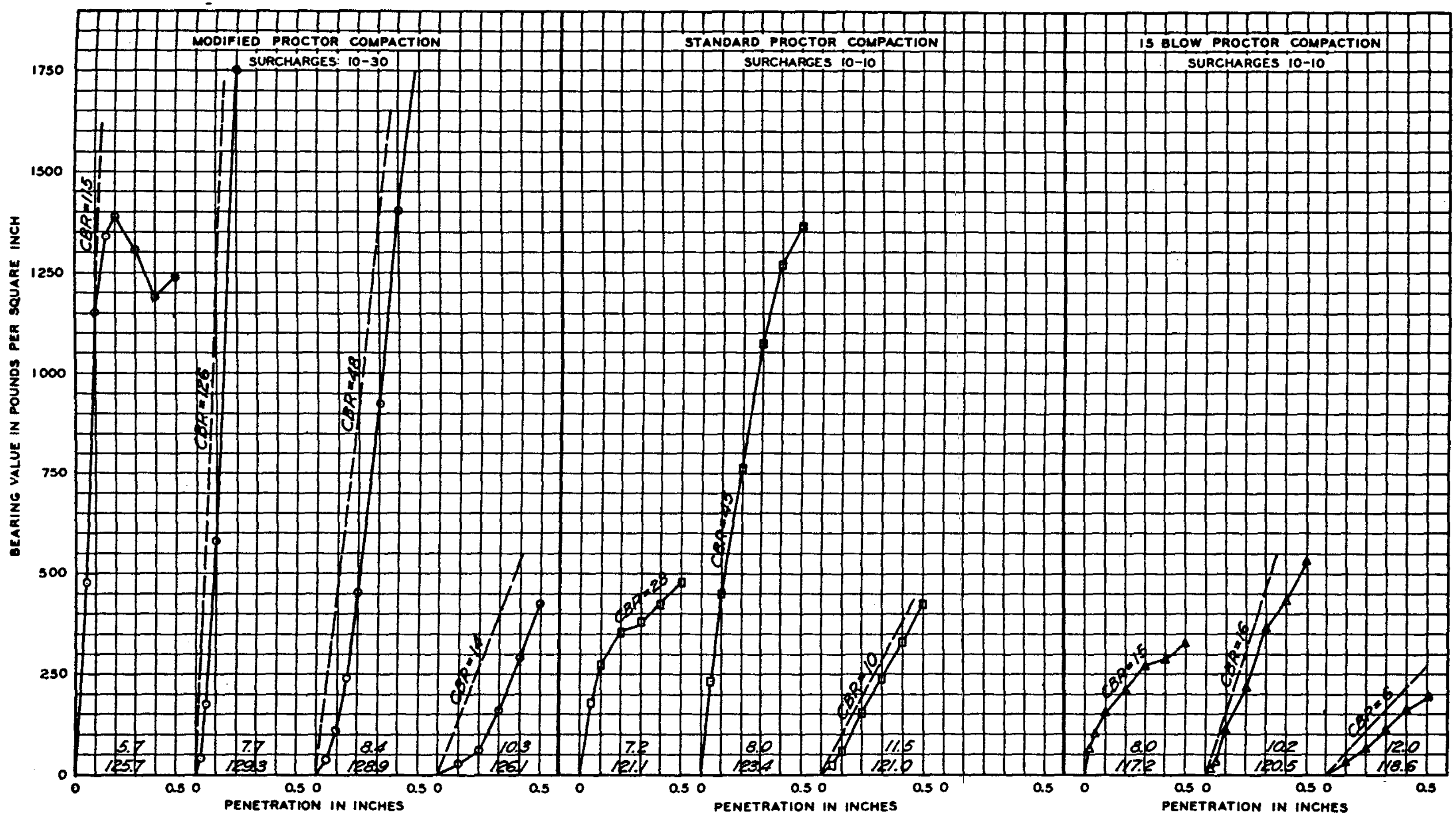


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

Sample 20

CBR VS RATE OF PENETRATION

Stockton Adobe Clay



NOTES: SPECIMENS SOAKED TOP AND BOTTOM.
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.

LEGEND

——— DATA FROM TEST

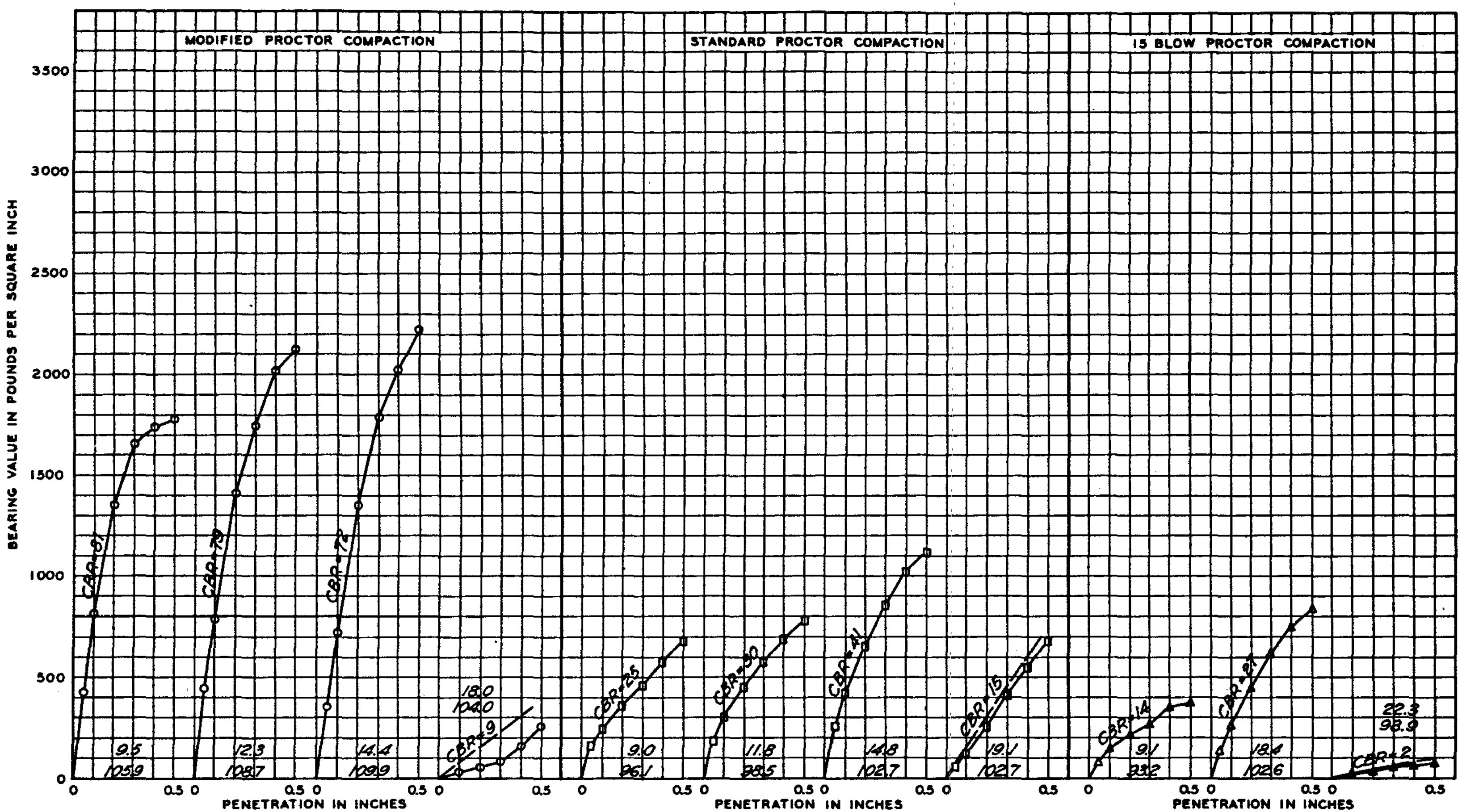
- - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES

SAMPLE I DE RIDDER CLAY SAND

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY



NOTES: SPECIMENS TESTED AS MOLDED.
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
PENETRATION SURCHARGE = 10 LBS.

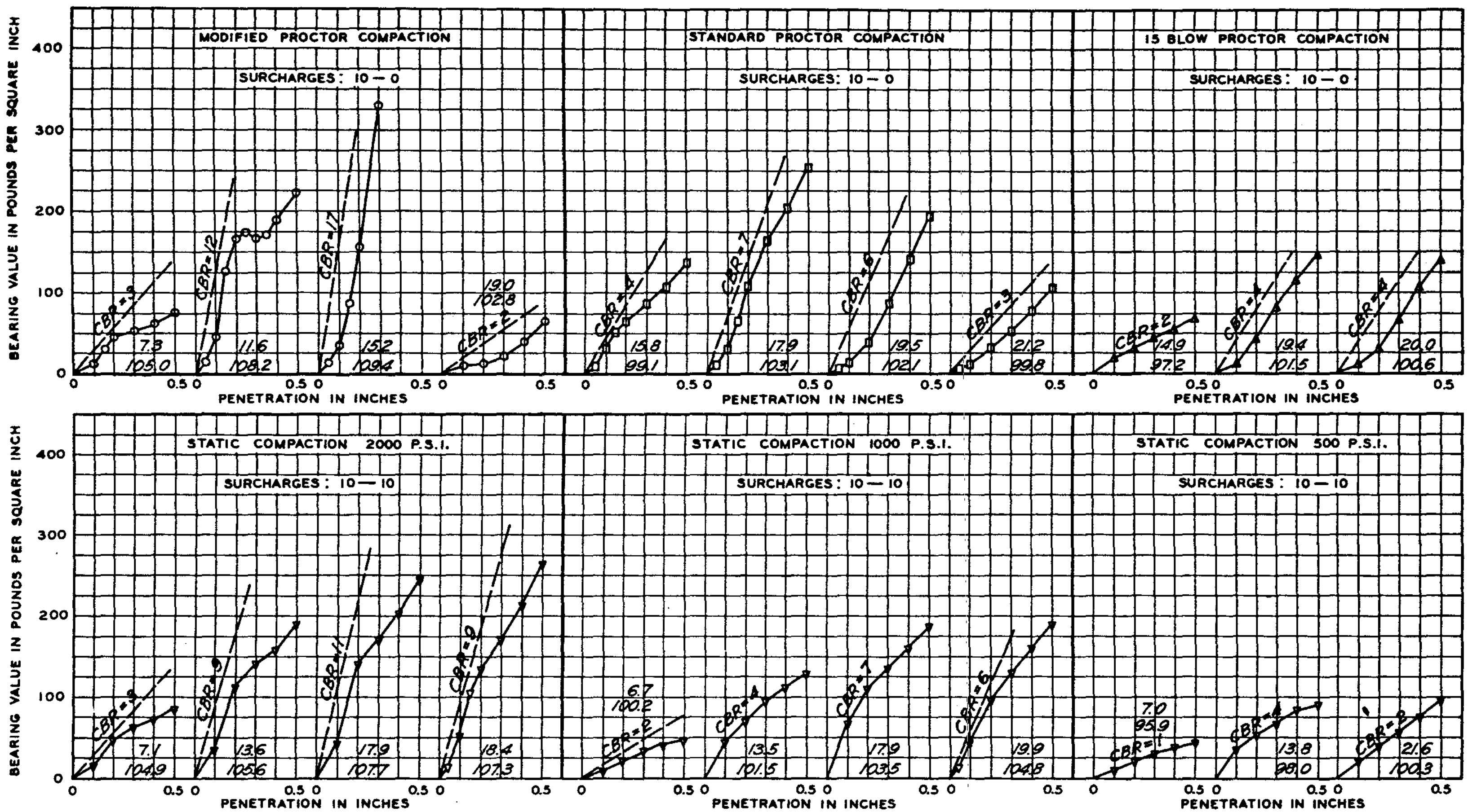
LEGEND

——— DATA FROM TEST

- - - - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES
SAMPLE 3 VICKSBURG LOESS



NOTES: SPECIMENS SOAKED TOP AND BOTTOM.
FIGURES AT BOTTOM OF CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.

LEGEND

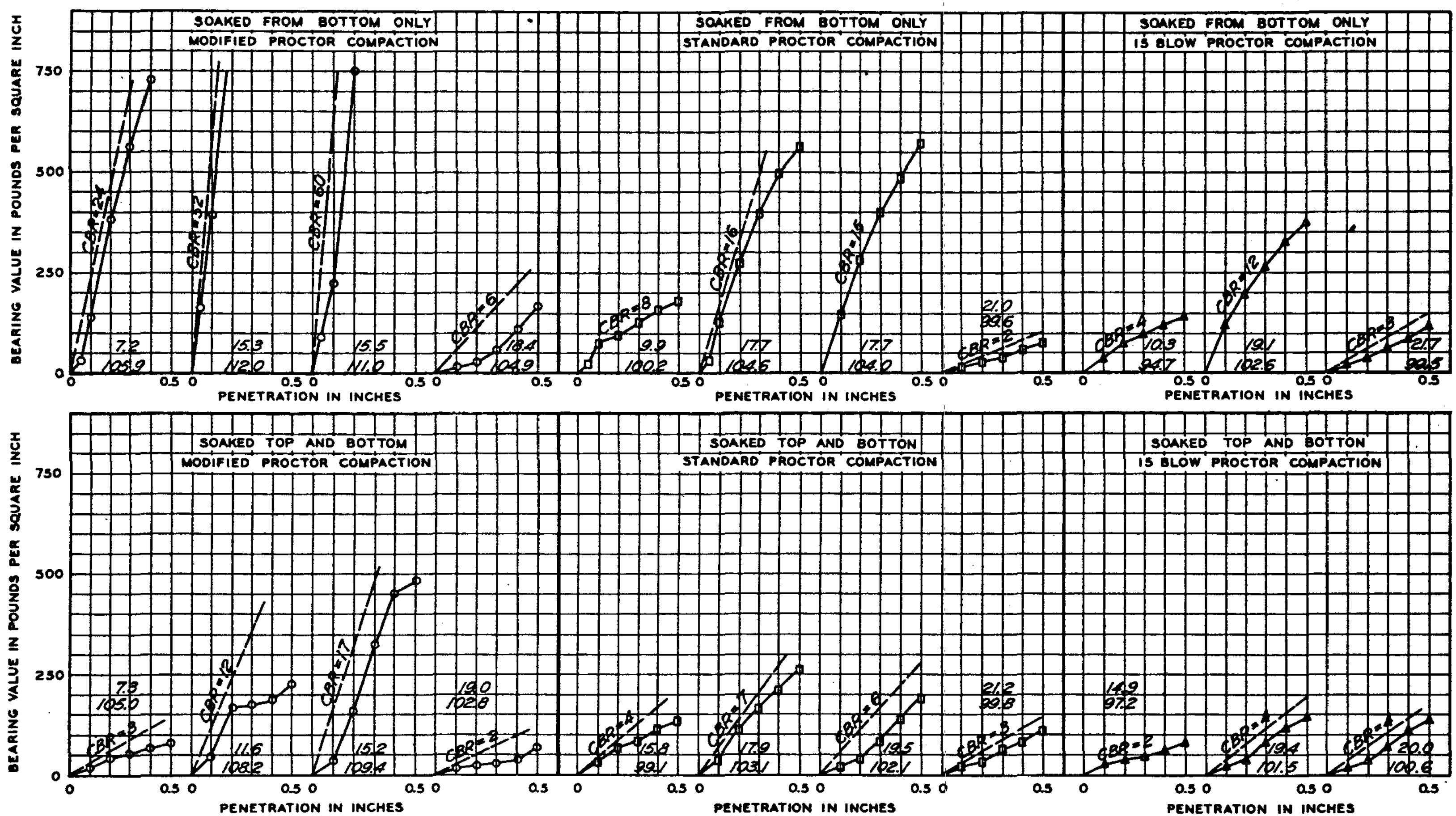
——— DATA FROM TEST

- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES

COMPARISON OF PENETRATION CURVES OF DYNAMICALLY COMPACTED WITH STATICALLY COMPACTED SPECIMENS
SAMPLE 3 VICKSBURG LOESS



NOTES: FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGES: 10 — 0

LEGEND
——— DATA FROM TEST
- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES

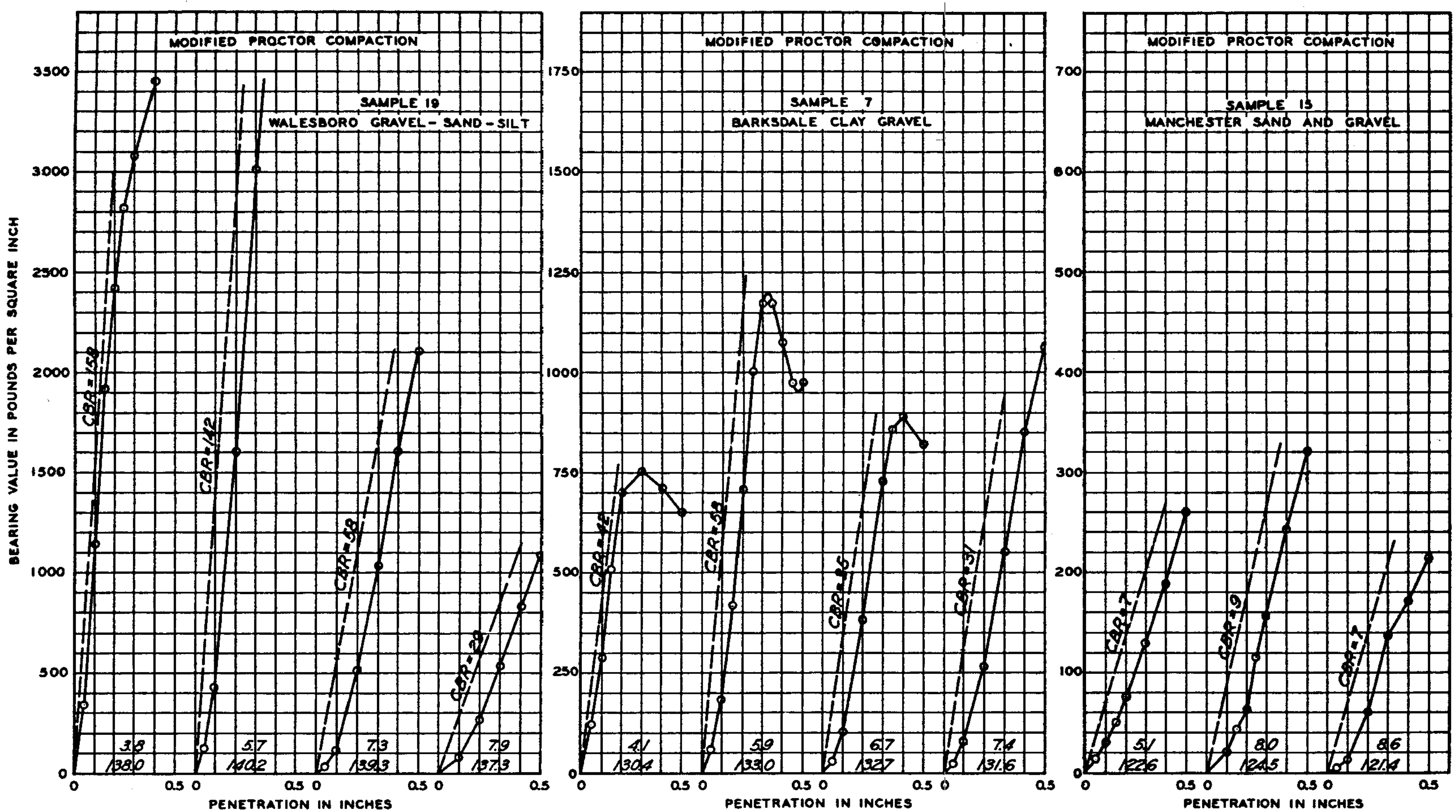
SAMPLE 3 VICKSBURG LOESS

NOTES: SPECIMENS SOAKED FROM TOP AND BOTTOM
FIGURES AT BOTTOM OF CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGES: 10-0

LEGEND
——— DATA FROM TEST
- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES
SAMPLE 5 DE RIDDER SAND CLAY (LOAM)

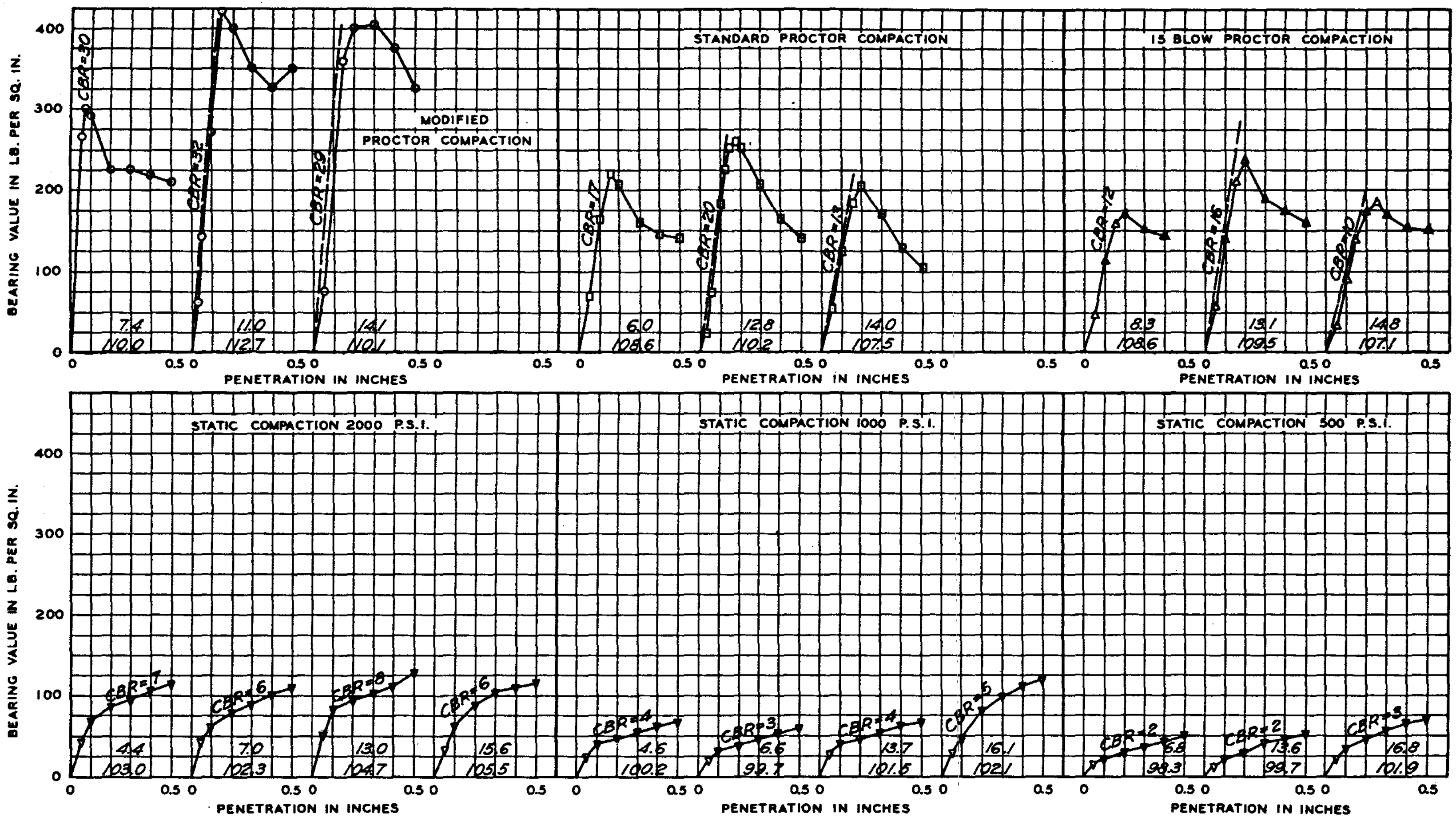


NOTES: ALL SPECIMENS SOAKED FROM BOTTOM ONLY
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGES: 10-10

LEGEND
——— DATA FROM TEST
- - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES
SAMPLES 19, 7, AND 15



NOTES: ALL SPECIMENS SOAKED TOP AND BOTTOM.
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGES: 10-10

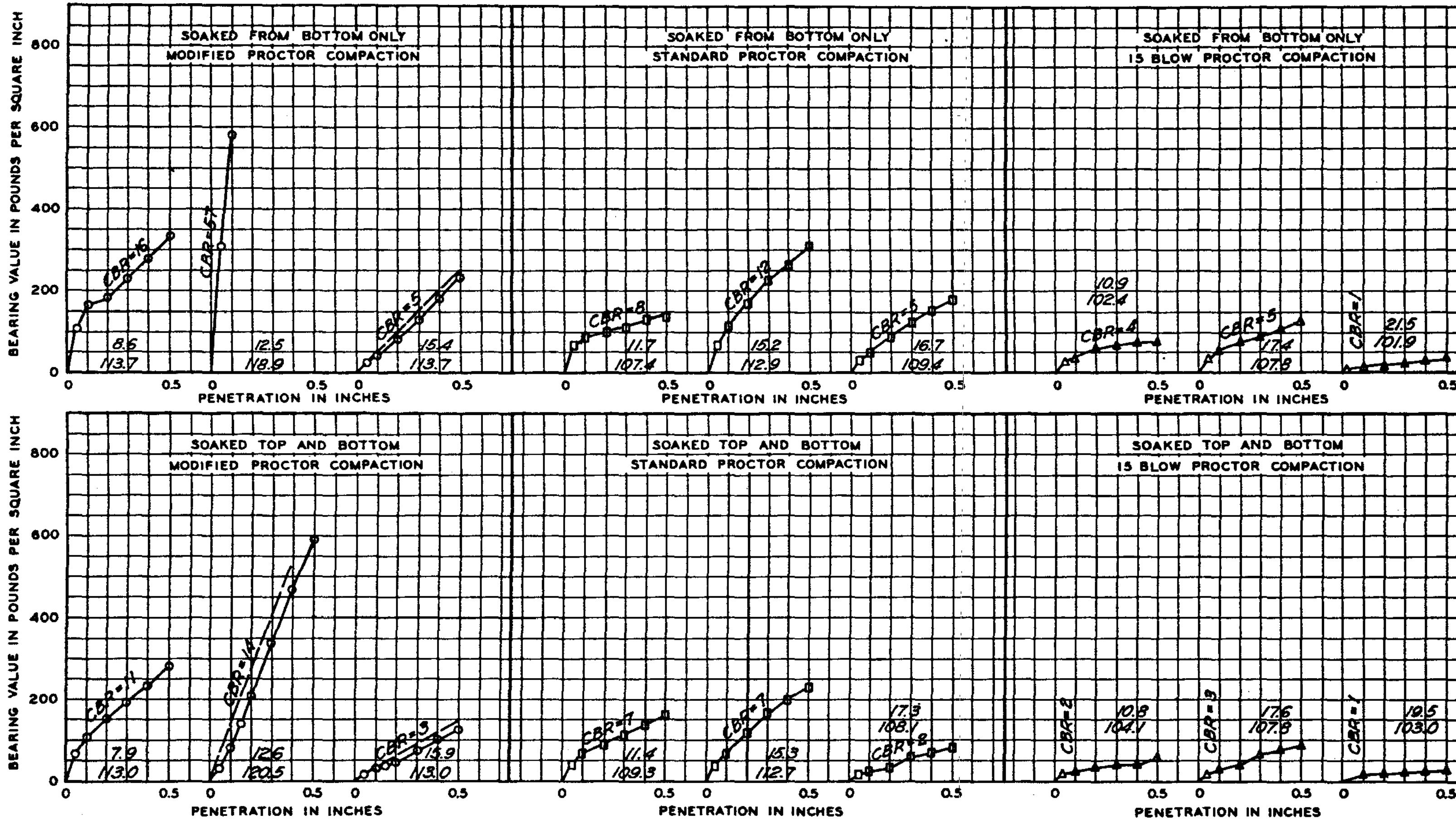
LEGEND

——— DATA FROM TEST
- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES

SAMPLE 10 EGLIN FIELD SAND



NOTES: FIGURES AT BOTTOM OF CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGES: 10-0

LEGEND

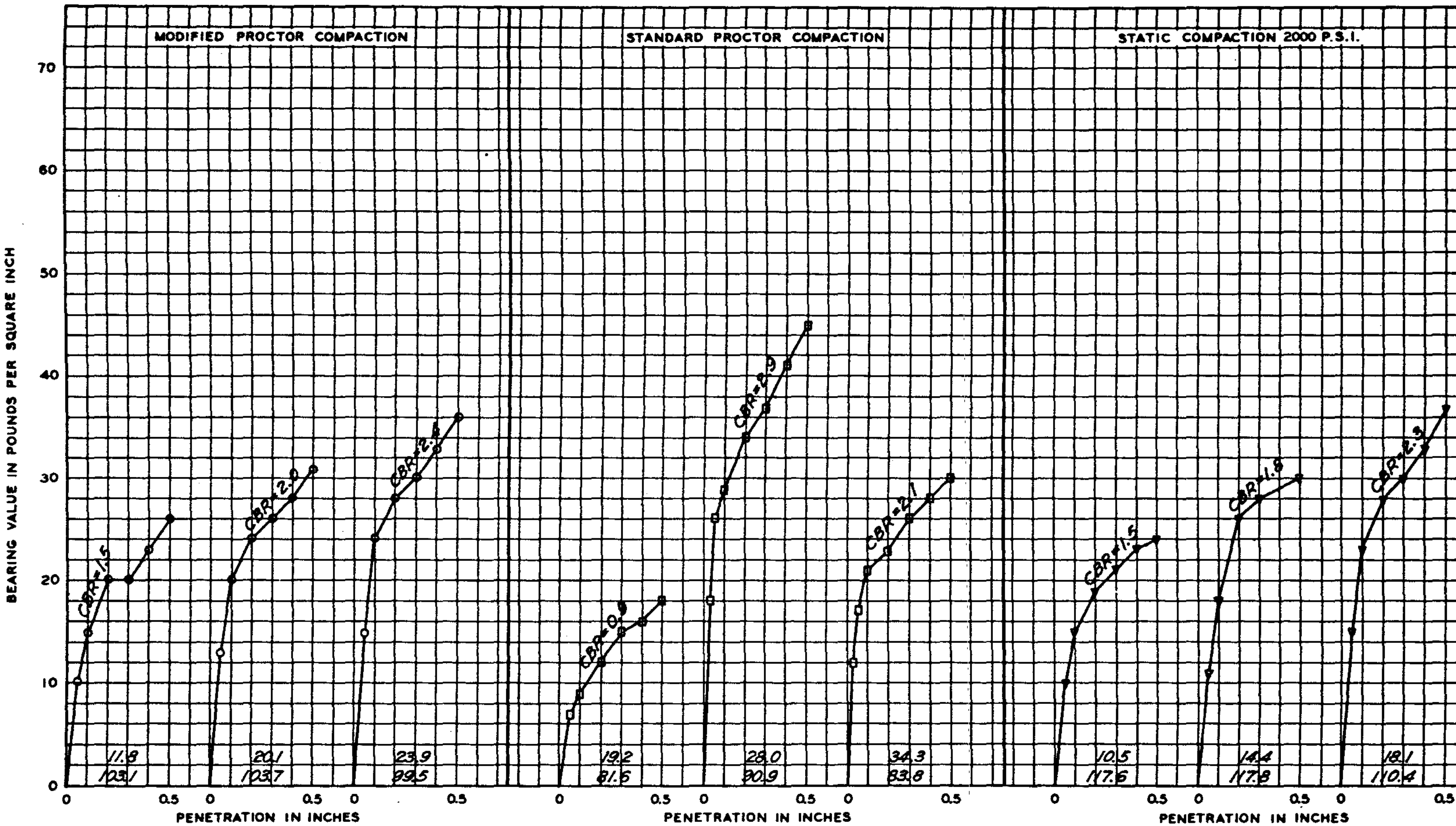
——— DATA FROM TEST

- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES

SAMPLE 18 BERRY FIELD LEAN CLAY



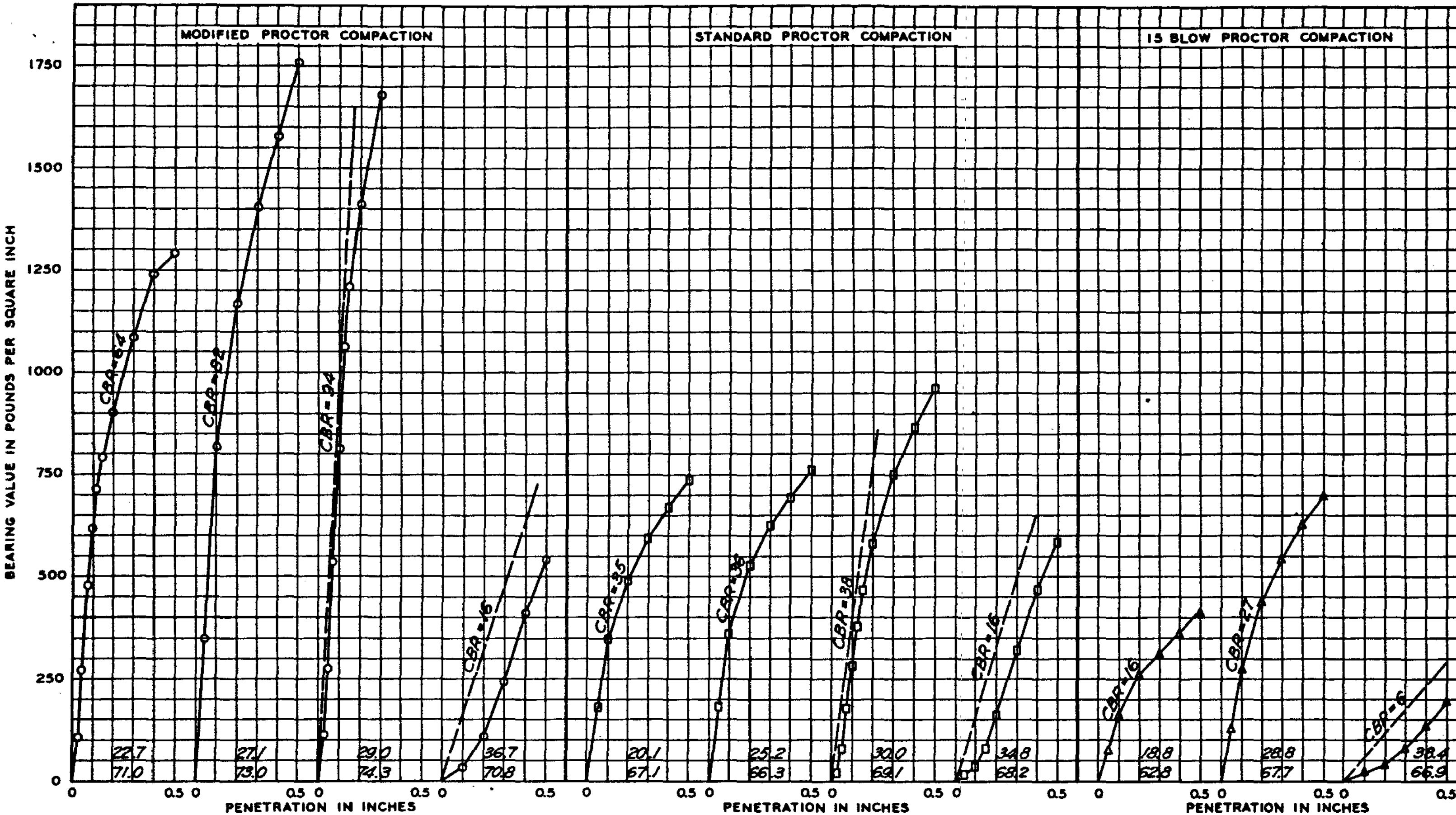
NOTES: ALL SPECIMENS SOAKED TOP AND BOTTOM.
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
NO CORRECTION REQUIRED ON ANY TEST.
SURCHARGES: 10-0.

LEGEND
——— DATA FROM TEST
- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES

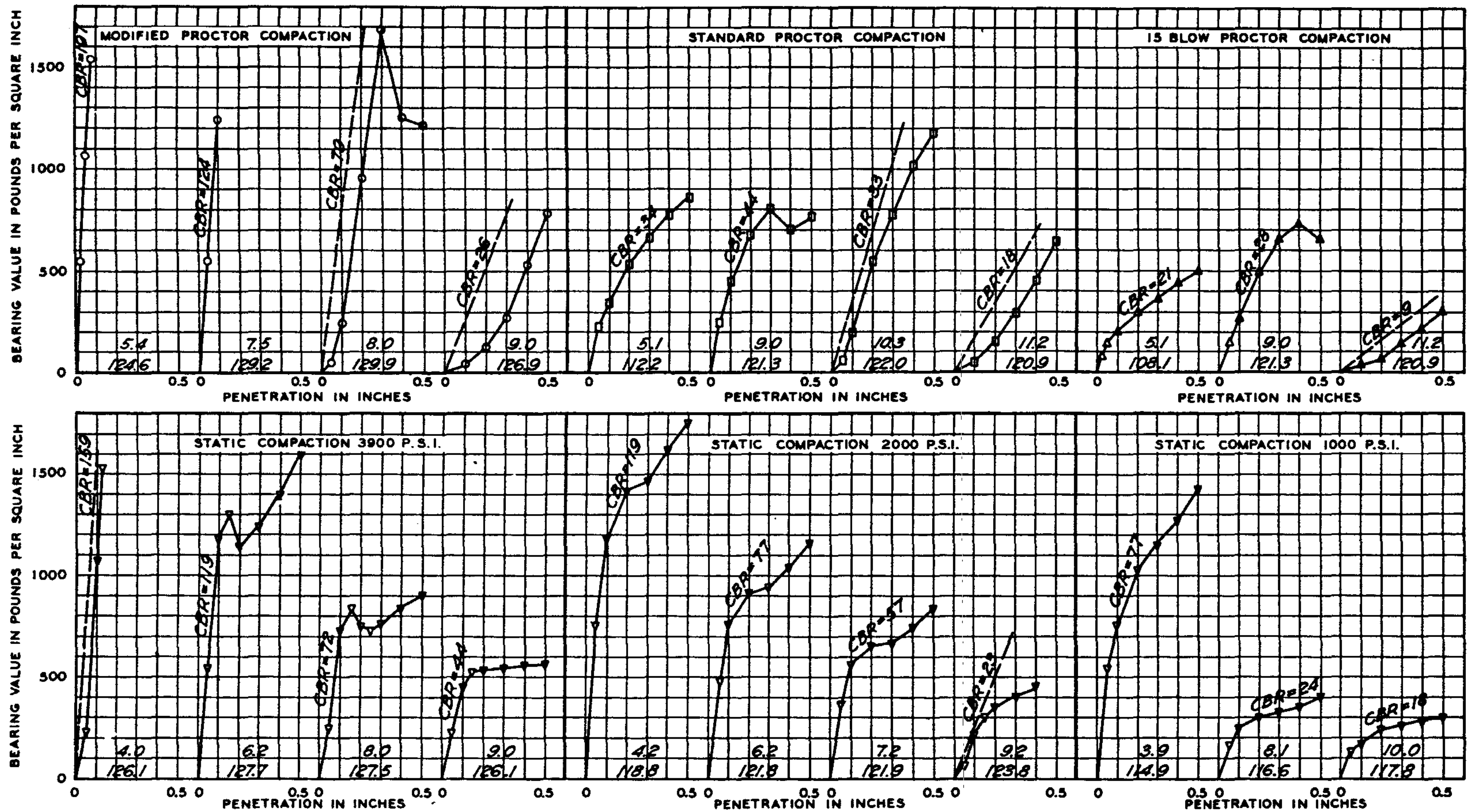
SAMPLE 20 STOCKTON. ADOBE CLAY



NOTES: SPECIMENS SOAKED FROM BOTTOM ONLY.
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGE: 10-10

LEGEND
——— DATA FROM TEST
- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION
TYPICAL CBR PENETRATION CURVES
SAMPLE 21 CALIFORNIA PUMICE



MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES

COMPARISON OF PENETRATION CURVES OF DYNAMICALLY COMPACTED WITH STATICALLY COMPACTED SPECIMENS
SAMPLE 23 VICKSBURG CLAY SAND

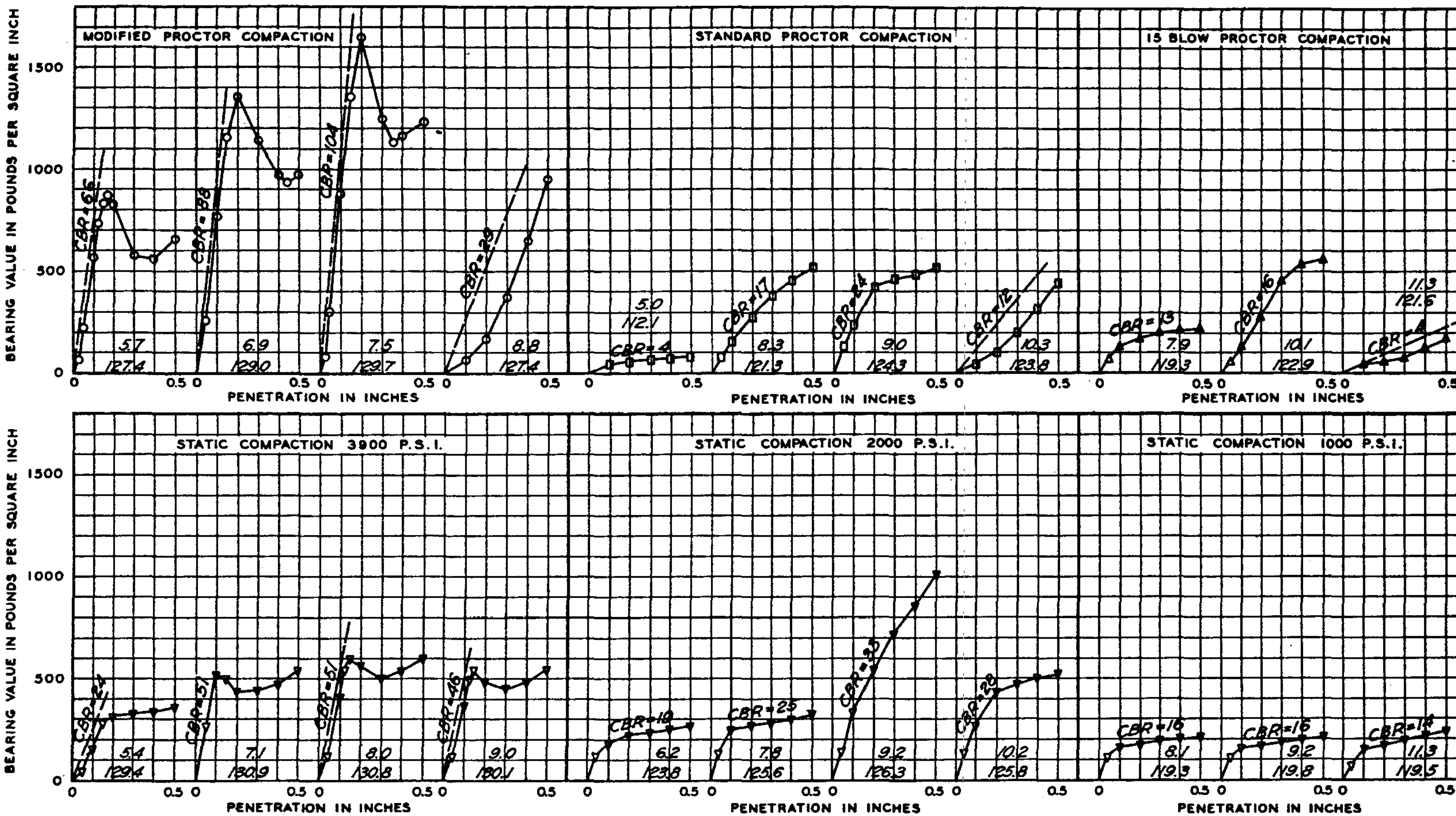
NOTES: TESTED AS MOLDED.
FIGURES AT BOTTOM OF CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
PENETRATION SURCHARGE = 10 LBS

LEGEND

——— DATA FROM TEST
- - - - - CORRECTED CBR CURVE

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



NOTES: SPECIMENS SOAKED FROM TOP AND BOTTOM.
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGES: 10 – 10.

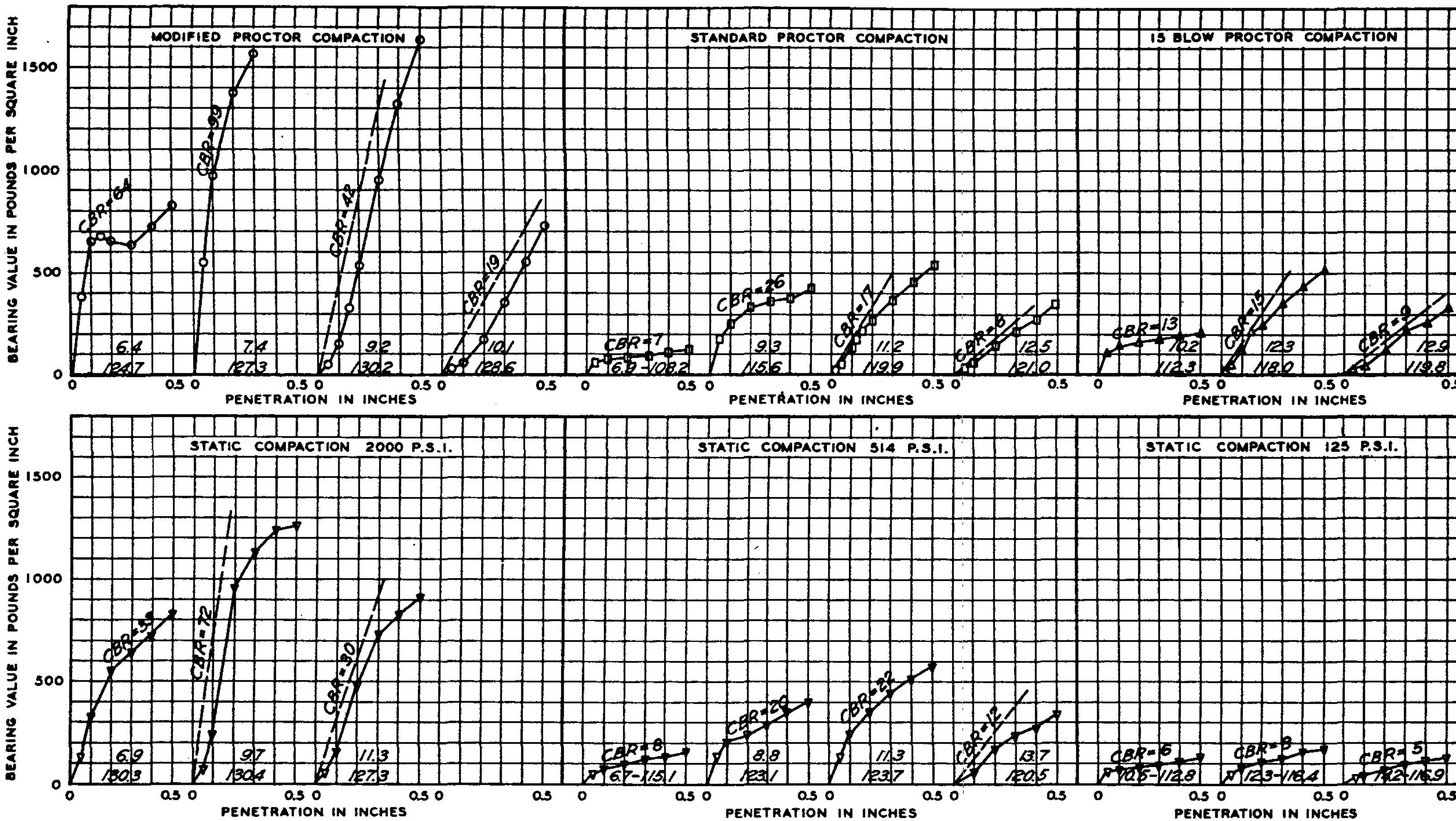
LEGEND

——— DATA FROM TEST

- - - - - CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES
SAMPLE 23 VICKSBURG CLAY SAND

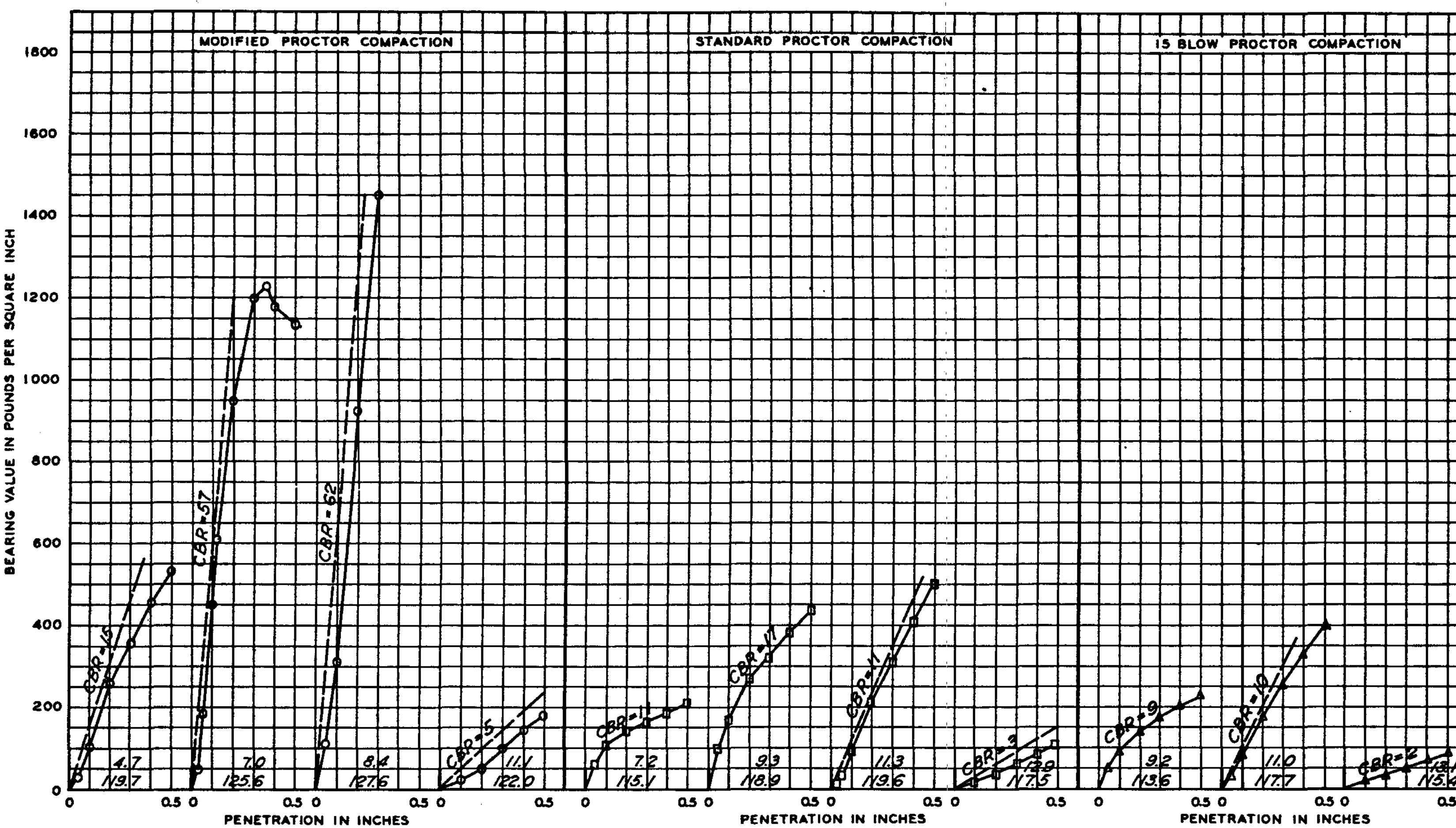


NOTES: ALL SPECIMENS SOAKED FROM TOP AND BOTTOM.
FIGURES AT BOTTOM OF CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGES : 10 – 0.

LEGEND
——— DATA FROM TEST
------- CORRECTED CBR CURVE

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES
MARIETTA MIX GEORGIA CLAY SAND



NOTES: SPECIMENS SOAKED FROM TOP AND BOTTOM.
FIGURES AT BOTTOM OF CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.
SURCHARGE: 10 – 10

LEGEND
——— DATA FROM TEST
- - - - - CORRECTED CBR CURVE

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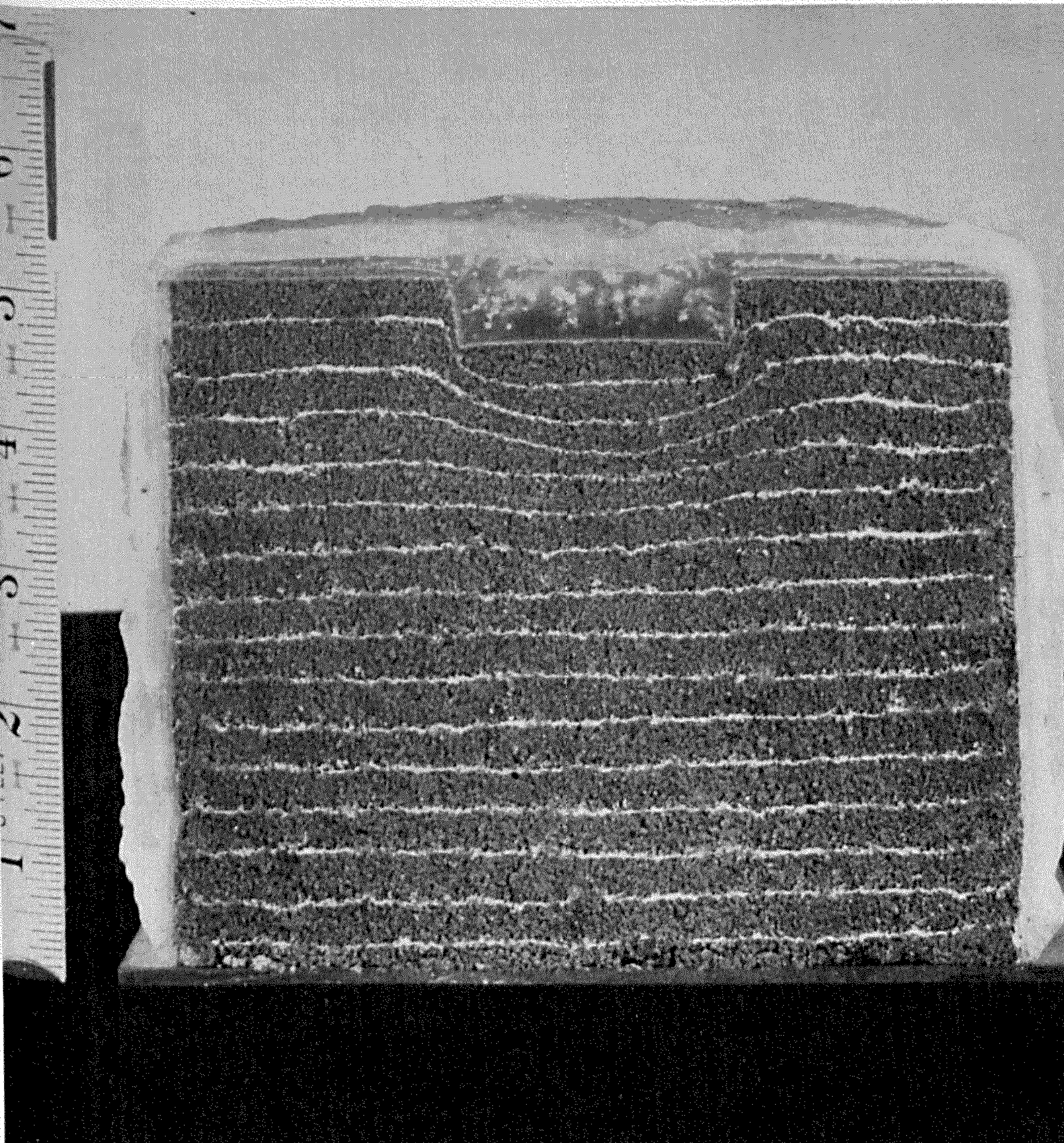
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

TYPICAL CBR PENETRATION CURVES
KELLOGG FIELD CLAY SAND



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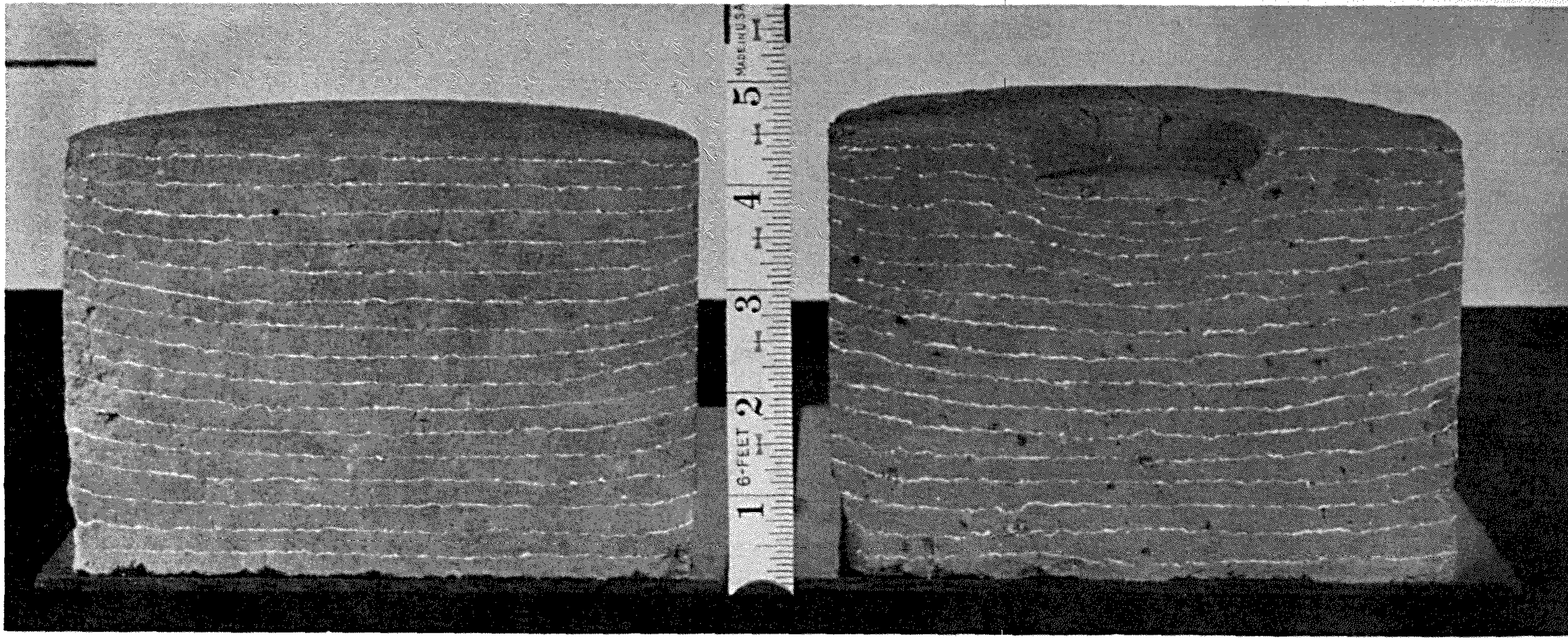
California Bearing Ratio Investigation

Test 1 Sample 10 Eglin Field Sand

DEPTH OF SAMPLE AFFECTED BY PENETRATION PISTON

Specimen compacted in CBR mold at 12% water content in 16 layers; 6 blows of spacing piston per layer. Thin layer of chalk dust placed between each layer. Molded dry density = 106.6 lb per cu ft (modified Proctor dry density = 112.5 lb per cu ft at 12% water content). Specimen soaked from bottom until free water on top. Top of specimen as compacted was penetrated. Surcharge 10-10. Corrected CBR = 7. After penetration, specimen was pushed from mold and coated with wax, then cut in half on band saw.



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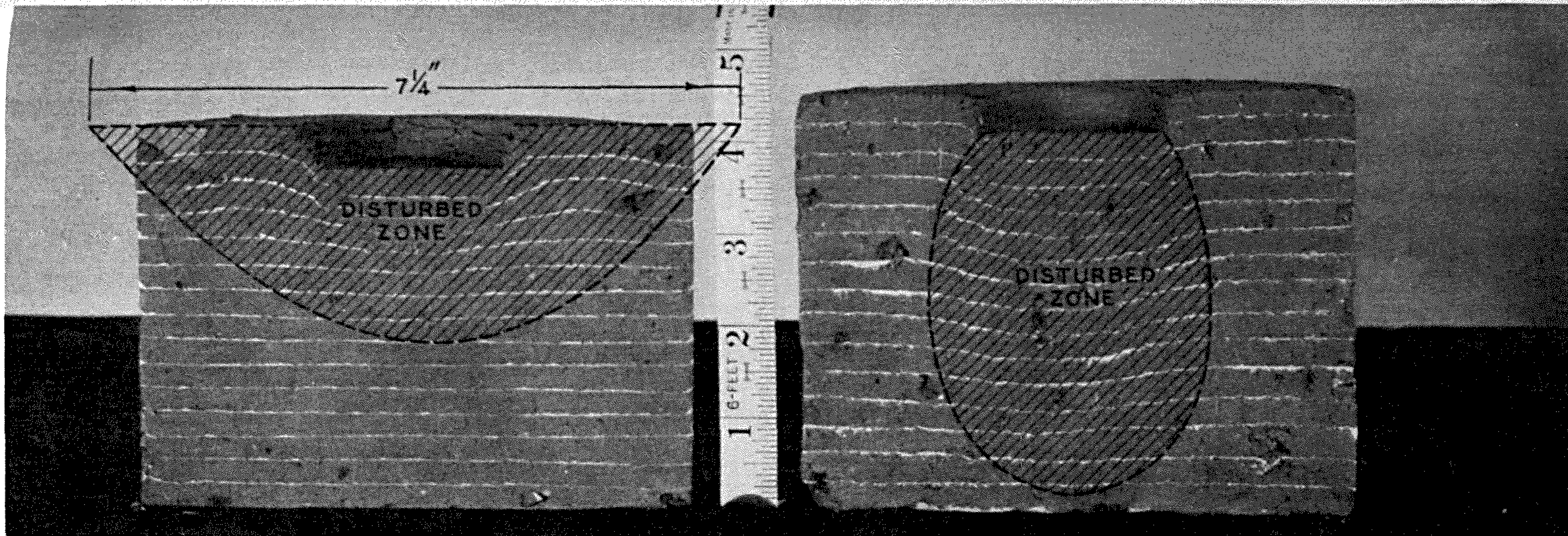
California Bearing Ratio Investigation

Test 1 Sample 3 Vicksburg Loess

DEPTH OF SAMPLE AFFECTED BY PENETRATION PISTON

Specimen compacted in CBR mold at 8% water content in 16 layers; spacing piston placed on top of each layer and given 2 blows with 5-1/2-lb hammer. Thin layer of chalk dust placed between each layer. Whole specimen then subjected to 2000 lb per sq in. static pressure. Molded dry density = 104.9 lb per cu ft (modified Proctor dry density = 111 lb per cu ft at 14.5% water content). Duplicate specimens were molded to obtain picture before and after penetration. Penetrated specimen was first soaked from bottom until free water on top. Water content after soaking = 25.9%. Surcharge 10-10. Corrected CBR = 12. Top of specimen as compacted was penetrated. After testing, penetrated specimen was pushed from mold and cut in half on band saw.



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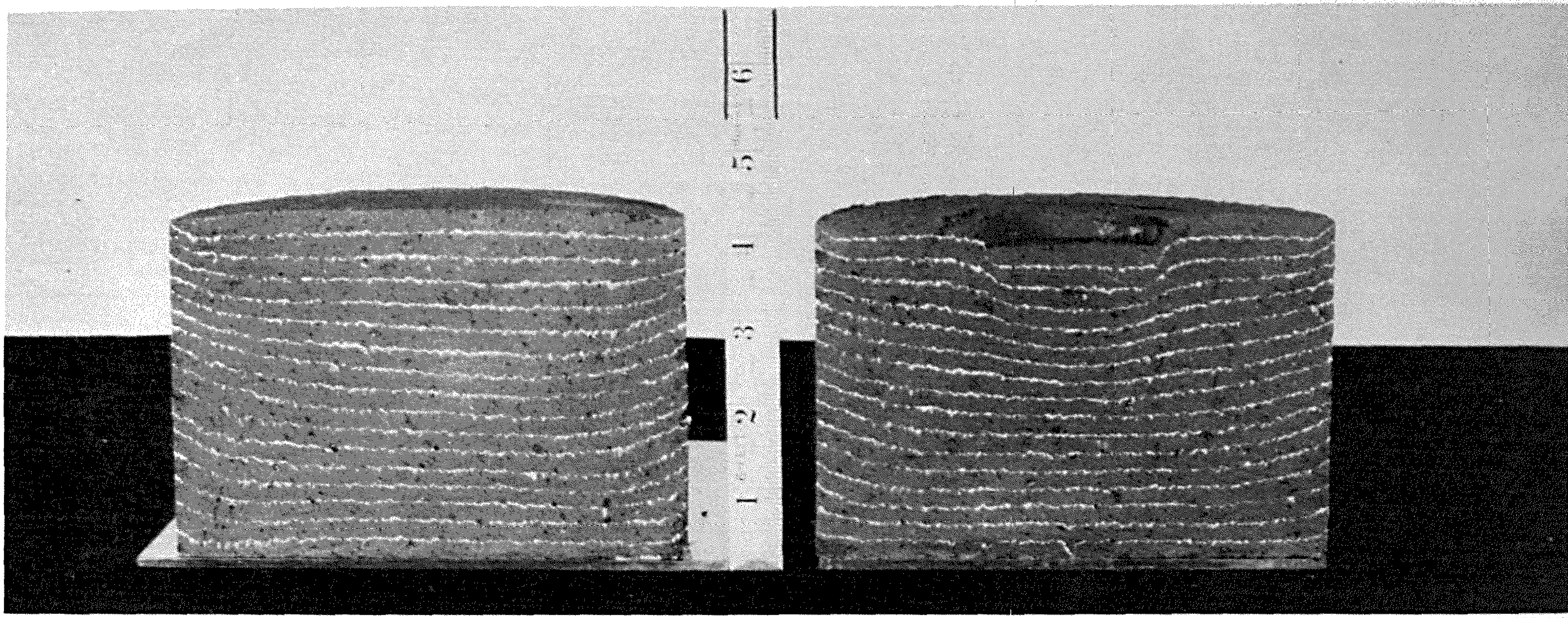
California Bearing Ratio Investigation

Sample 3 Vicksburg Loess

PORTION OF SAMPLE AFFECTED BY PENETRATION PISTON AND MOLD

<u>Test 2</u>. Specimen compacted at 8% water content in CBR mold in 15 layers; 2000 lb per sq in. static load per layer. Thin layer of chalk dust placed between each layer. Molded dry density = 110.7 lb per cu ft. Specimen soaked from bottom until free water on top. Water content after soaking = 18.5%. Surcharge 10-10. Corrected CBR = 19. Top of specimen as compacted was penetrated. After penetration specimen was pushed from mold and cut in half on band saw.

<u>Test 3</u>. Specimen compacted at 8% water content in CBR mold in 15 layers. Spacing piston placed on top of each layer and given 5 blows with 10-lb hammer. Thin layer of chalk dust placed between each layer. Molded dry density = 94.7 lb per cu ft. Specimen soaked from bottom until free water on top. Water content after soaking = 21%. Surcharge 10-10. Corrected CBR = 2. Top of specimen as compacted was penetrated. Specimen pushed from mold after penetration and cut in half on band saw.



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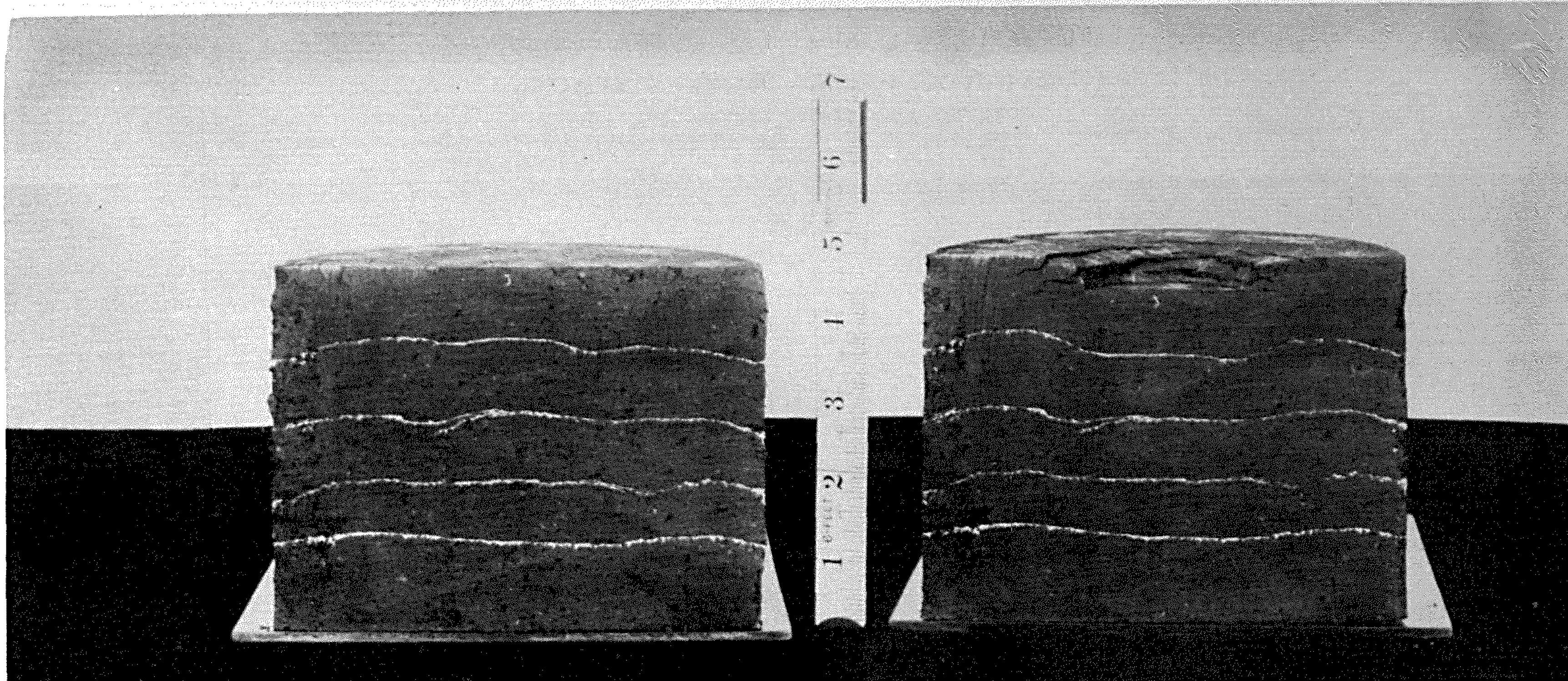
Test 1 Sample 18 Berry Field Lean Clay

DEPTH OF SAMPLE AFFECTED BY PENETRATION PISTON

Specimen compacted in CBR mold at 18% water content in 16 layers. Spacing piston placed on top of each layer and given 2 blows with 5-1/2-lb hammer. Thin layer of chalk dust placed between each layer. Whole specimen then subjected to 1350 lb per sq in. static pressure. (Free water appeared at this load.) Molded dry density = 111.8 lb per cu ft (modified Proctor dry density = 119.5 lb per cu ft at 12.5% water content). Duplicate specimens molded to obtain picture before and after penetration. Specimen was penetrated as compacted. Top of specimen as compacted was penetrated. Penetration surcharge = 10 lb. Corrected CBR = 7. After testing, penetrated specimen was pushed from mold and cut in half on band saw.



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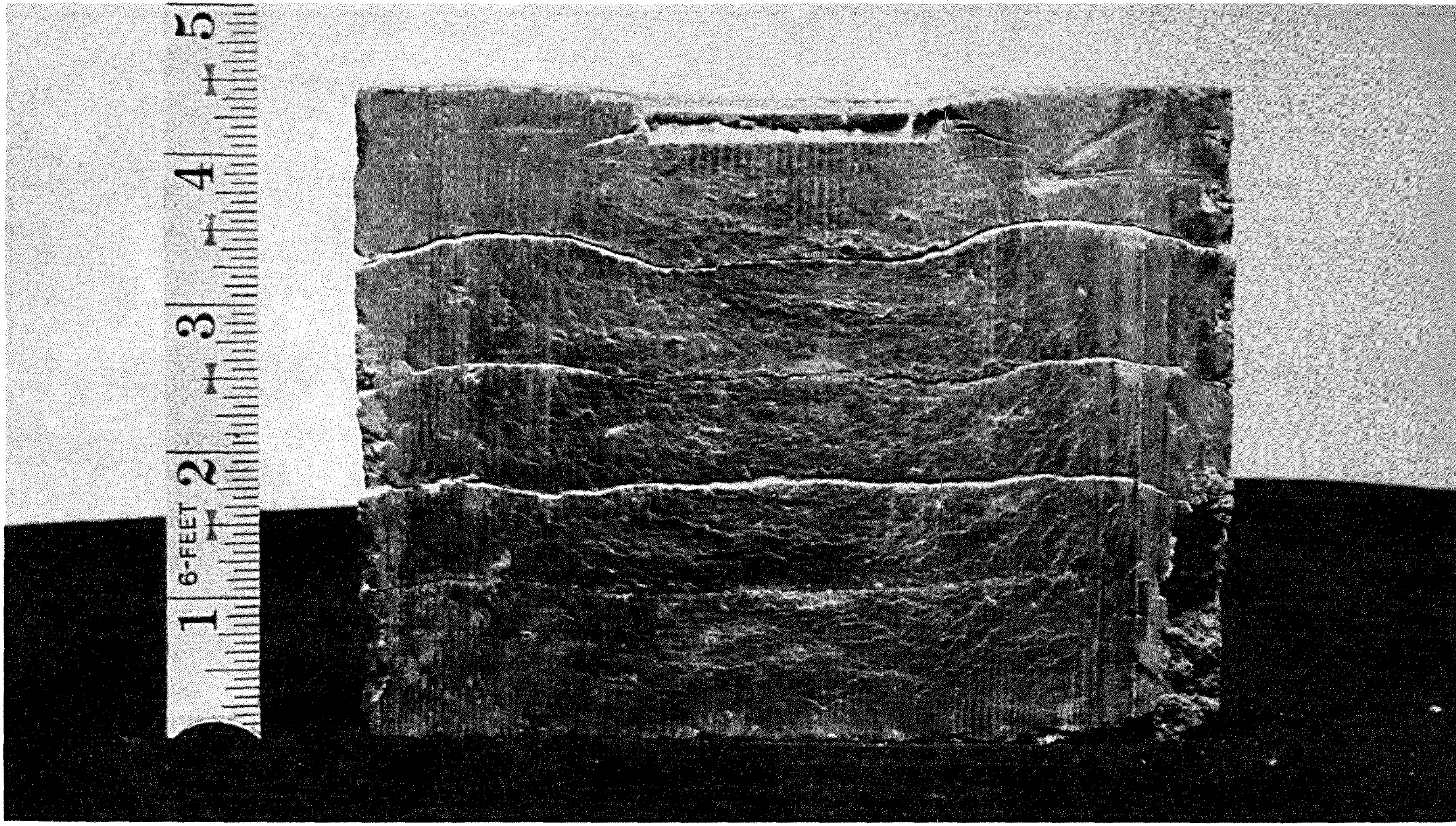
Test 2 Sample 18 Berry Field Lean Clay

DEPTH OF SAMPLE AFFECTED BY PENETRATION PISTON

Specimen compacted in CBR mold at 15.5% water content in 5 layers; 55 blows per layer with 10-lb hammer. Thin layer of chalk dust placed between each layer. Molded dry density = 115.5 lb per cu ft. After compaction, specimen was pushed from mold, cut in half on band saw and photographed. Specimen was then carefully put back in mold and penetrated without soaking. Top of specimen as penetrated was bottom as compacted. Penetration surcharge = 10 lb. Corrected CBR = 6. After penetration, specimen was pushed from mold and photographed.

PLATE 58



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California Bearing Ratio Investigation

Test 2 Sample 18 Berry Field Lean Clay

DEPTH OF SAMPLE AFFECTED BY PENETRATION PISTON

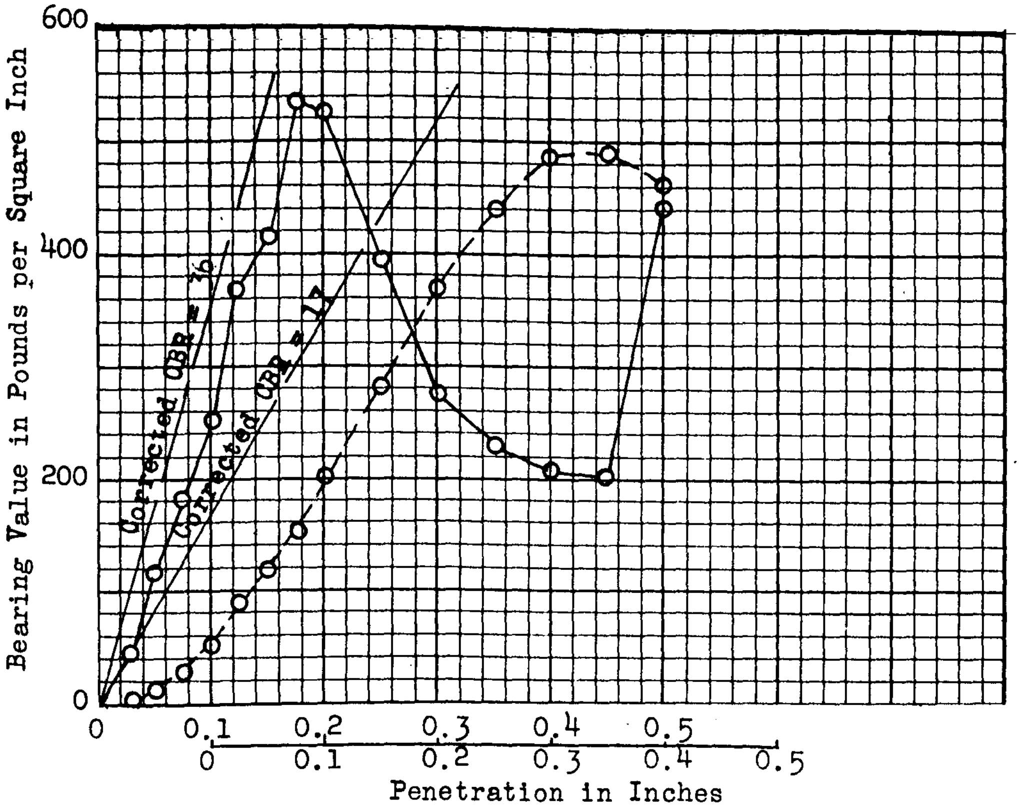
Specimens soaked top and bottom

Legend

——6-in. diam. mold, 2-in. diam. penetration piston
Specimen height, 4.97 in., surcharges*10-10.
As molded: w = 11.2%, γ_d = 112.0 lb/cu ft.
Soaking: w = 13.1%, γ_d = 112.0 lb/cu ft.

----12-in. diam. mold, 4-in. diam. penetration piston.
Specimen height, 5.95 in., surcharges *40-40.
As molded: w = 11.8, γ_d = 112.1 lb/cu ft.
Soaked: w = 12.3, γ_d = 112.1 lb/cu ft.

*10-lb surcharge in 6-in. mold is equivalent to 40-lb surcharge in 12-in. mold.



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California Bearing Ratio Investigation

PENETRATION AT MODIFIED PROCTOR DENSITY

(Comparison of CBR in 6 and 12-Inch Molds)

Sample 10 Eglin Field Sand

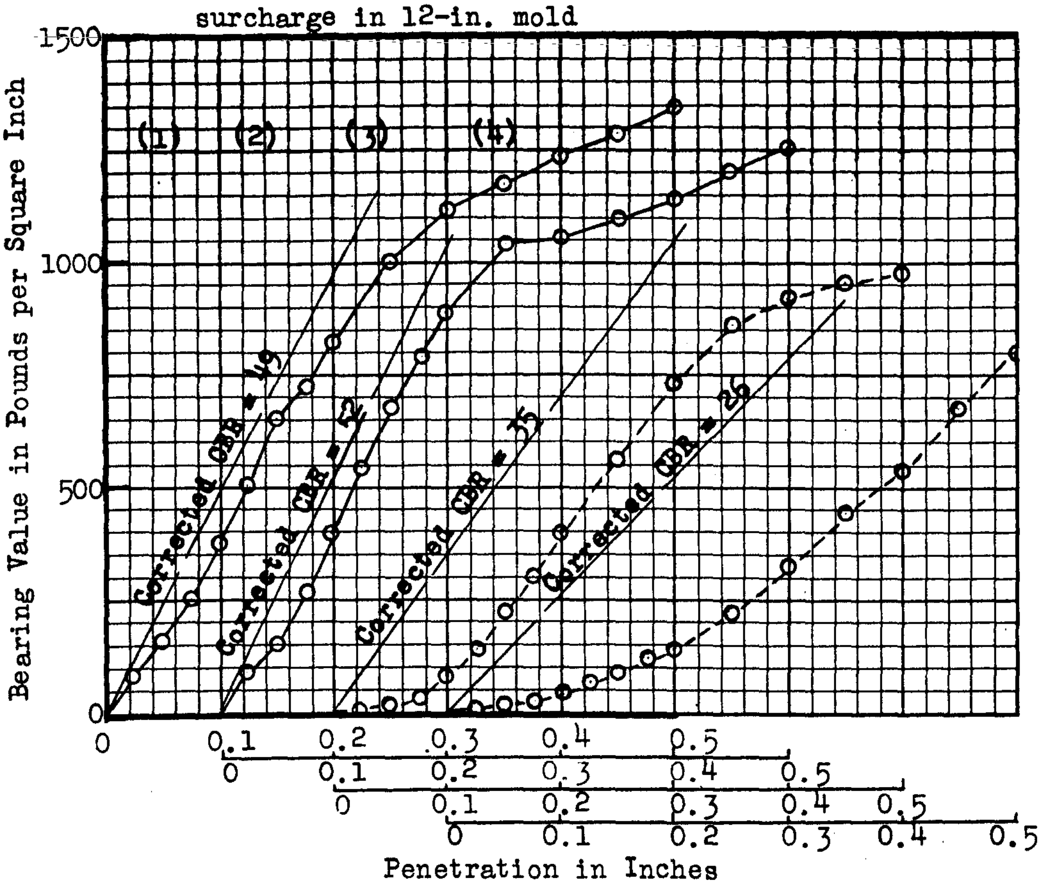
Specimens soaked from bottom only

Legend

—— 6-in. dia. mold, 2-in. dia. penetration piston
---- 12-in. dia. mold, 4-in. dia. penetration piston

Specimen	Height Inches	Molded w	Molded γ_d	Soaked w	Soaked γ_d	*Surcharges
1	4.50	15.5	111.8	17.6	110.5	10-10
2	4.50	15.5	111.2	17.7	111.0	10-10
3	6.05	13.0	113.0	17.2	112.5	40-40
4	6.17	15.0	112.8	16.8	112.4	40-40

*10-lb surcharge in 6-in. mold equivalent to 40-lb surcharge in 12-in. mold



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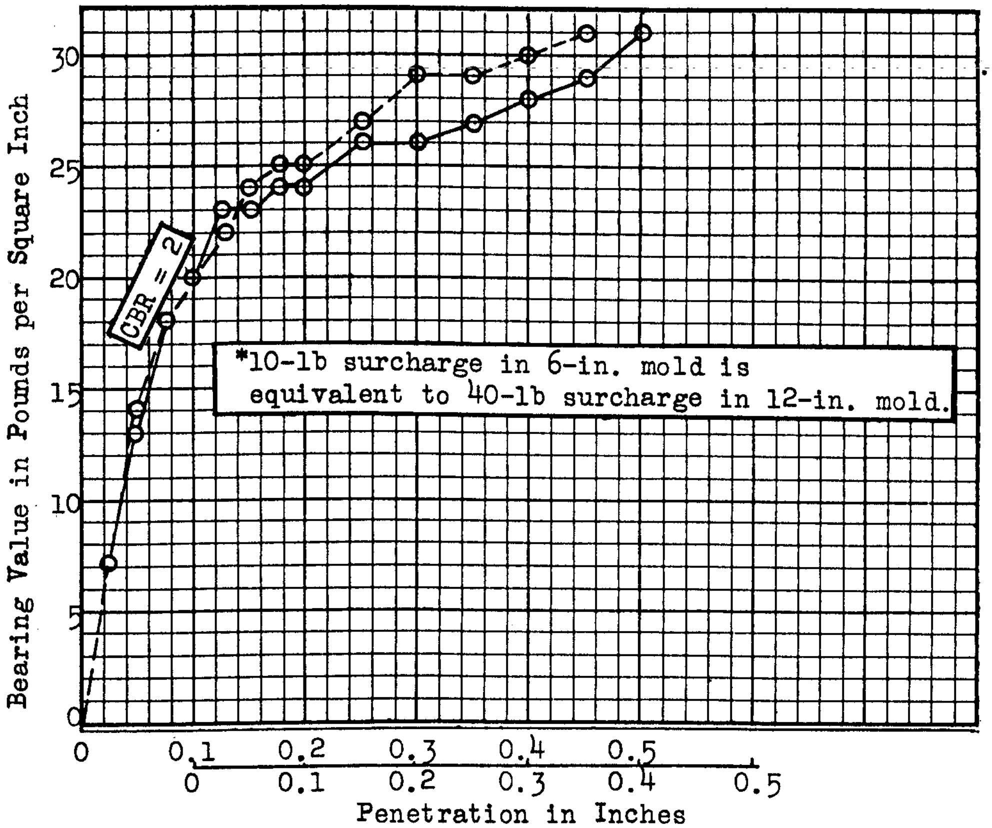
PENETRATION AT MODIFIED PROCTOR DENSITY
Comparison of CBR in 6- and 12-Inch Molds
Sample 3 Vicksburg Loess

Specimens soaked top and bottom

Legend

——— 6-in. diam. mold, 2-in. diam. penetration piston.
Specimen height, 4.85 in.
Surcharges *10-0
As molded: w = 20.1, γ_d = 103.7 lb/cu ft
Soaked: w = 25.7, γ_d = 96.4 lb/cu ft

- - - 12-in. diam. mold, 4-in. diam. penetration piston.
Specimen height, 6.60 in.
Surcharges *40-0
As molded: w = 21.8, γ_d = 104.7 lb/cu ft
Soaked: w = 27.6, γ_d = 96.5 lb/cu ft



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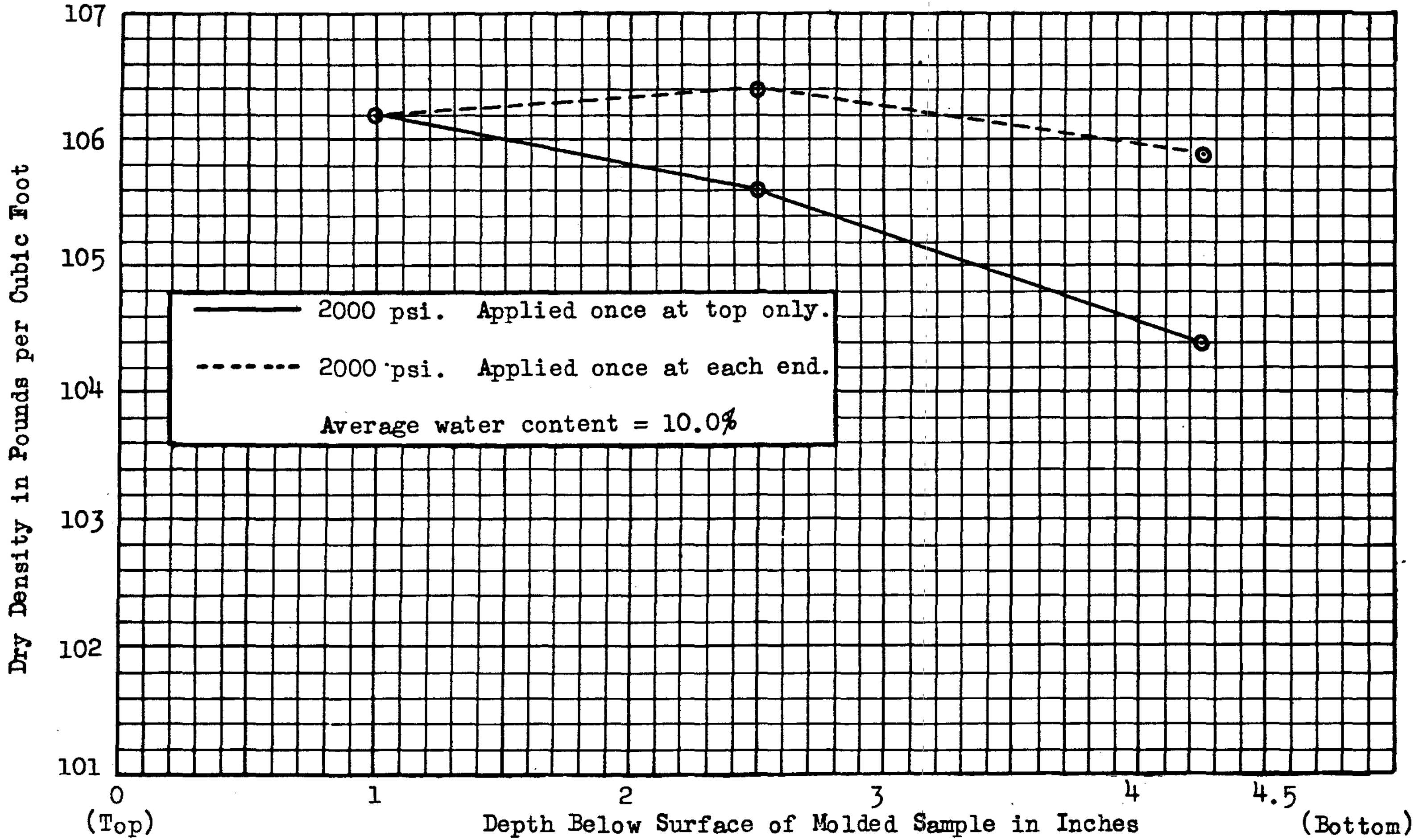
California Bearing Ratio Investigation

PENETRATION AT MODIFIED PROCTOR DENSITY

(Comparison of CBR in 6 and 12-Inch Molds)

Sample 20 Stockton Adobe Clay



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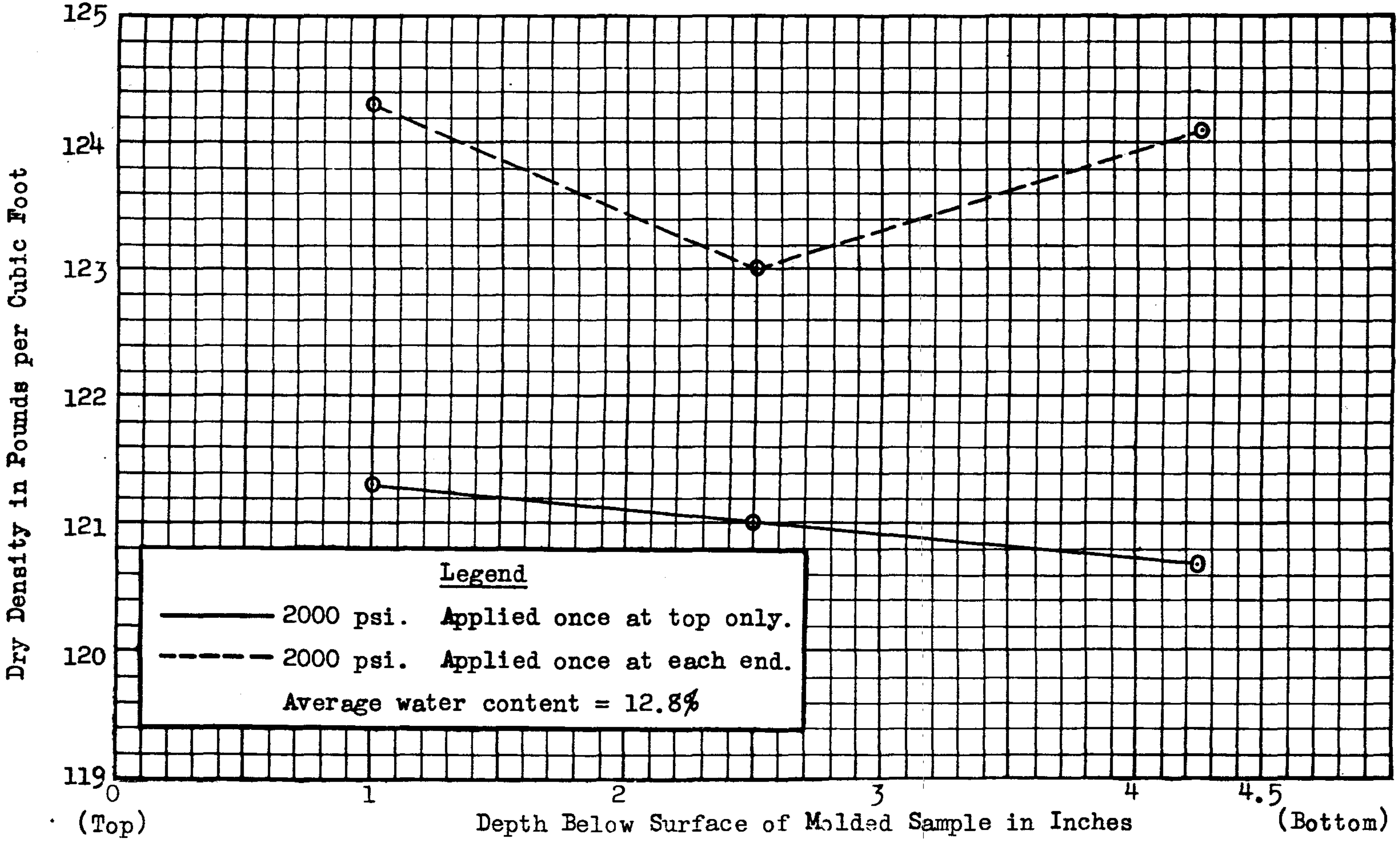
Sample 10 DENSITY GRADIENT IN CBR MOLD Eglin Field Sand



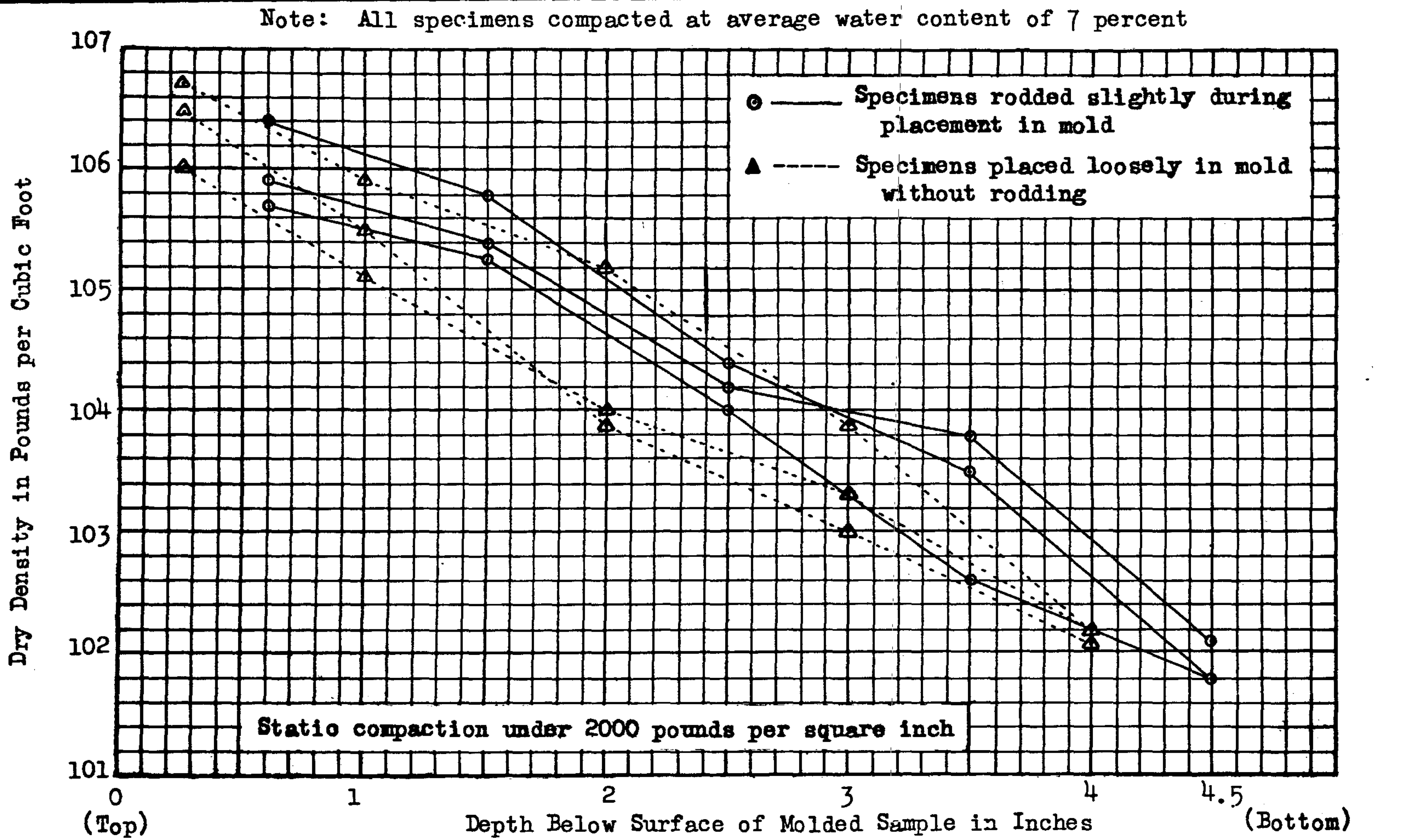
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

Sample 18

DENSITY GRADIENT IN CBR MOLD

Berry Field Lean Clay

WAR DEPARTMENT CORPS OF ENGINEERS, U.S. ARMY



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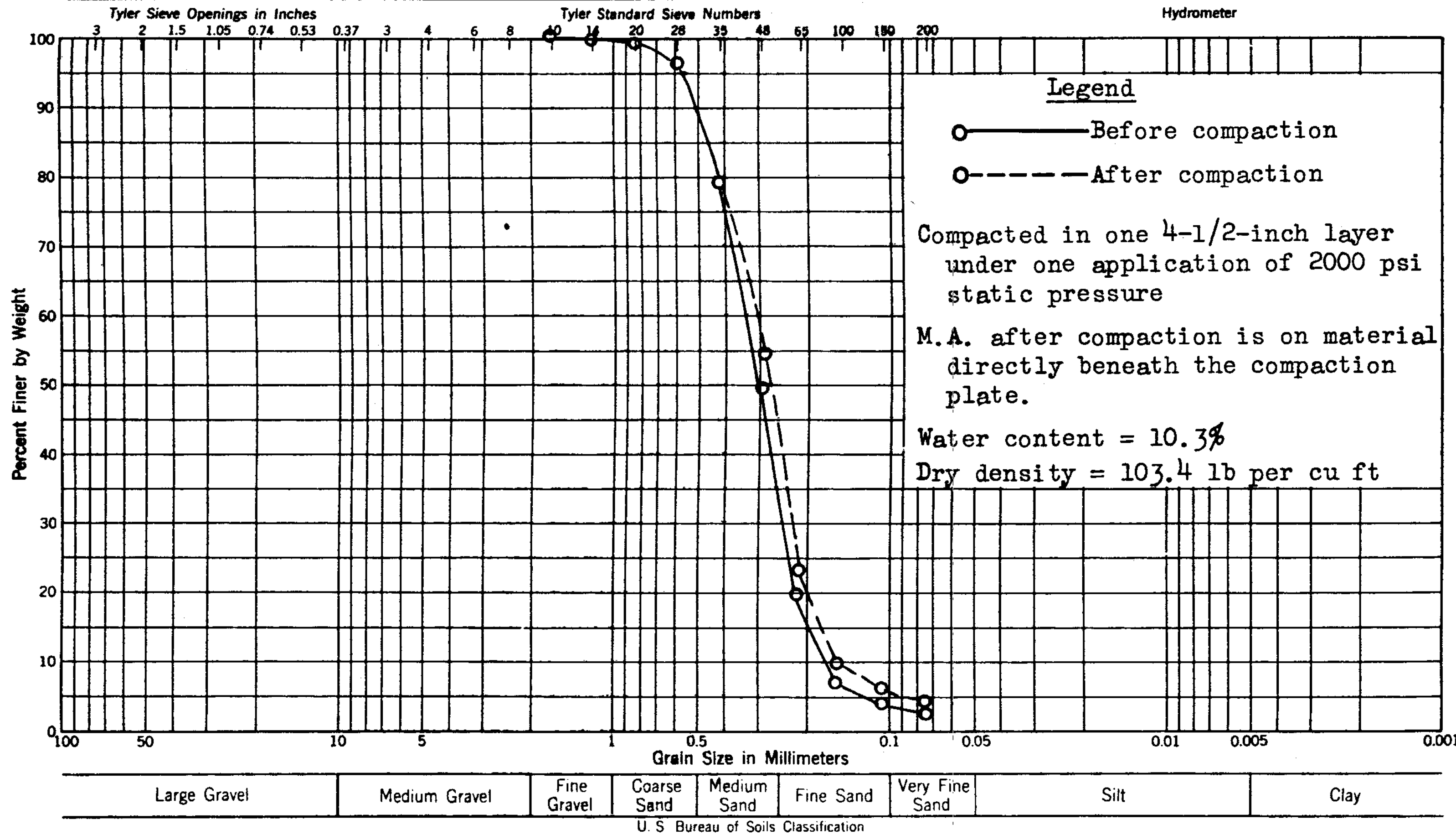
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

Sample 3 DENSITY GRADIENT IN CBR MOLD Vicksburg Loess

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

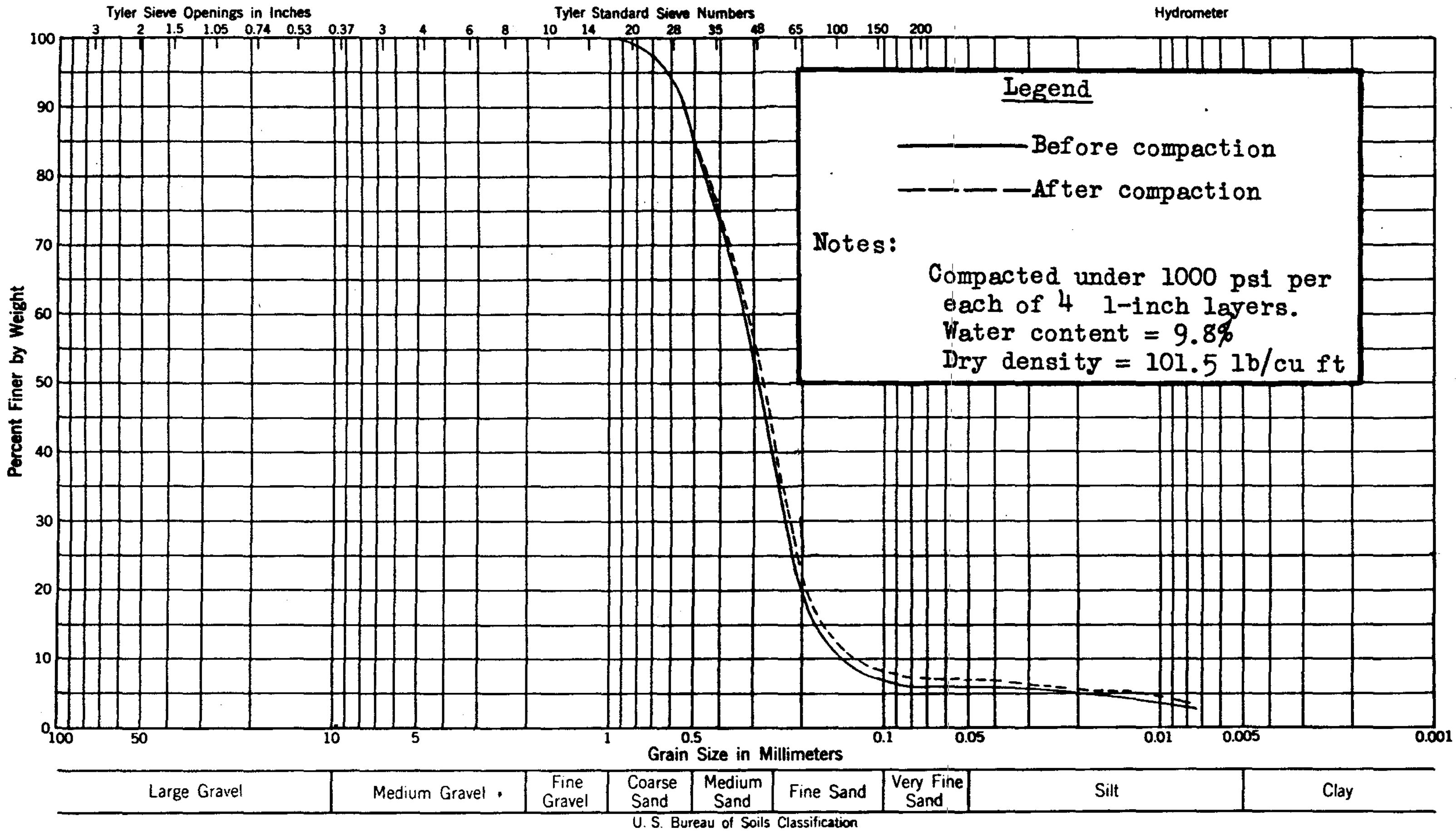
GRAIN SIZE DISTRIBUTION BEFORE AND AFTER APPLICATION OF STATIC LOAD

Sample 10 Eglin Field Sand

PLATE 65

PLATE 66

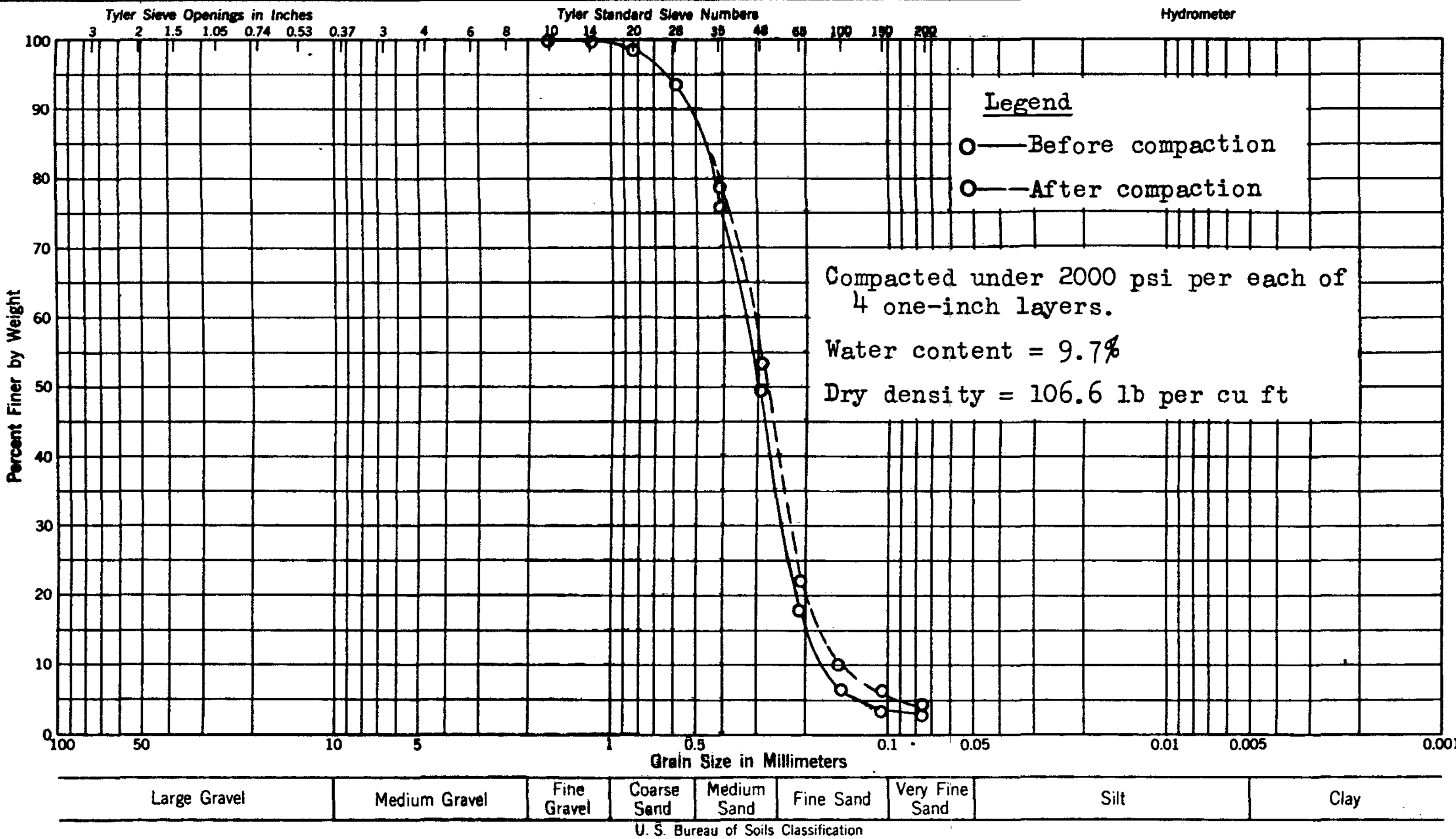
WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

GRAIN SIZE DISTRIBUTION BEFORE AND AFTER APPLICATION OF STATIC LOAD

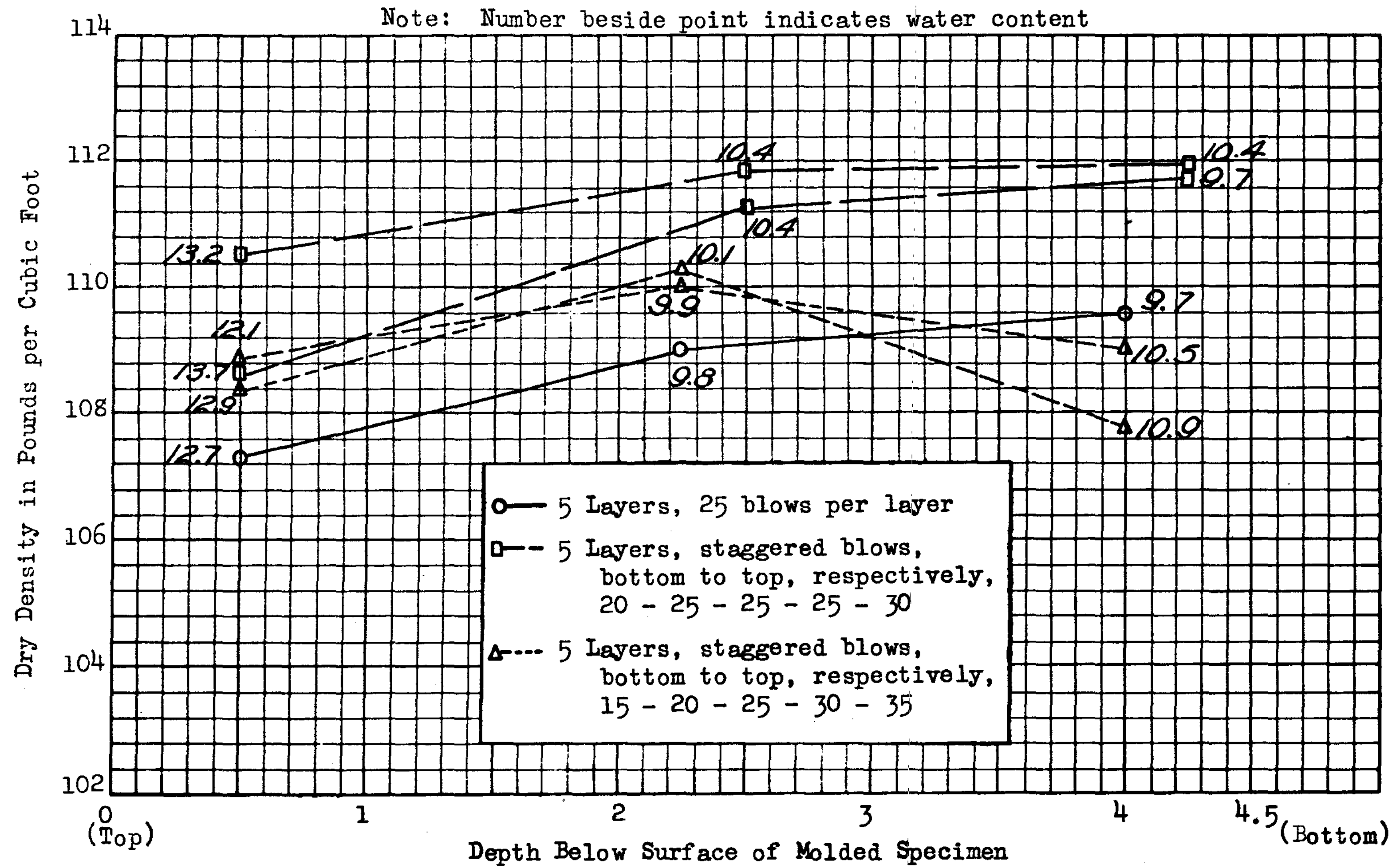
Sample 10 Eglin Field Sand



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

GRAIN SIZE DISTRIBUTION BEFORE AND AFTER APPLICATION OF STATIC LOAD

Sample 10 Eglin Field Sand



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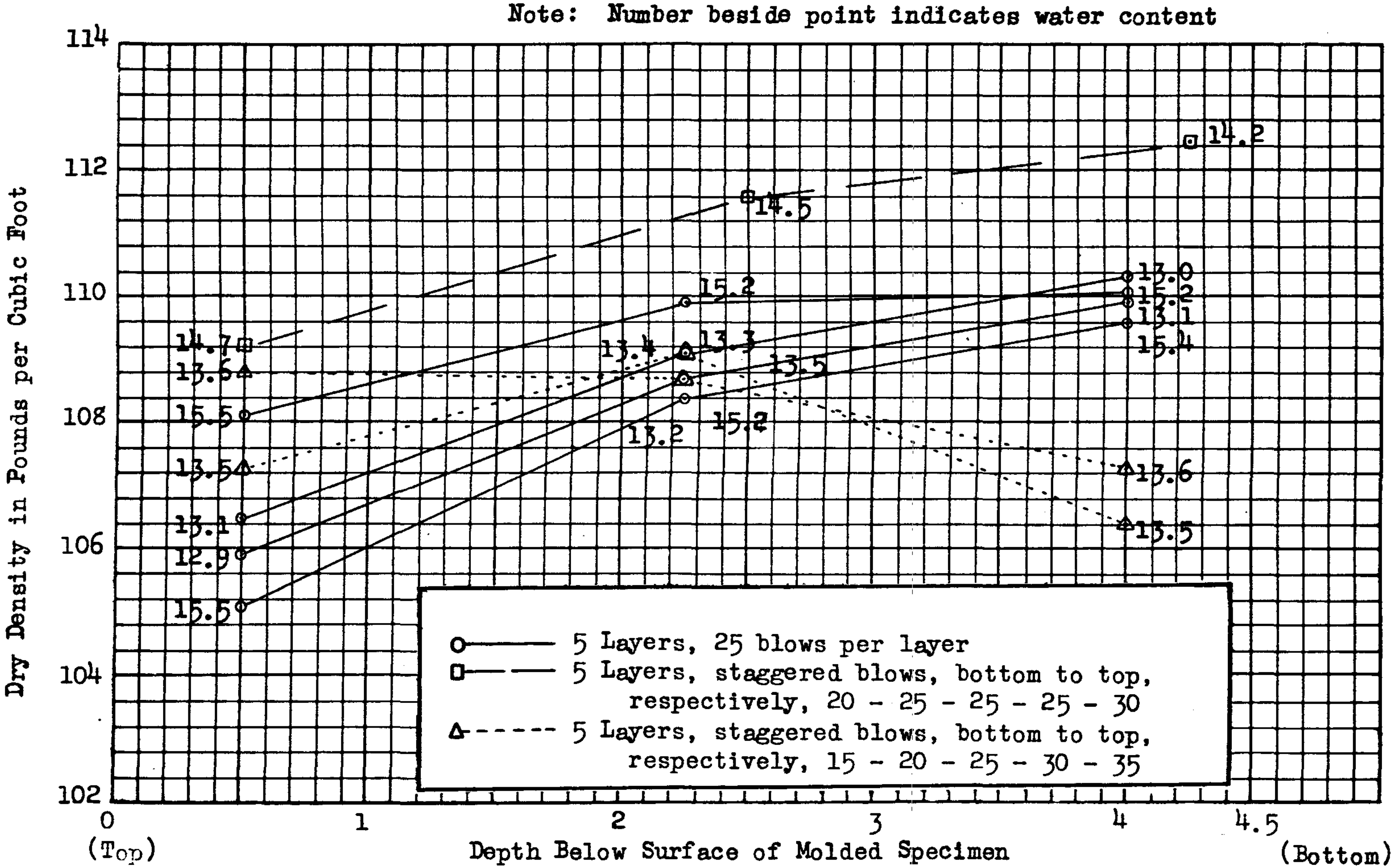
California Bearing Ratio Investigation

Sample 10 — DENSITY GRADIENT IN PROCTOR MOLD — Eglin Sand



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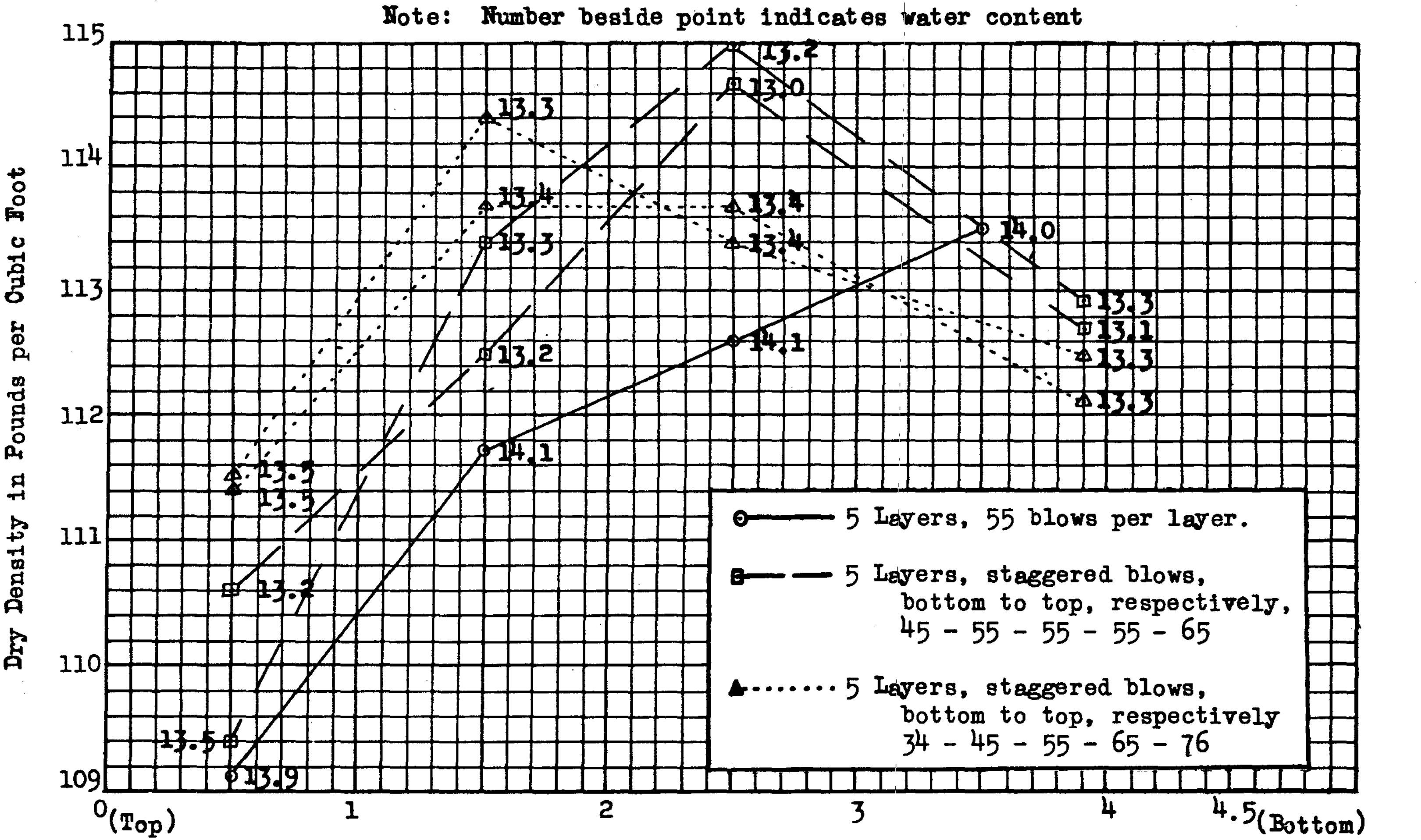
Sample 3

DENSITY GRADIENT IN PROCTOR MOLD

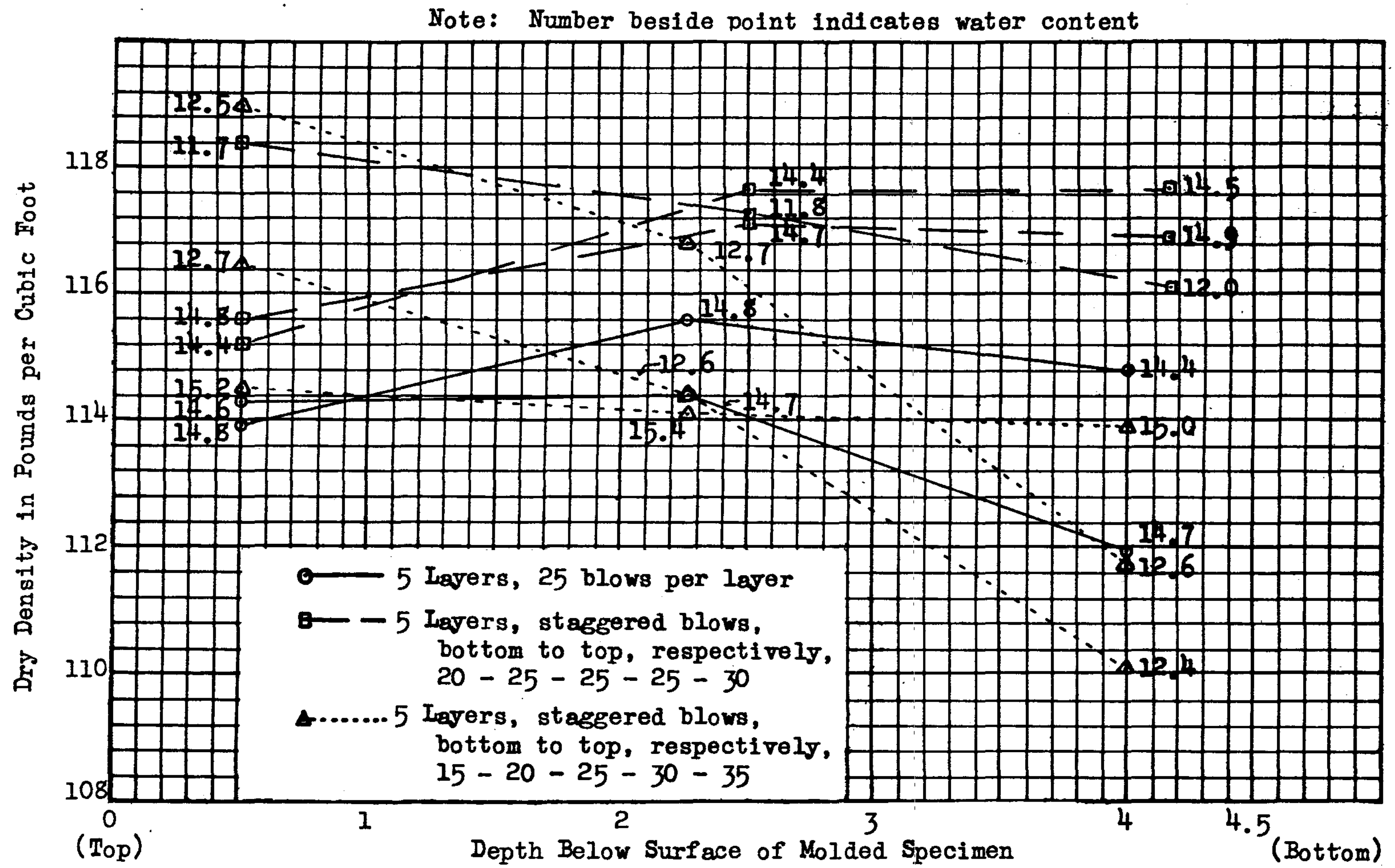
Vicksburg Loess



Depth Below Surface of Molded Specimen

Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

Sample 3 DENSITY GRADIENT IN CBR MOLD Vicksburg Loess



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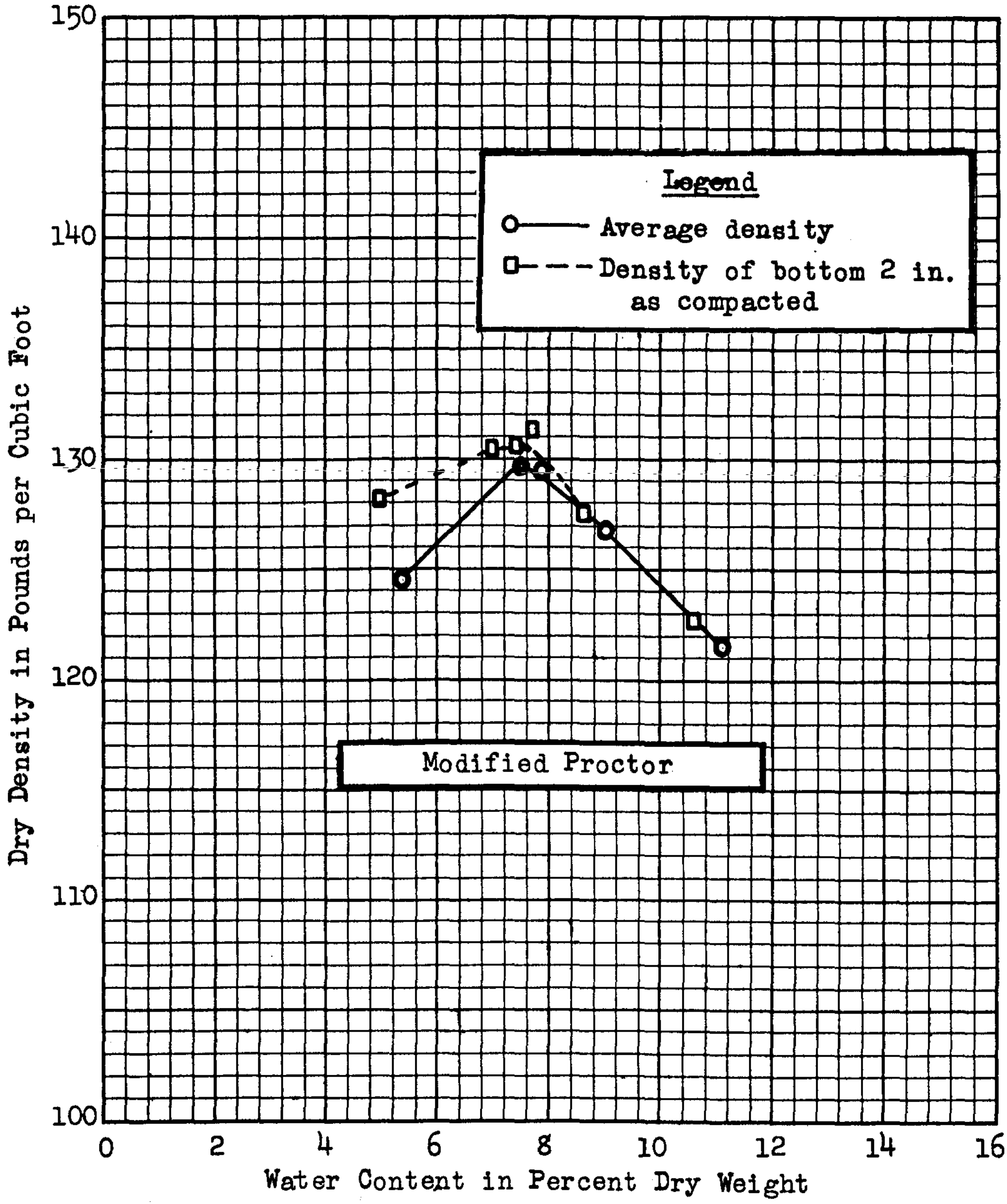
California Bearing Ratio Investigation

Sample 18 DENSITY GRADIENT IN PROCTOR MOLD Berry Field Lean Clay



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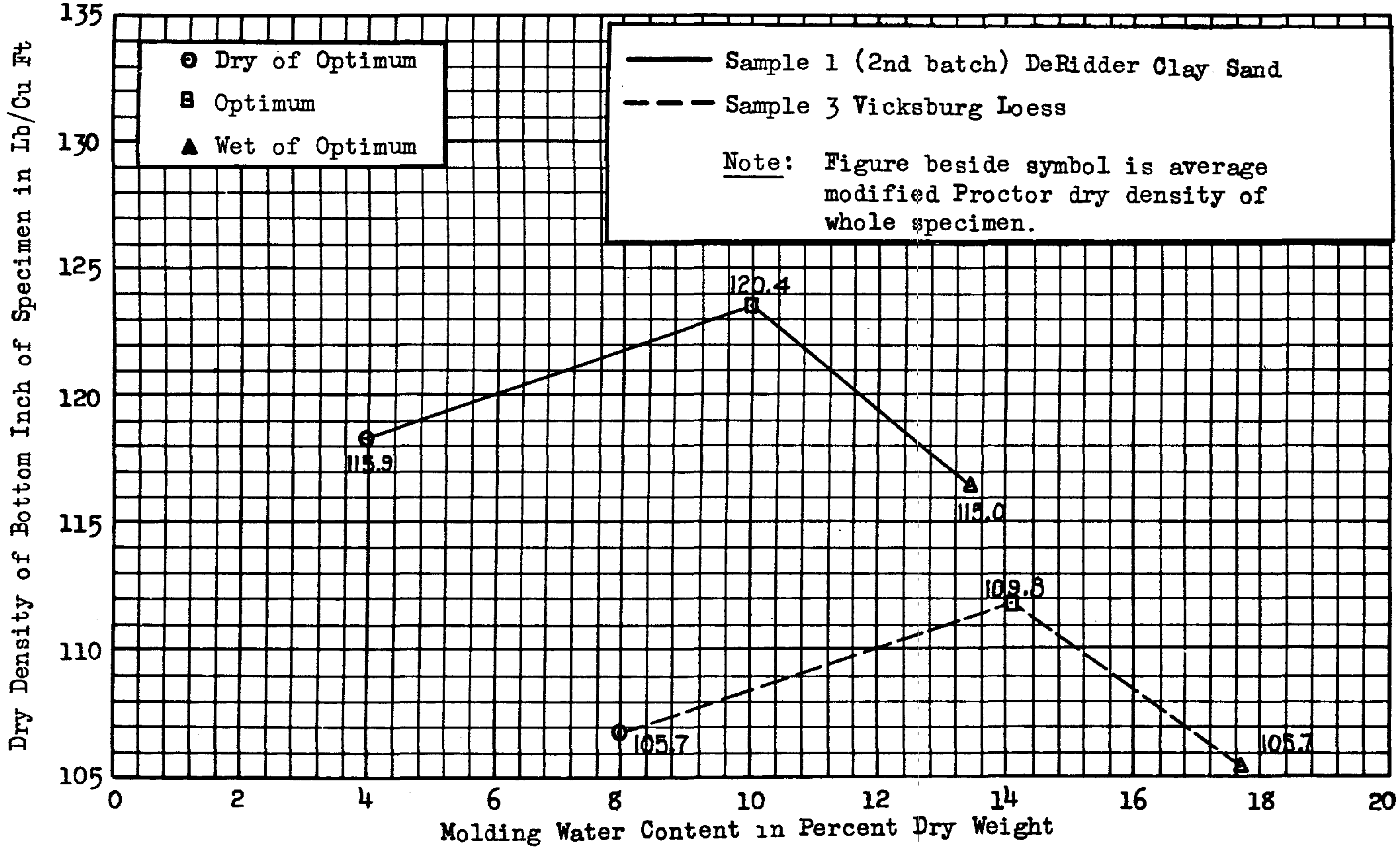
California Bearing Ratio Investigation

EFFECT OF MOLDING WATER CONTENT ON DENSITY GRADIENT

Sample 23 Vicksburg Clay Sand



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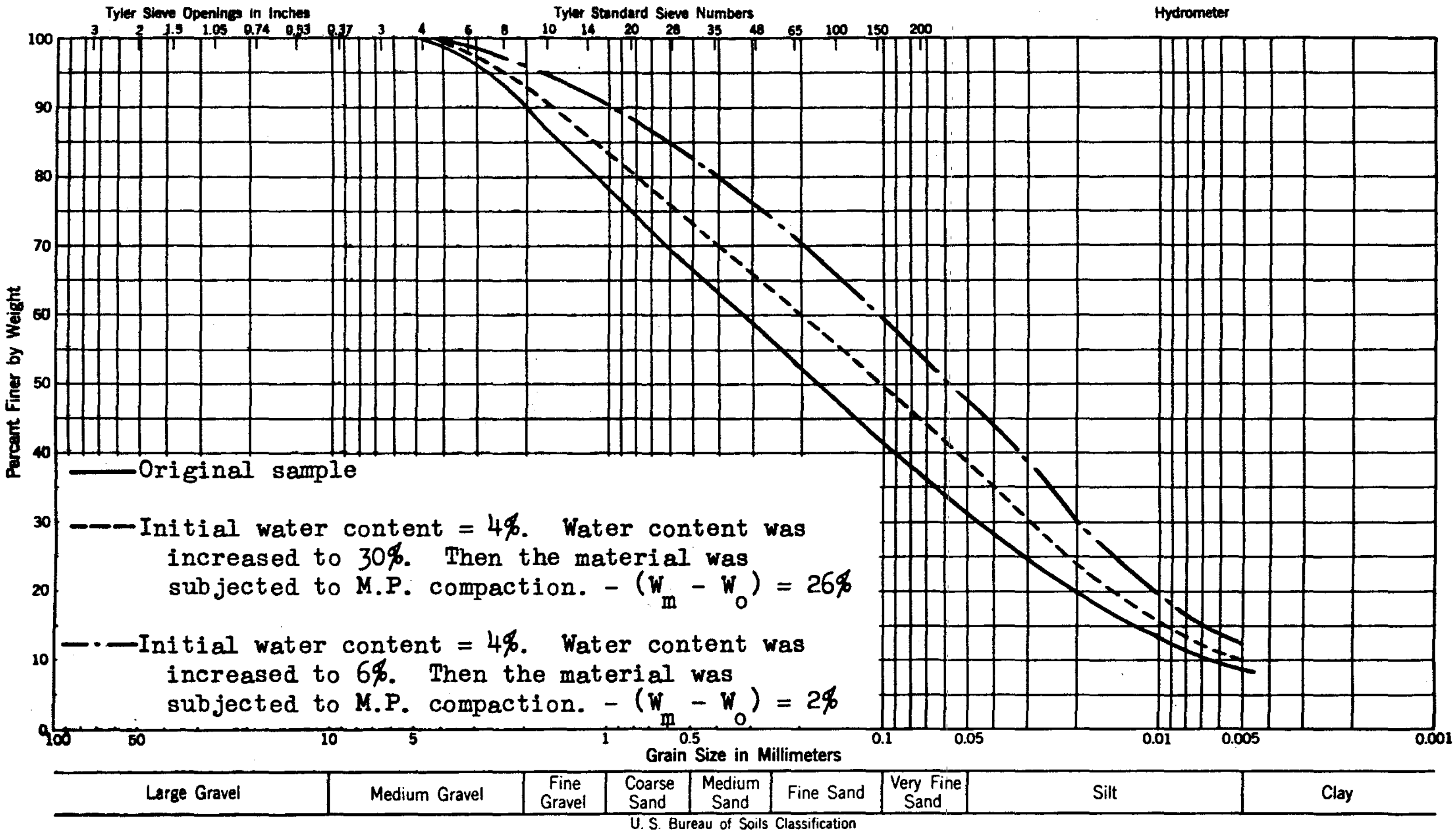
California Bearing Ratio Investigation

MOLDING WATER CONTENT VS DRY DENSITY OF BOTTOM INCH OF SPECIMEN

WAR DEPARTMENT

CORPS OF ENGINEERS U. S. ARMY



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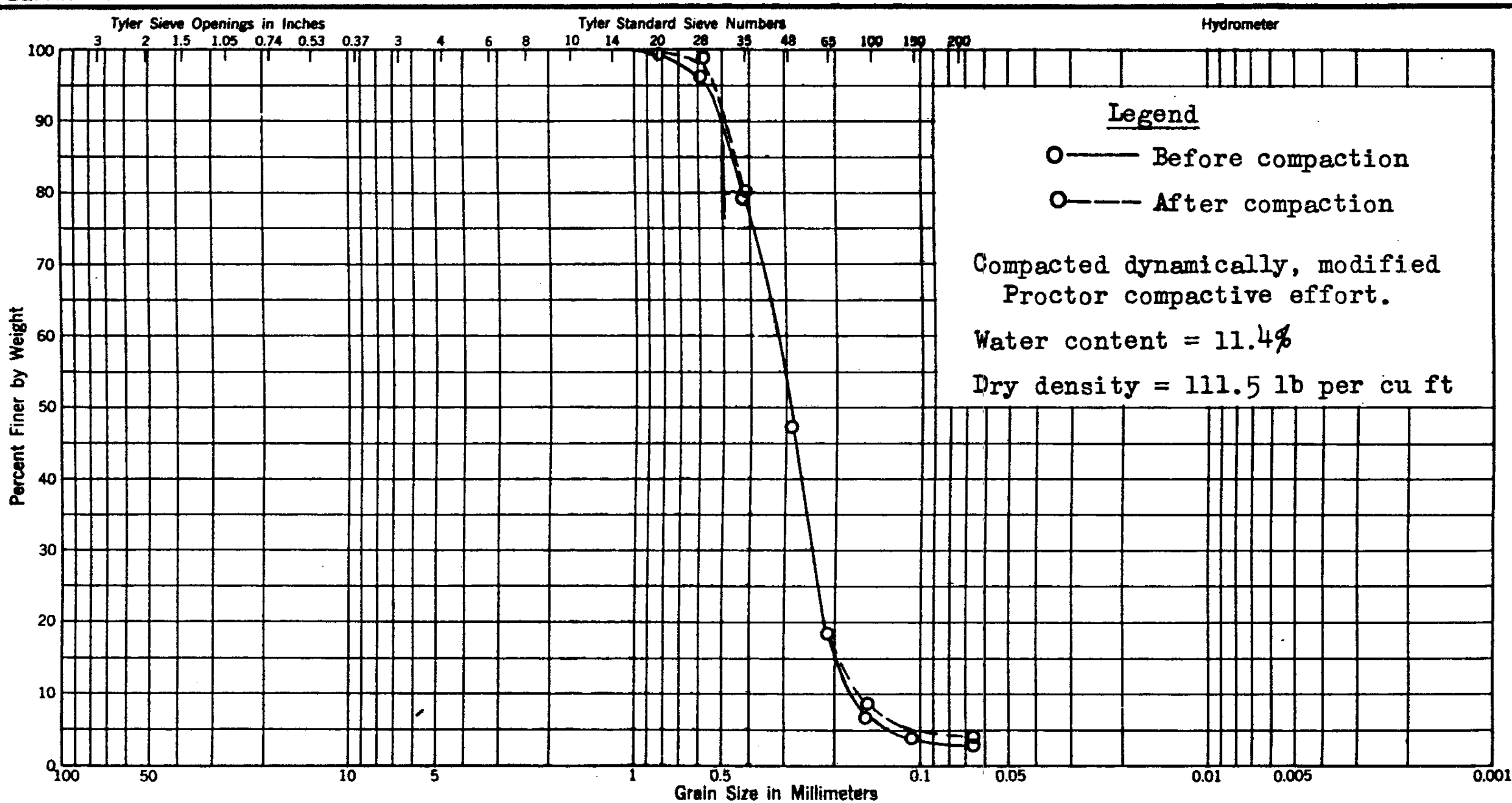
EFFECT OF COMPACTION ON GRAIN SIZE DISTRIBUTION

Sample 21 California Pumice



Large Gravel	Medium Gravel	Fine Gravel	Coarse Sand	Medium Sand	Fine Sand	Very Fine Sand	Silt	Clay

U. S Bureau of Soils Classification

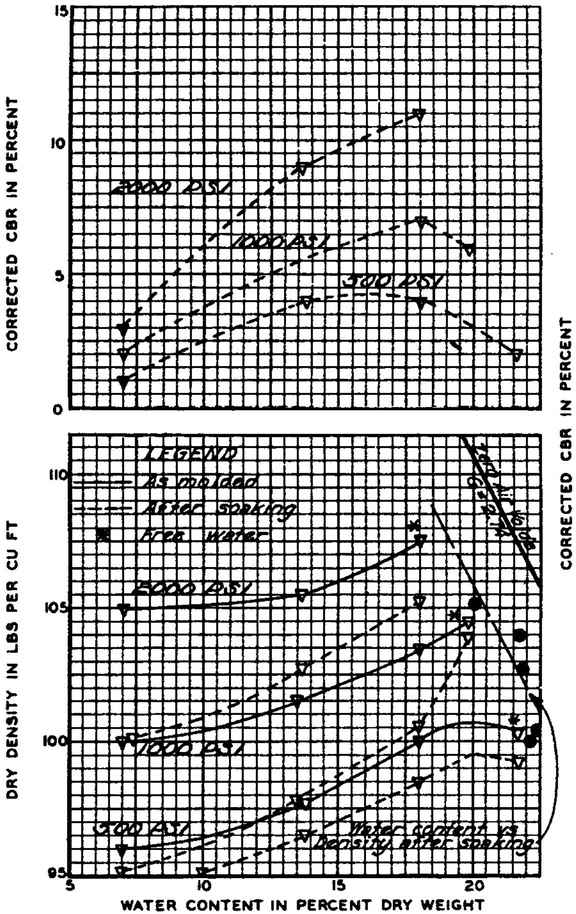
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

GRAIN SIZE DISTRIBUTION BEFORE AND AFTER BEING SUBJECTED TO DYNAMIC COMPACTION

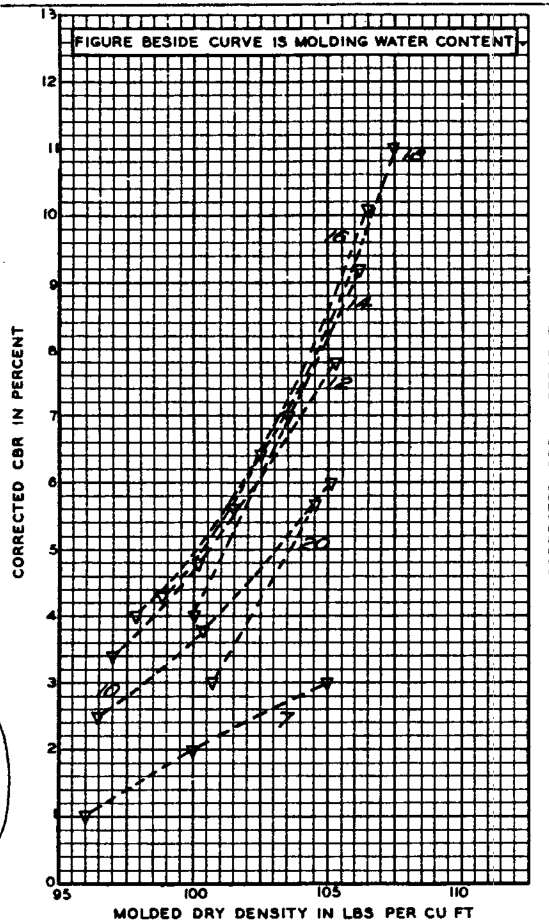
Sample 10 Eglin Field Sand

PLATE 76

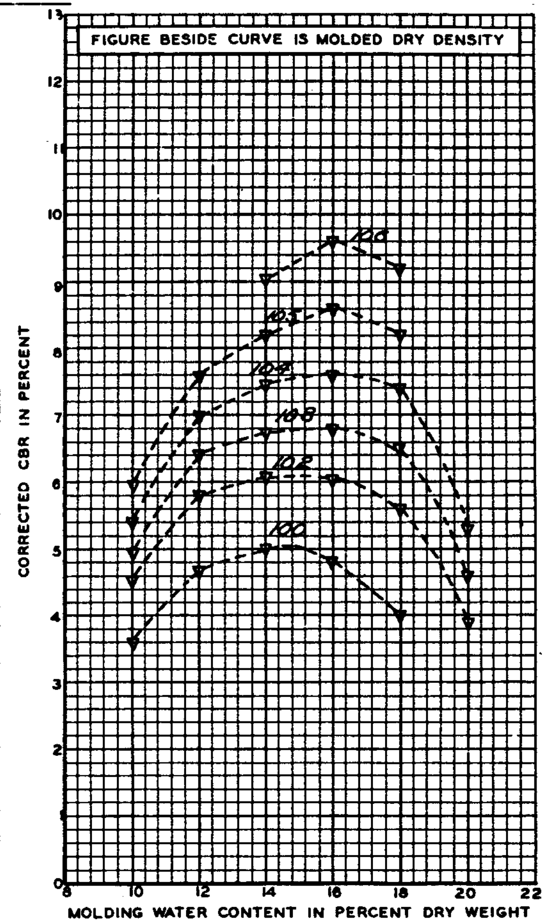
SPECIMENS SOAKED TOP AND BOTTOM FOR FOUR DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: STATIC COMPACTION
LOAD APPLIED ONCE FROM EACH END
SURCHARGES: 10 LB. SOAKING, 0 PENETRATION

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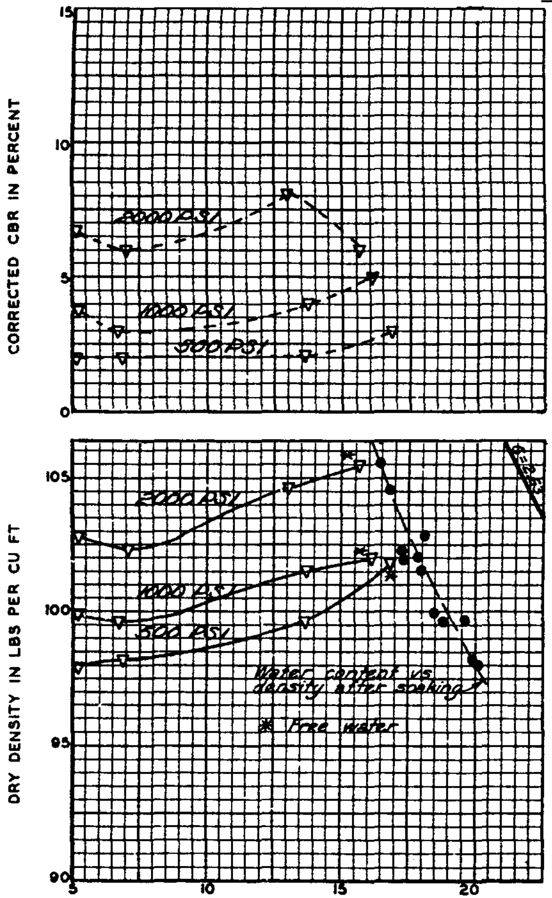
CALIFORNIA BEARING RATIO INVESTIGATION

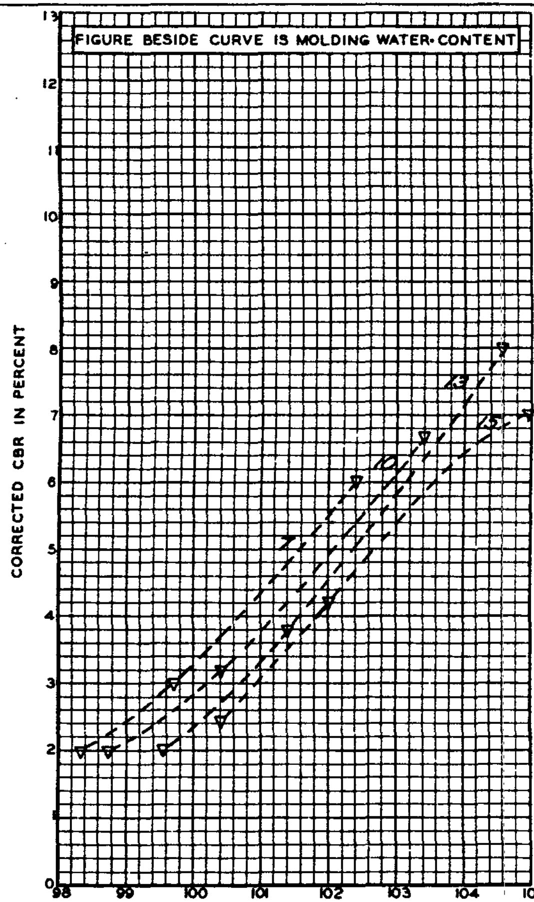
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 3 VICKSBURG LOESS

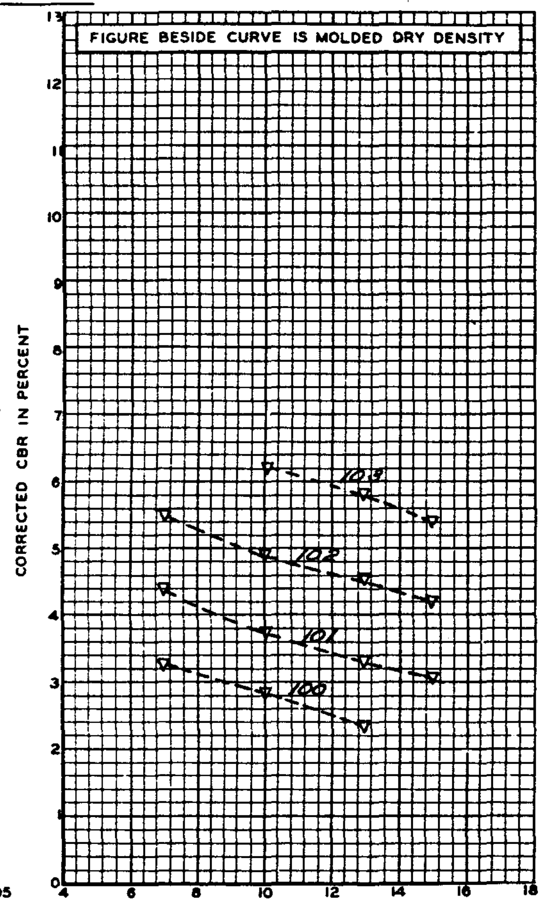
SPECIMENS SOAKED TOP AND BOTTOM FOR 1 TO 2 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: STATIC COMPACTION - LOAD APPLIED ONCE FROM ONE END.
SURCHARGE: 10 LB SOAKING, 10 LB PENETRATION.
NO SWELL DURING SOAKING.

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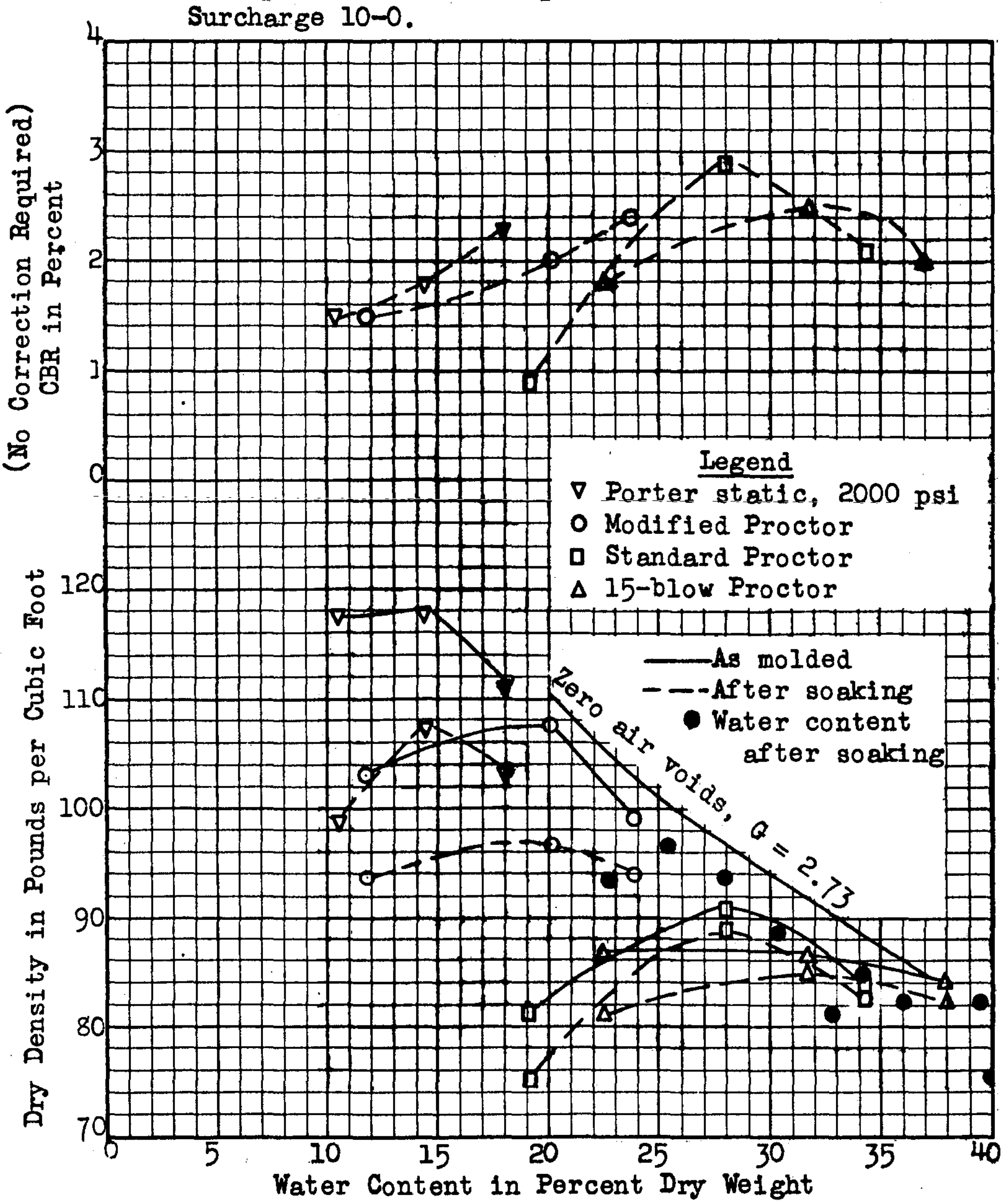
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 10 EGLIN FIELD SAND

Notes: The 15-blow compaction data are erratic, due to the material having been used previously for compaction tests.
All specimens soaked top and bottom for four days.
Surcharge 10-0.



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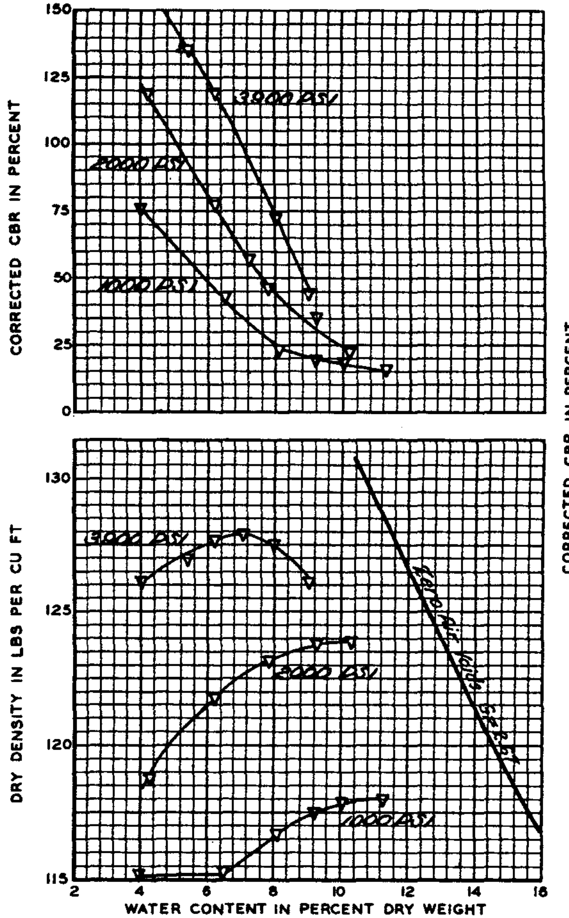
California Bearing Ratio Investigation

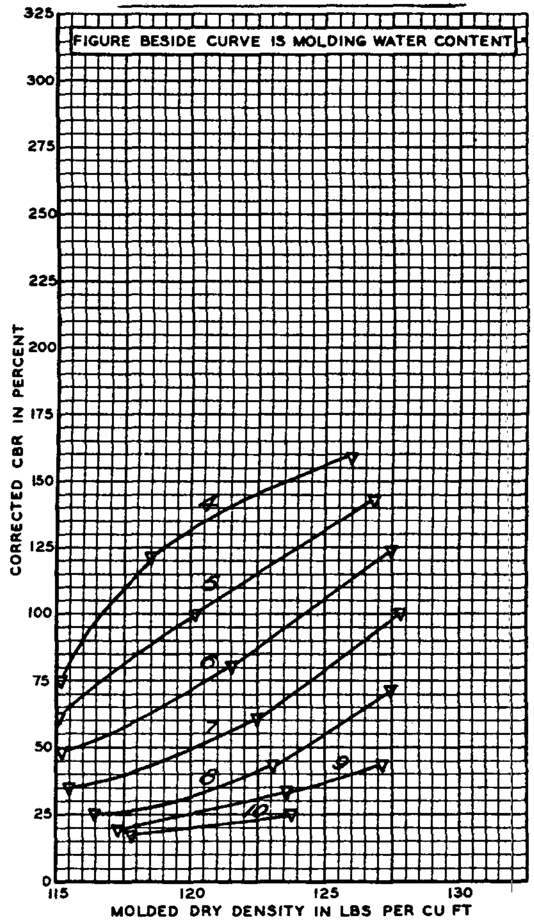
MOLDING WATER CONTENT VS DENSITY AND CBR

Sample 20 Stockton Adobe Clay

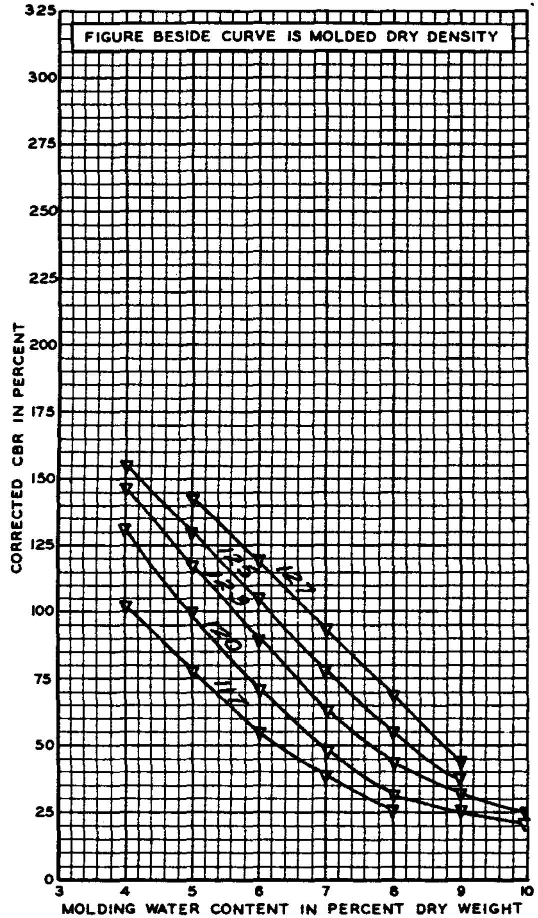
SPECIMENS TESTED AS MOLDED



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: STATIC COMPACTION.
LOAD APPLIED ONCE FROM ONE END.
SURCHARGE: 10 LB. PENETRATION.

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U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

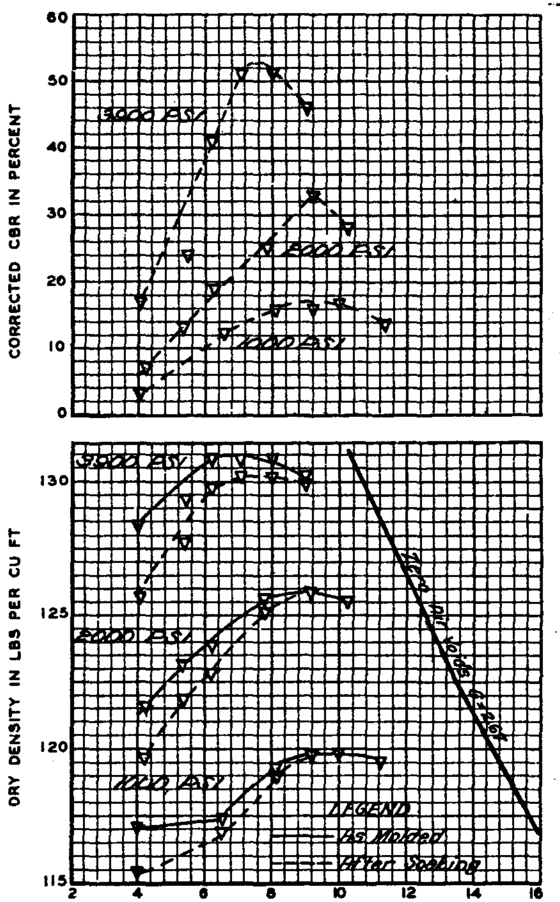
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 23—VICKSBURG CLAY SAND

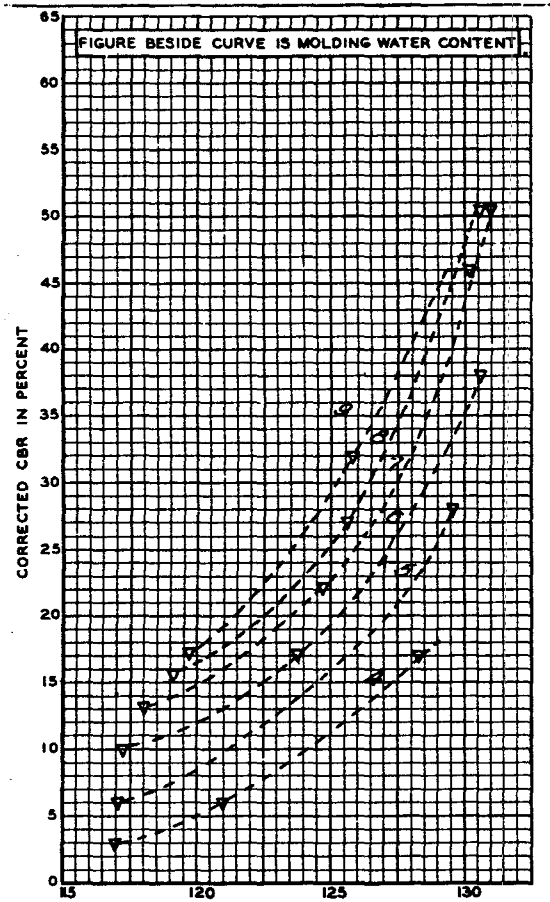
WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

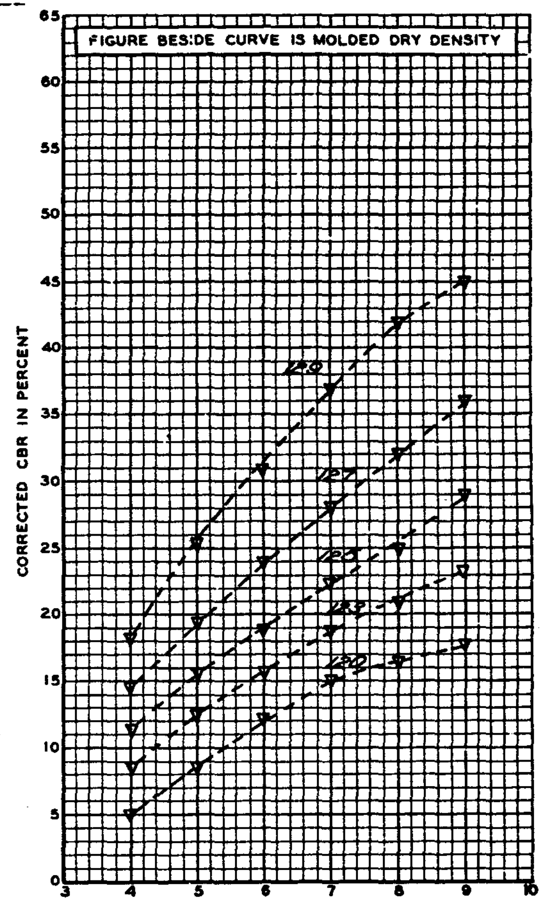
SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: STATIC COMPACTION.
LOAD APPLIED ONCE FROM EACH END.
SURCHARGES: 10 LB. SOAKING – 10 LB. PENETRATION.

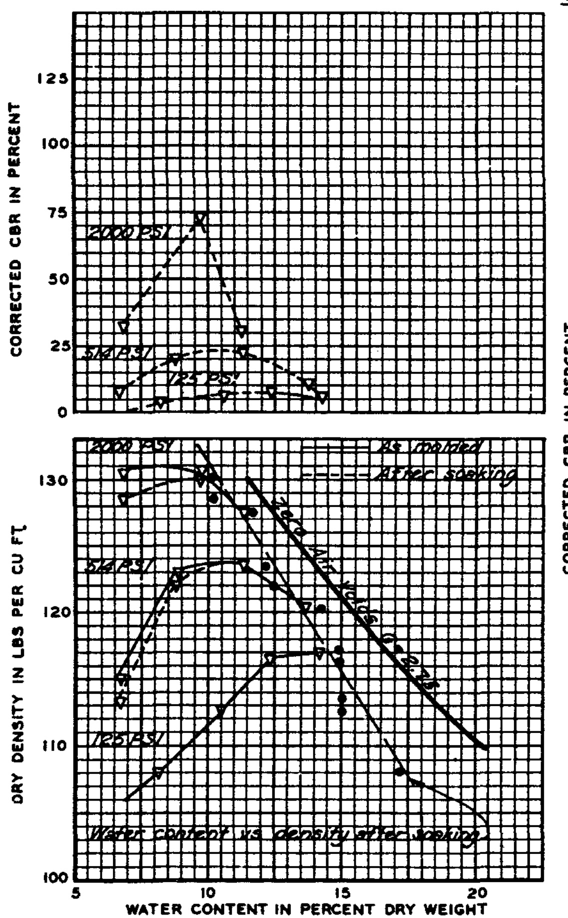
MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION

CALIFORNIA BEARING RATIO INVESTIGATION

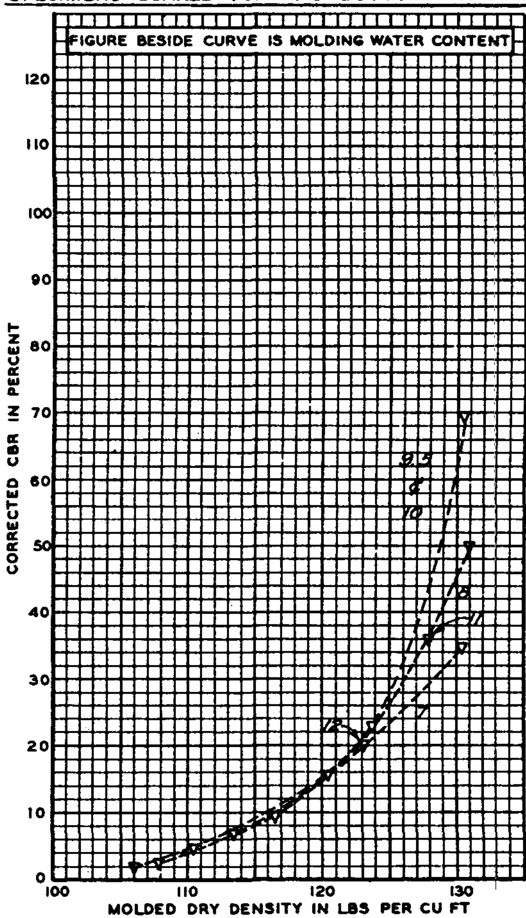
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 23 — VICKSBURG CLAY SAND

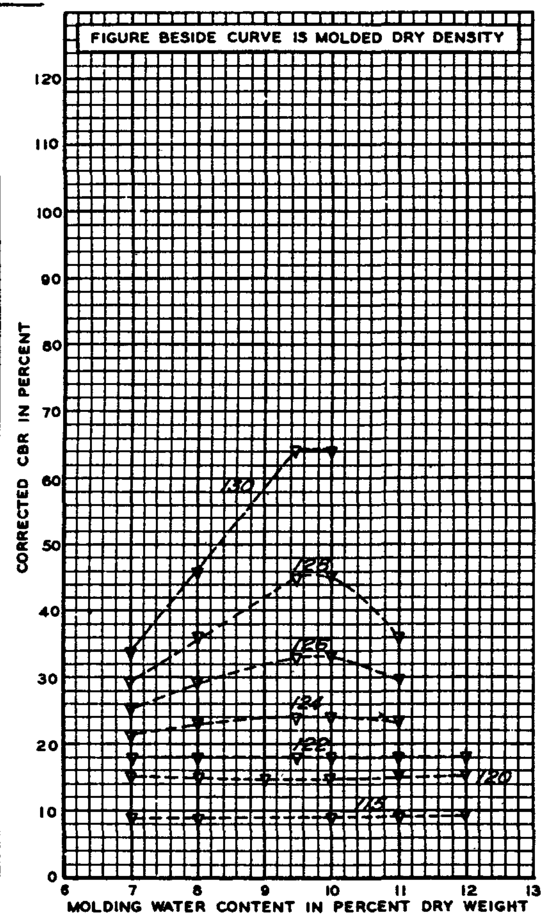
SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



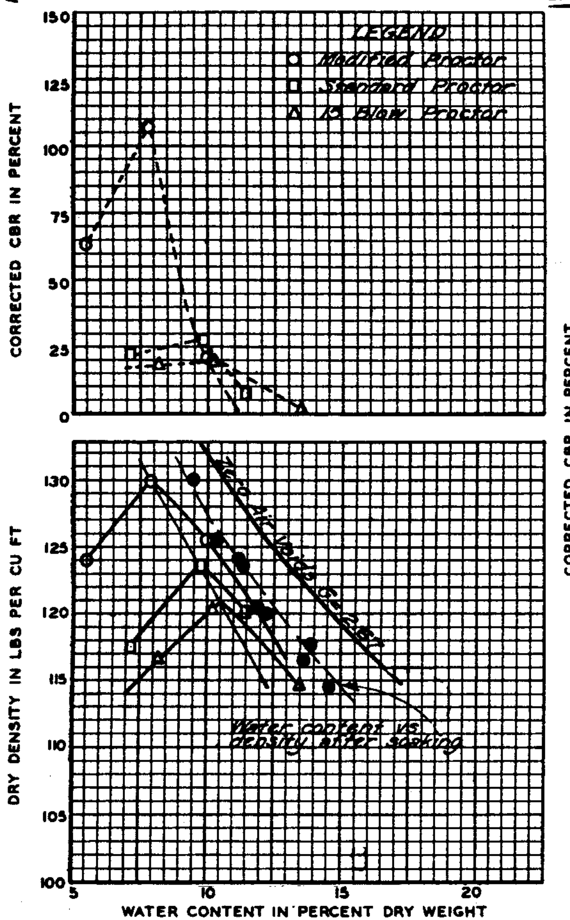
WATER CONTENT VS CBR

NOTES: STATIC COMPACTION. LOAD APPLIED ONCE FROM EACH END.
SURCHARGES: 10 LB SOAKING. 0 PENETRATION.

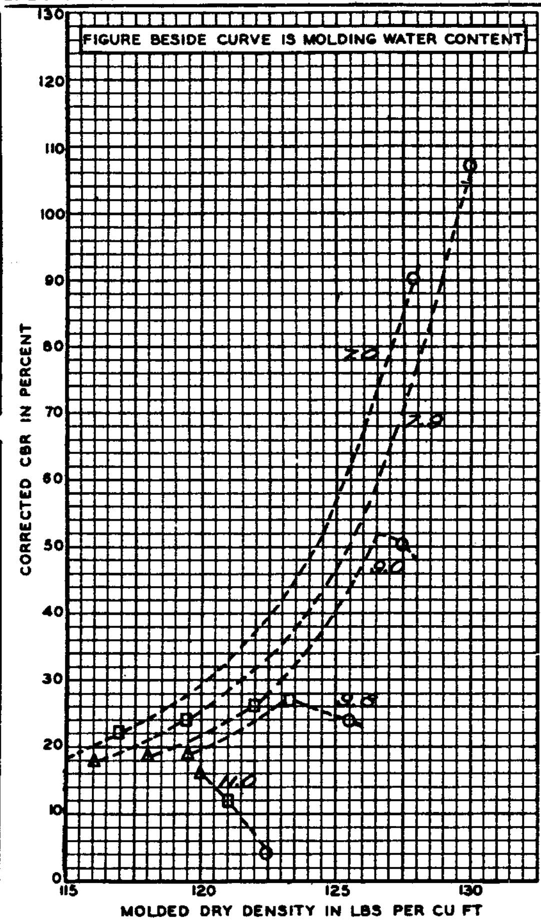
MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT
MARIETTA MIX CLAY SAND

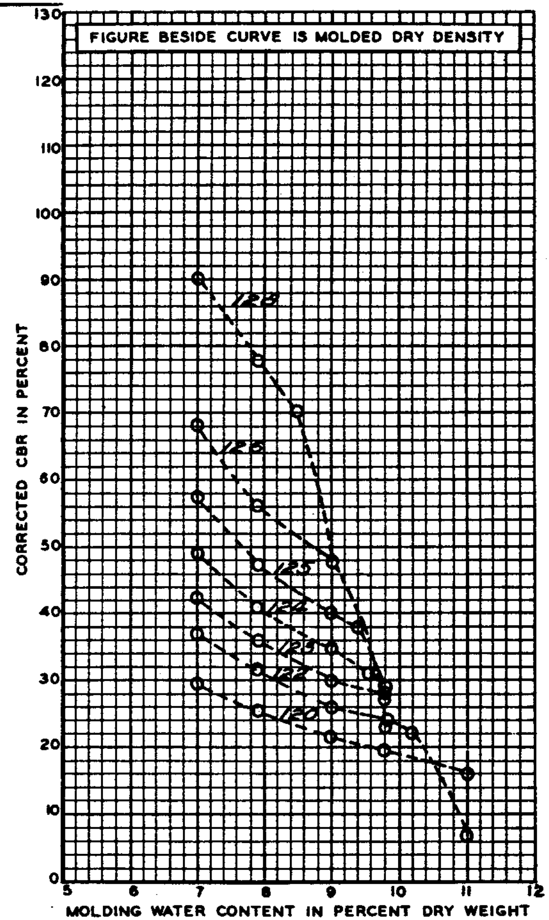
SPECIMENS SOAKED TOP AND BOTTOM FOR FOUR DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: SURCHARGES: 10 LBS SOAKING, 60 LBS PENETRATION
ALL POINTS PLOTTED ARE AVERAGE OF TWO OR MORE TESTS.
NO SWELL DURING SOAKING

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

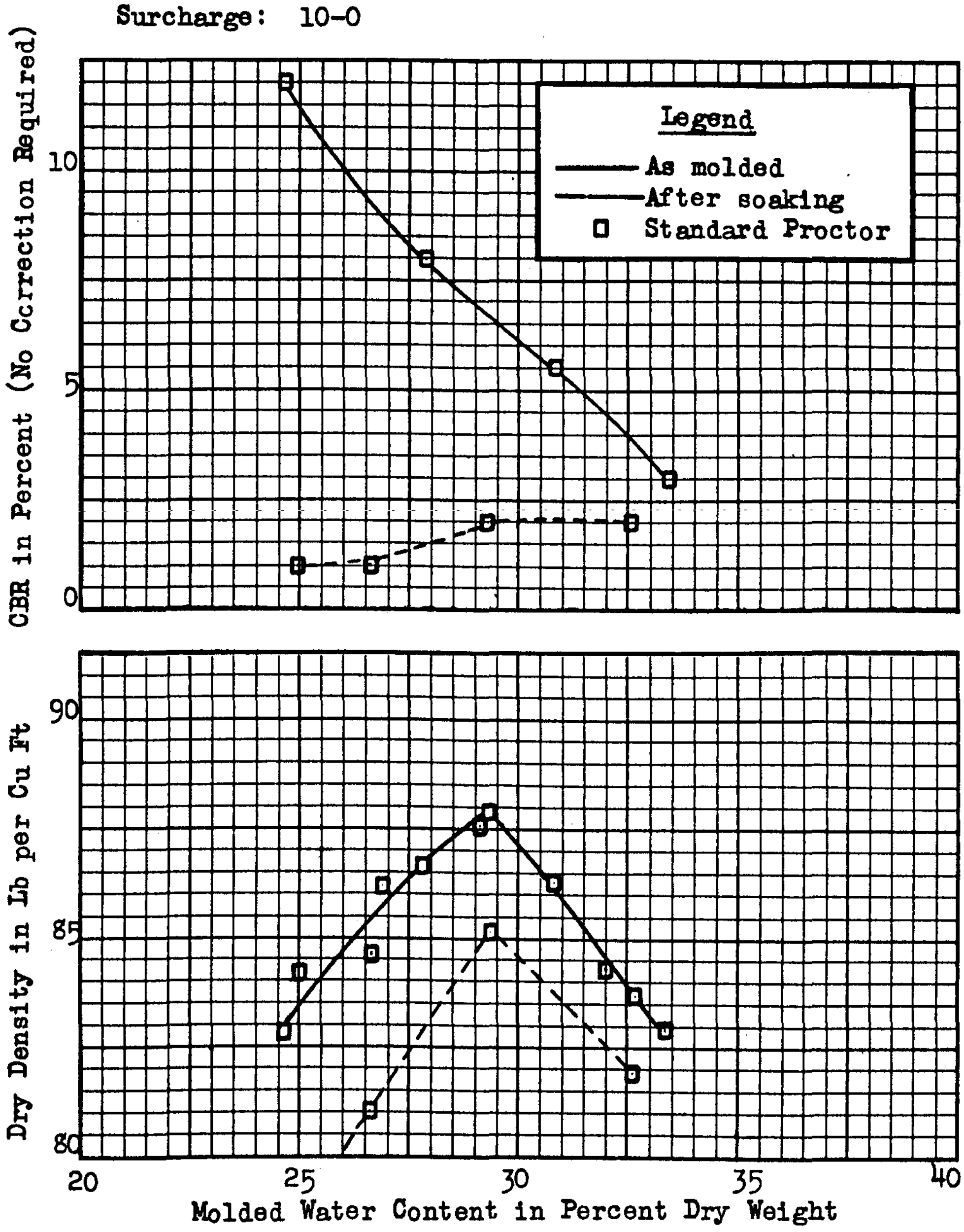
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE I DERIDDER CLAY SAND

Notes:

Soaked specimens soaked top and bottom for 5 days

Surcharge: 10-0



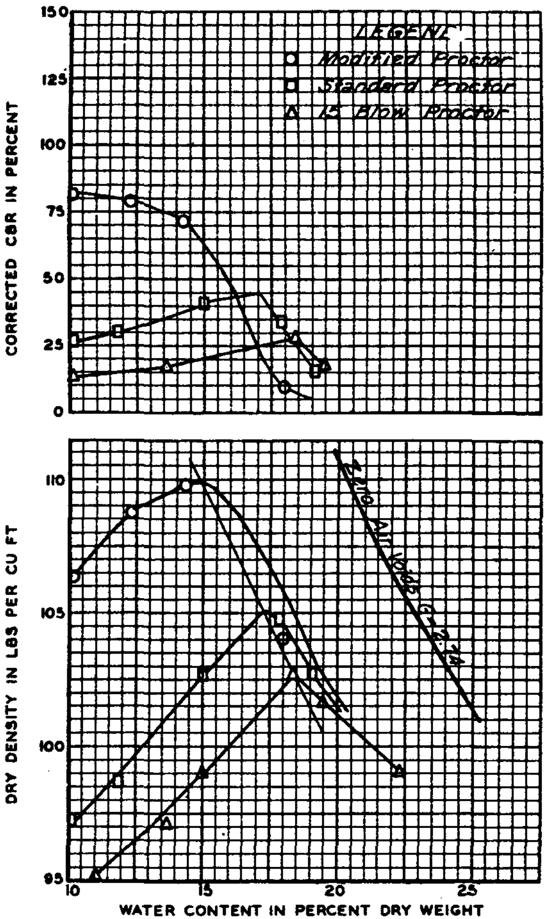
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MOLDING WATER CONTENT VS DENSITY AND CBR

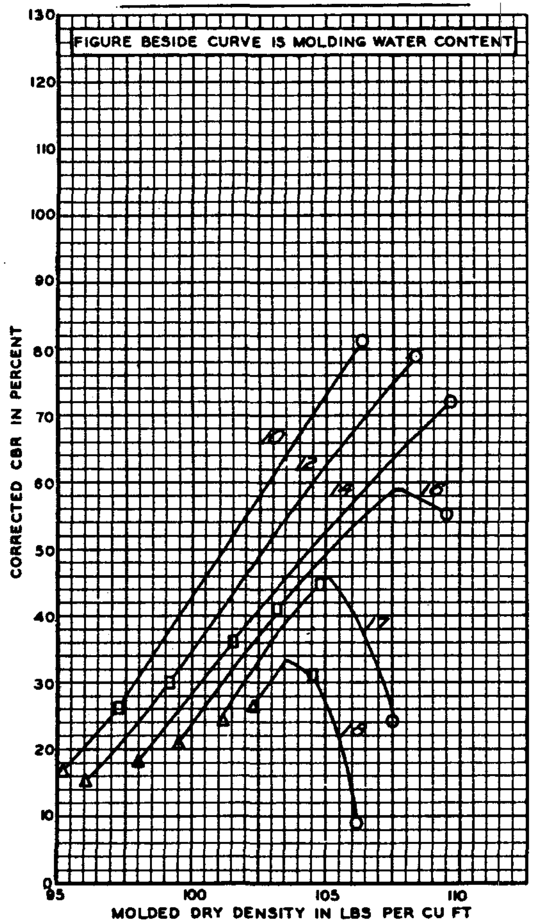
Sample 2 Louisiana Buckshot

WAR DEPARTMENT CORPS OF ENGINEERS. U.S. ARMY

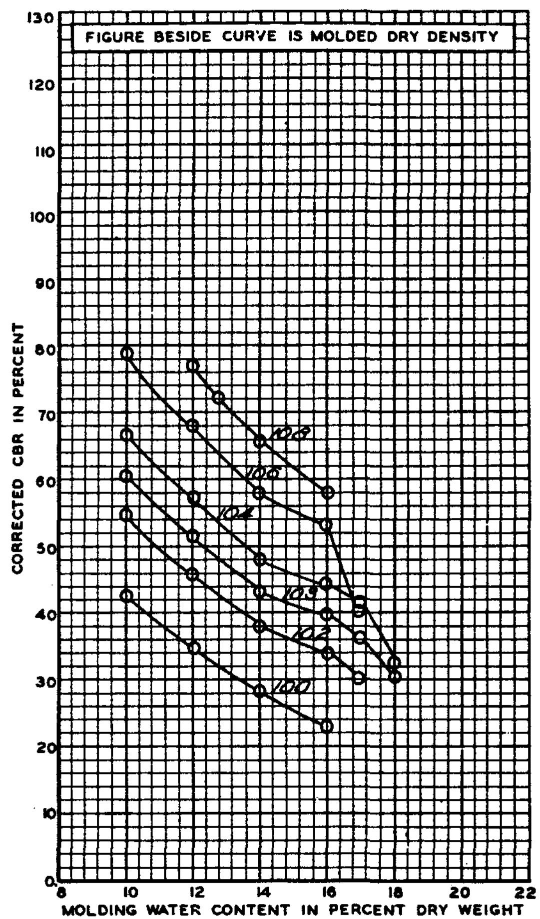
SPECIMENS TESTED AS MOLDED



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

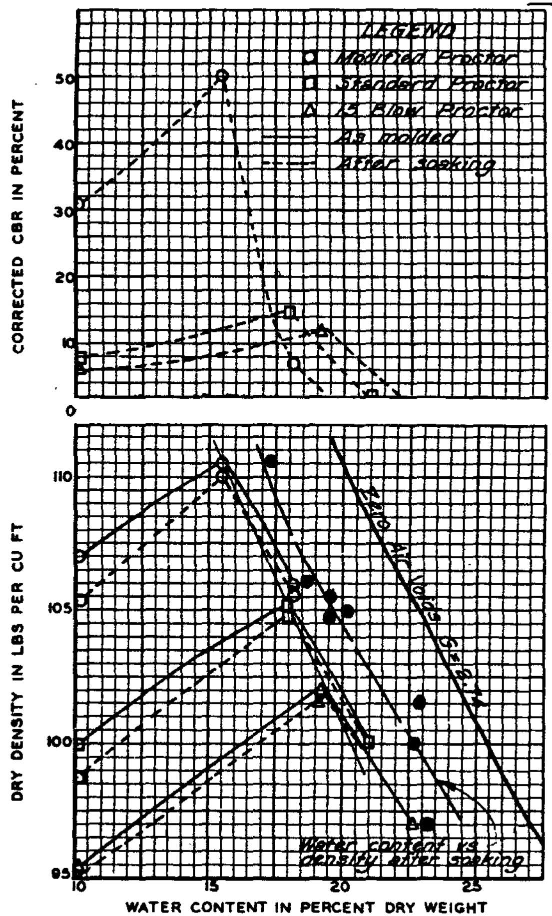
NOTES: SURCHARGE: 10 LB PENETRATION

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

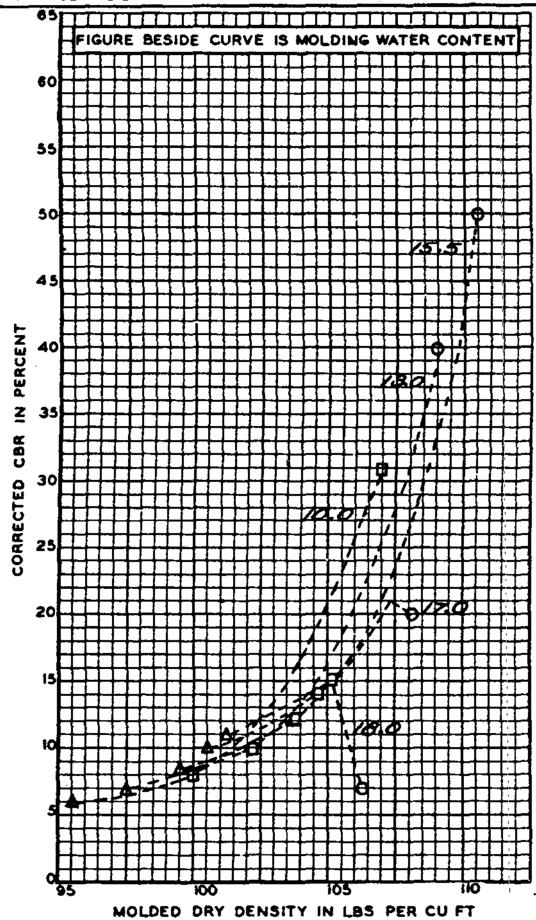
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 3 VICKSBURG LOESS

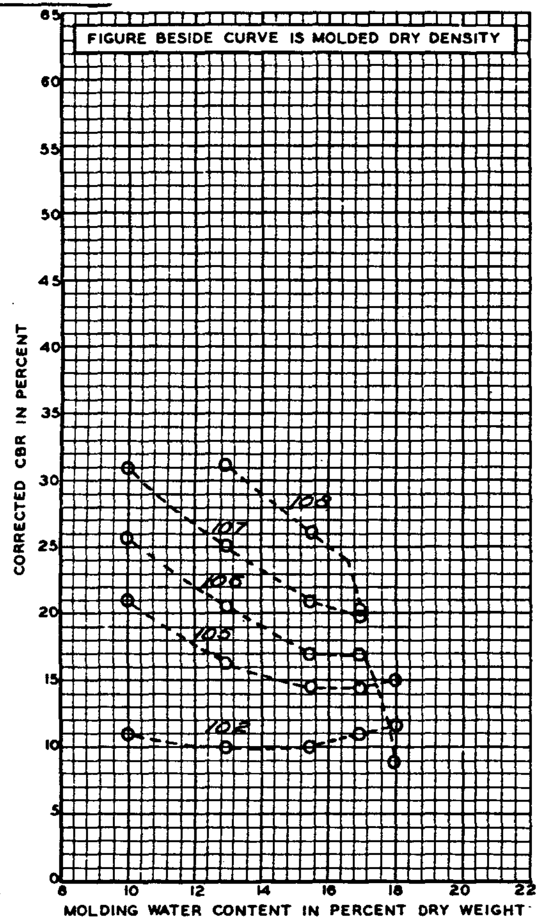
SPECIMENS SOAKED FROM BOTTOM ONLY 4 TO 6 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: SURCHARGES: 10 LB SOAKING, 10 LB PENETRATION.
ALL POINTS ARE AVERAGE OF TWO TEST.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

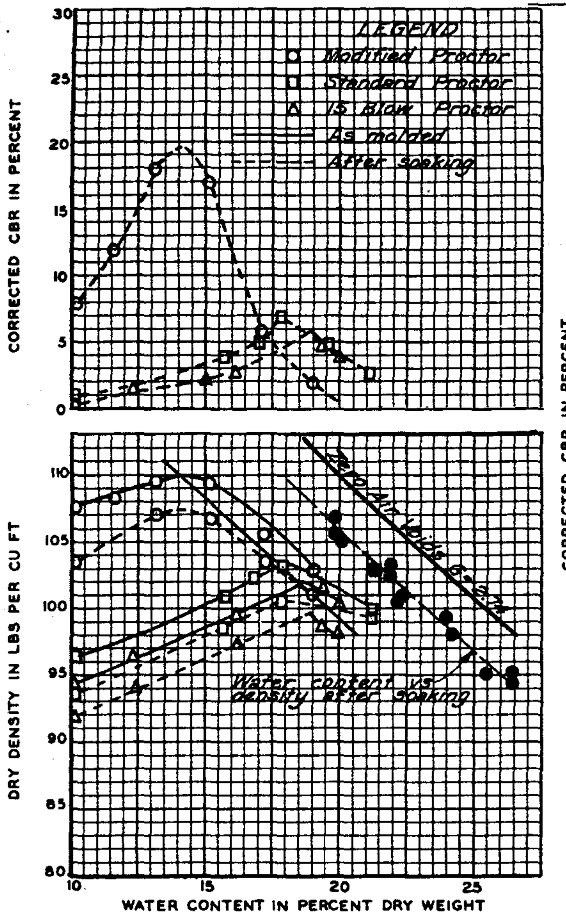
SAMPLE 3 VICKSBURG LOESS

PLATE 86

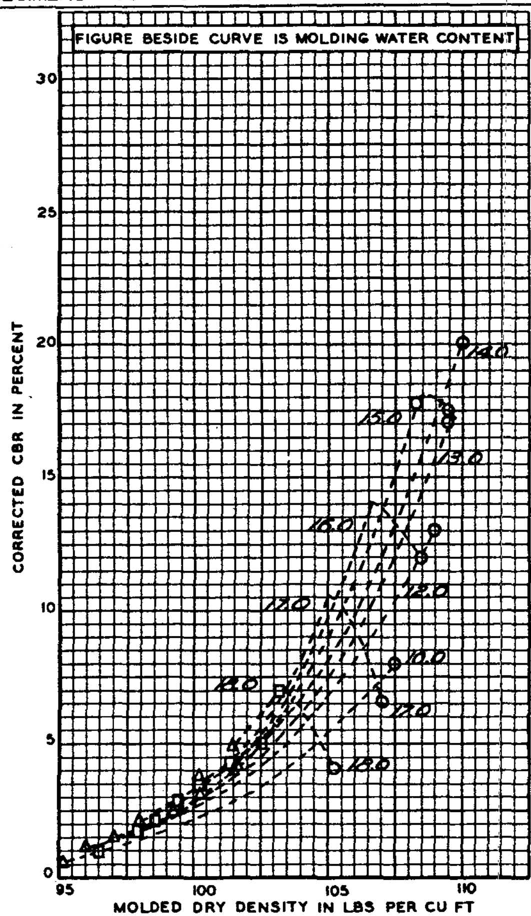
WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

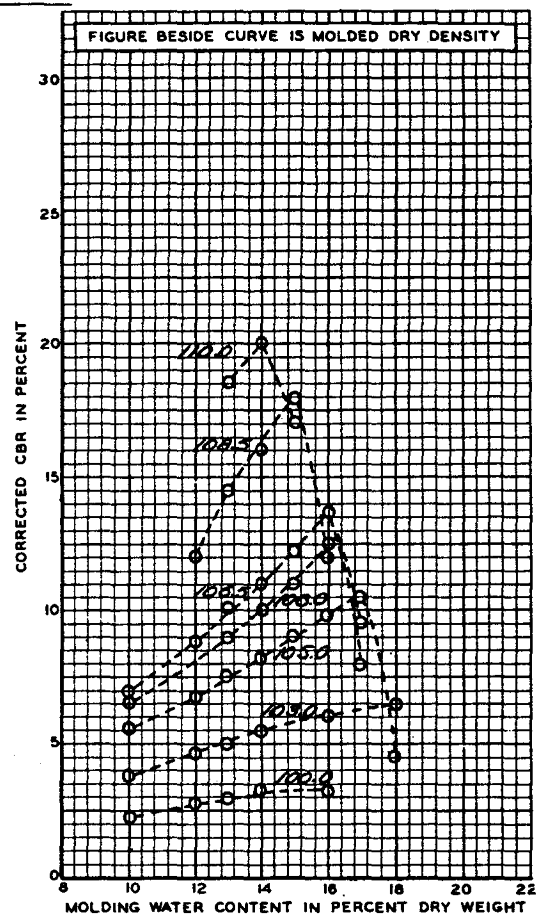
SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

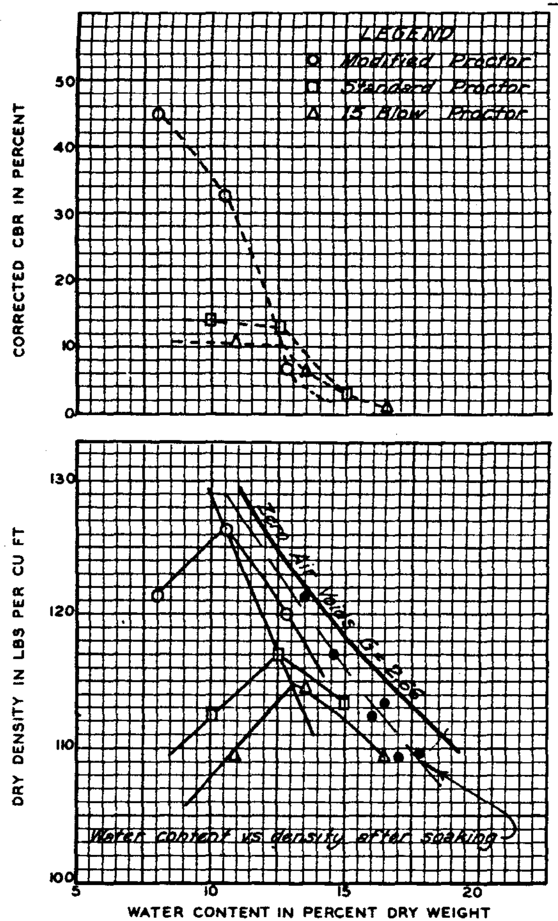
NOTES: SURCHARGE: 10 LB SOAKNG, 0 PENETRATION.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

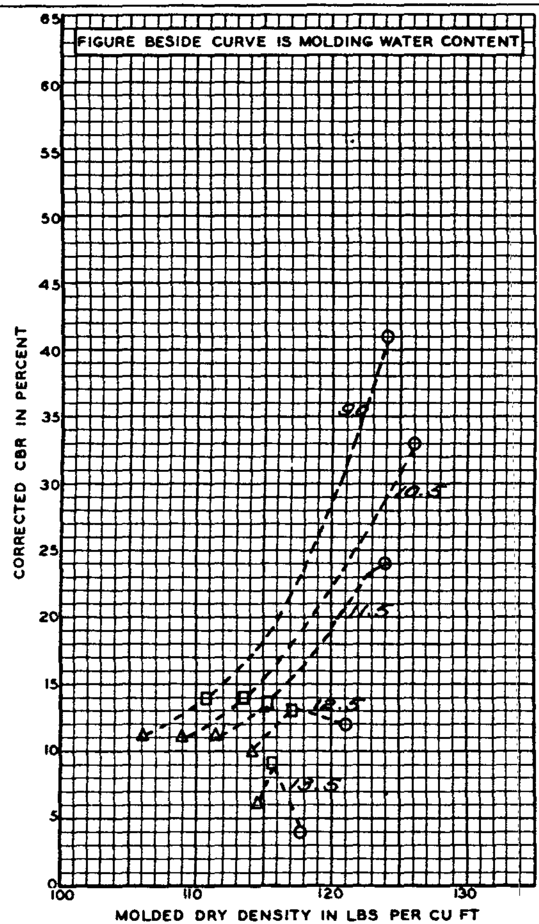
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 3 VICKSBURG LOESS

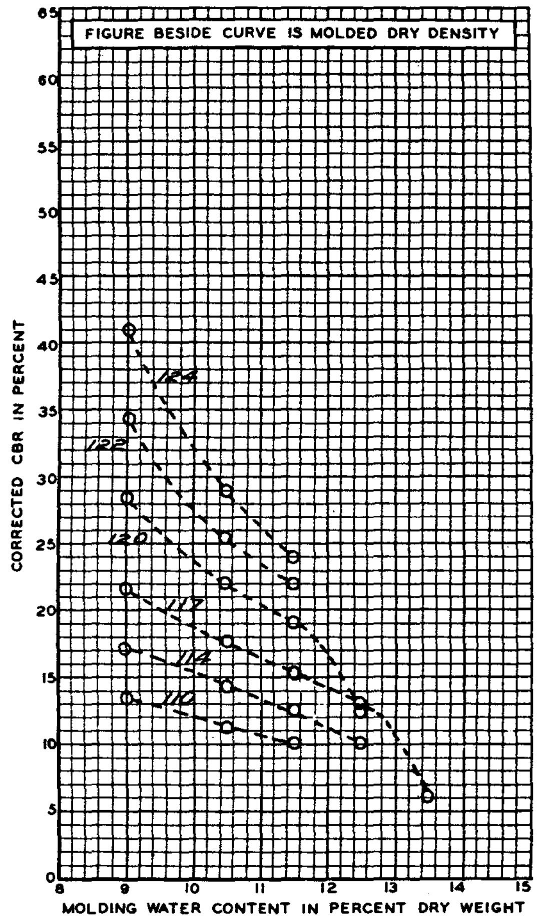
SPECIMENS SOAKED TOP AND BOTTOM FOR FOUR DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: SURCHARGES: 10 LB SOAKING, 0 PENETRATION.
ALL POINTS PLOTTED ARE AVERAGE OF TWO OR MORE TESTS.
NO SWELL DURING SOAKING.

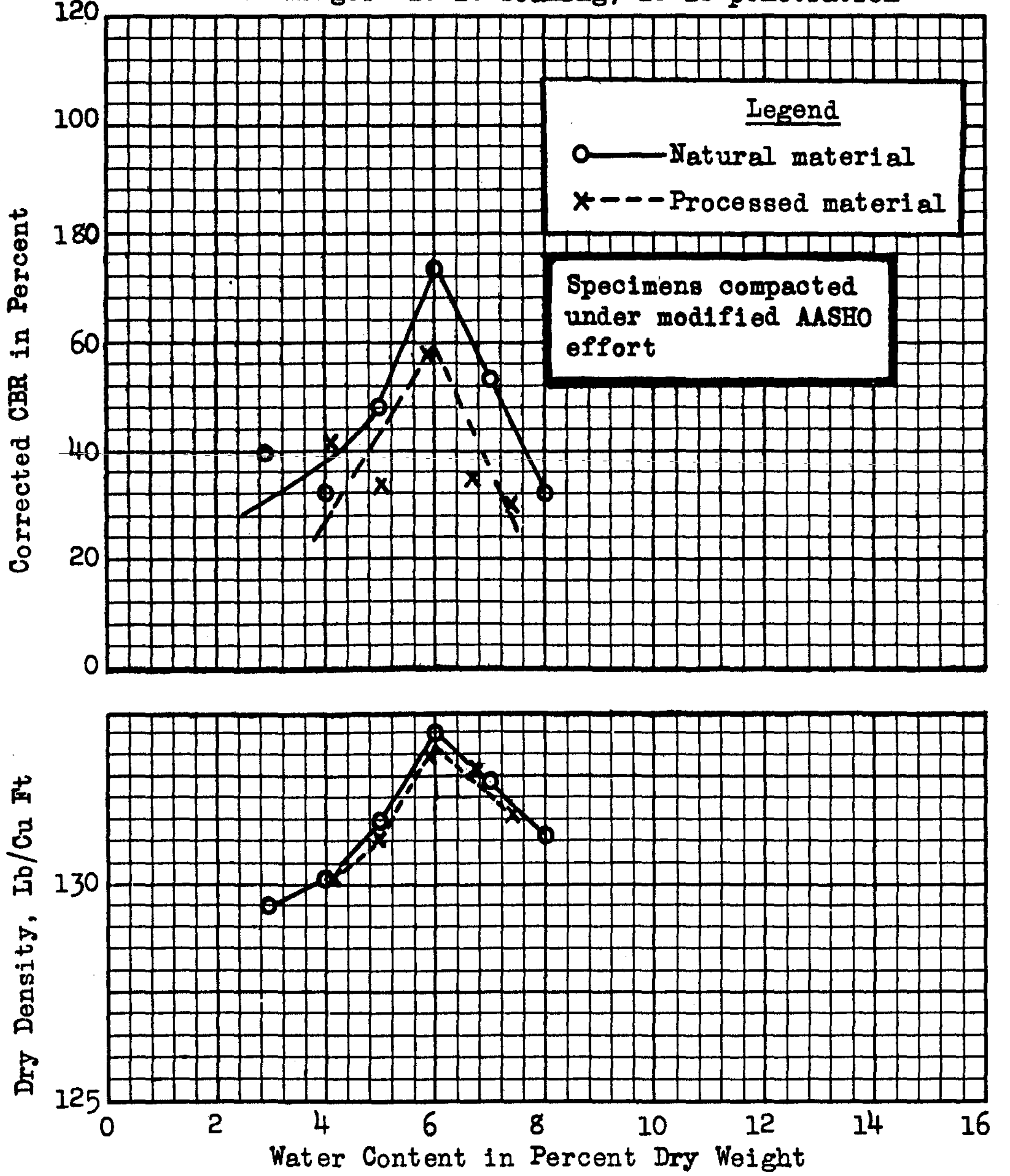
MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 5 DERIDDER SAND CLAY (LOAM)

Notes: All specimens soaked from bottom only until free water was present on top.

Surcharges: 10-lb soaking, 10-lb penetration

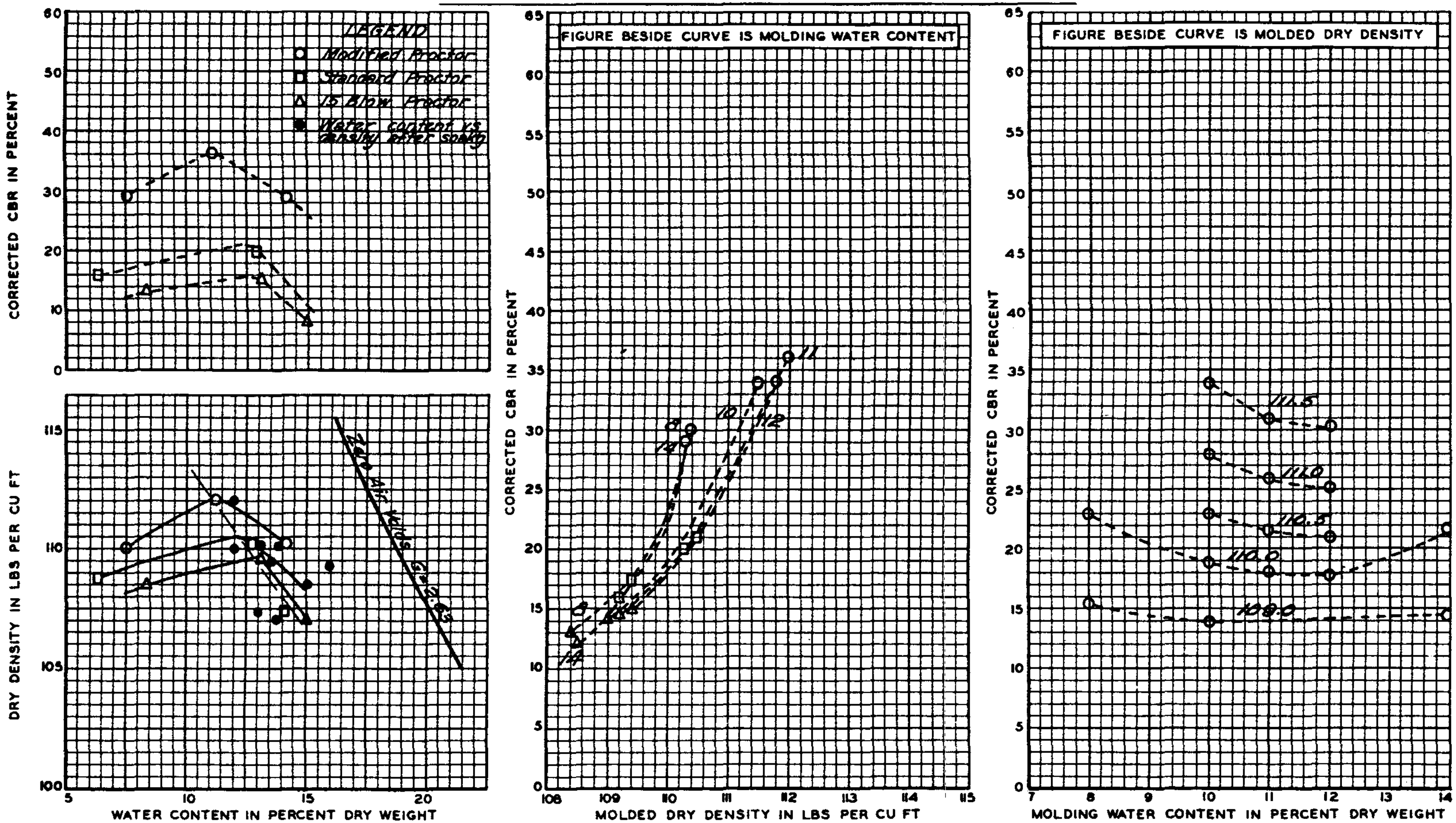


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MOLDING WATER CONTENT VS DRY DENSITY AND CBR

Sample 7 Barksdale Clay Gravel

SPECIMENS SOAKED FROM BOTTOM ONLY FOR 1 TO 6 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR

DENSITY VS CBR

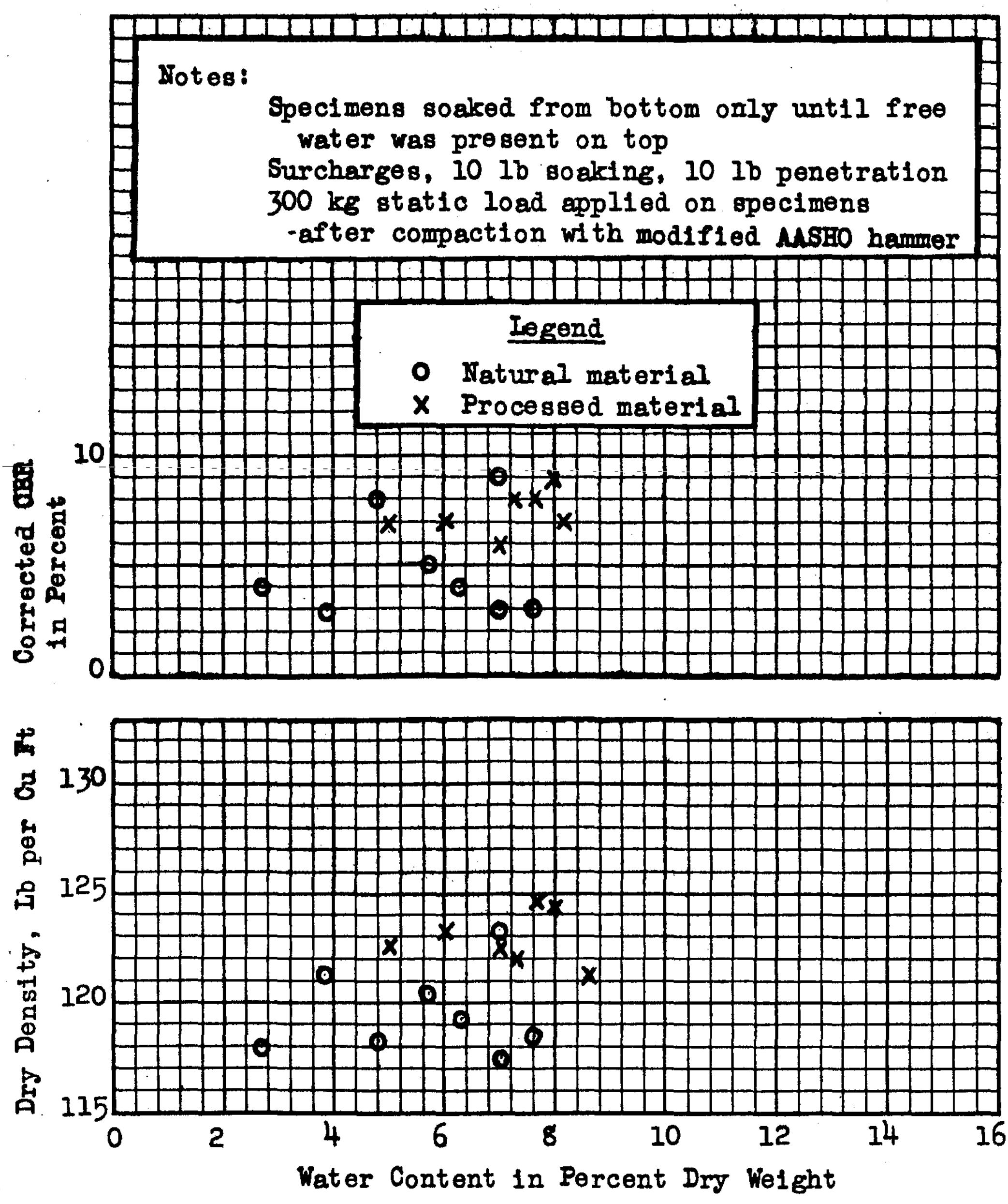
WATER CONTENT VS CBR

NOTES: SURCHARGE: 10 LB SOAKING; 10 LB PENETRATION.
ALL POINTS PLOTTED ARE AVERAGE OF TWO TESTS.
NO SWELL DURING SOAKING.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 10 EGLIN FIELD SAND

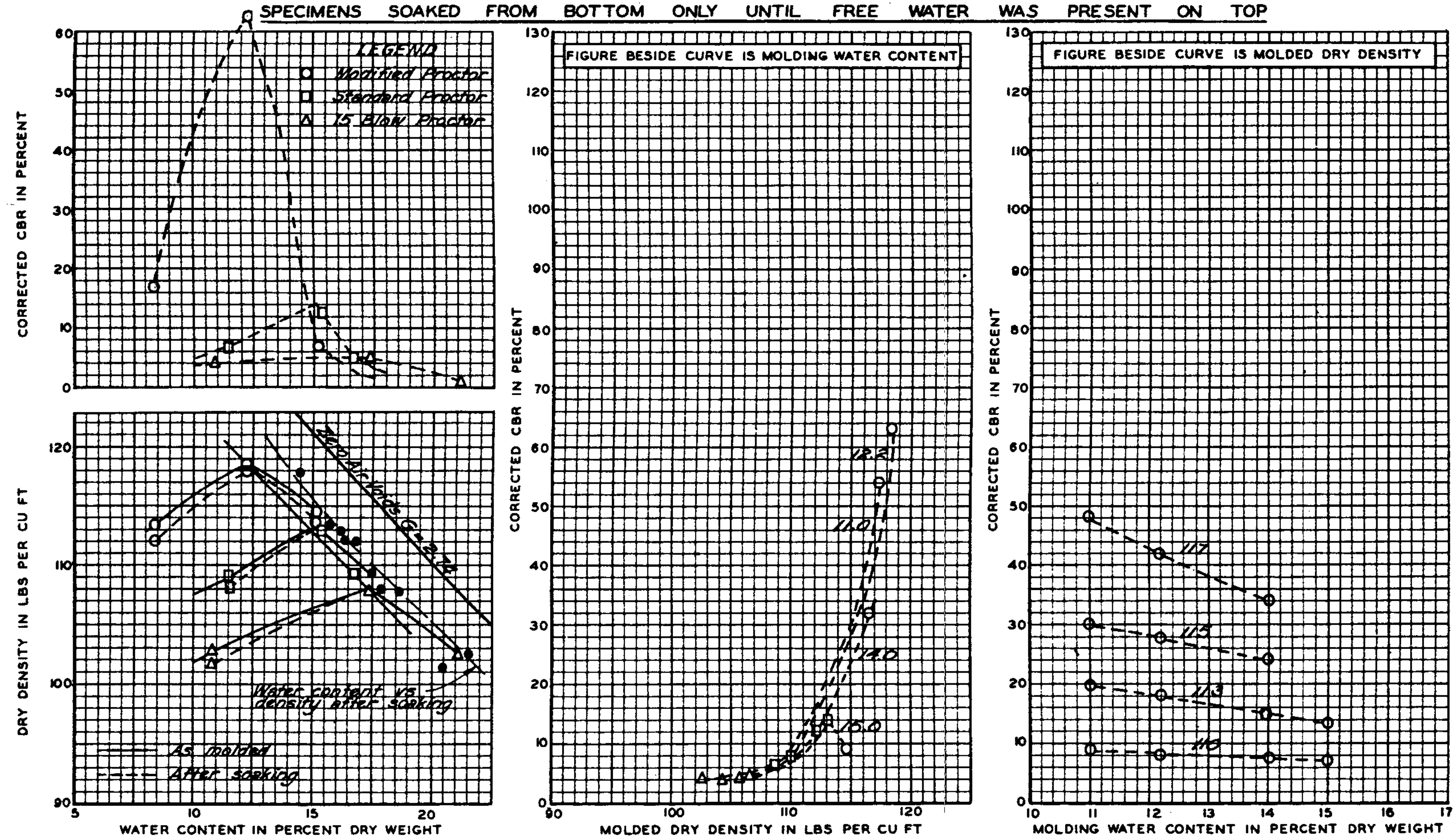


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MOLDING WATER CONTENT VS DRY DENSITY AND CBR

Sample 15 Manchester Sand and Gravel

SPECIMENS SOAKED FROM BOTTOM ONLY UNTIL FREE WATER WAS PRESENT ON TOP



MOLDING WATER CONTENT VS DENSITY AND CBR

DENSITY VS CBR

WATER CONTENT VS CBR

NOTES: SURCHARGE: 10 LB SOAKING, 10 LB PENETRATION.
ALL POINTS PLOTTED ARE AVERAGE OF TWO TESTS.
THIRD TIME MATERIAL USED FOR COMPACTION.

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION

CALIFORNIA BEARING RATIO INVESTIGATION

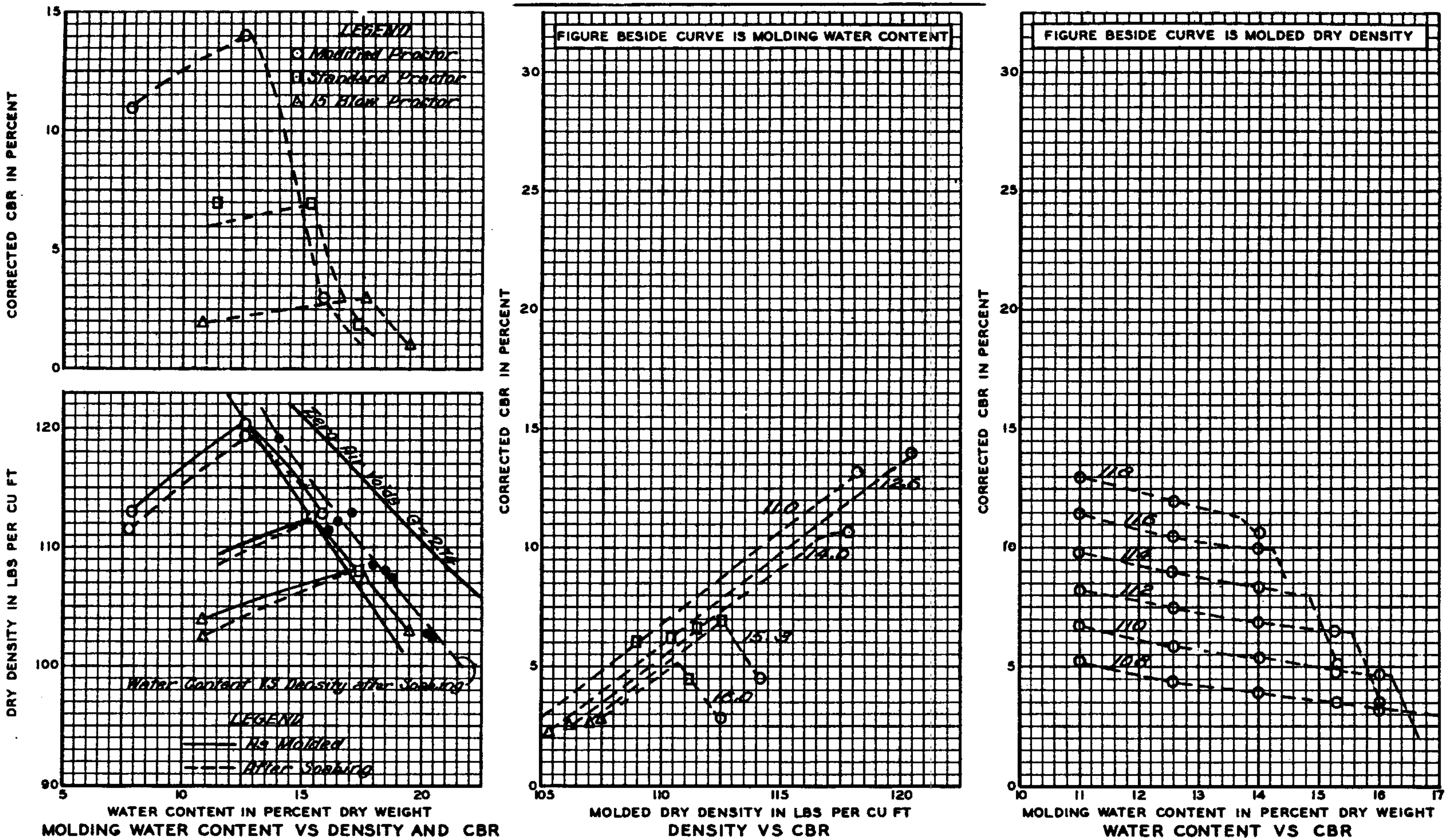
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 18 BERRY FIELD LEAN CLAY

PLATE 92

WAR DEPARTMENT CORPS OF ENGINEERS, U.S. ARMY

SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS

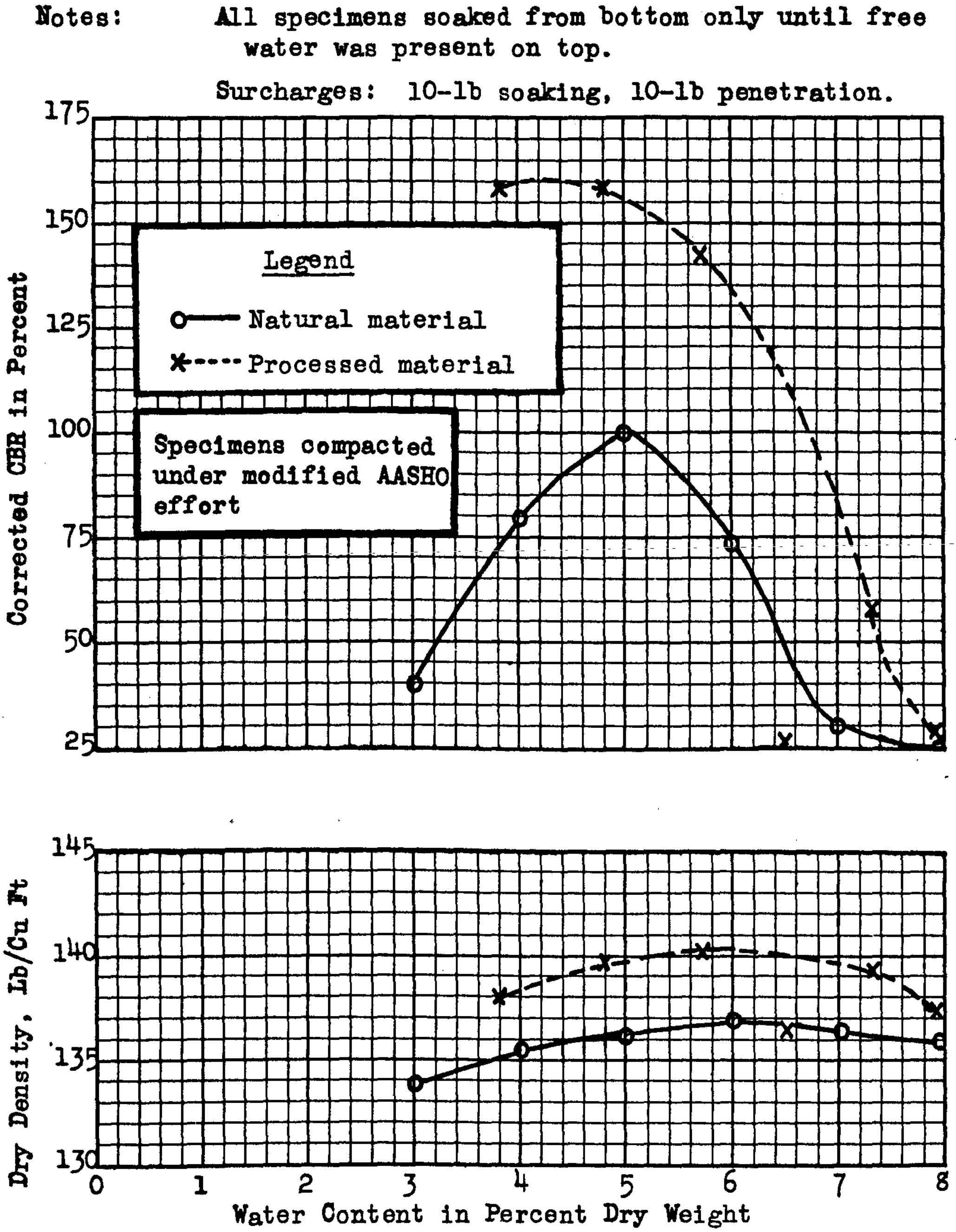


NOTES: SURCHARGE: 10LB. SOAKING, 0 PENETRATION.
3RD TIME MATERIAL USED FOR COMPACTION.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 18 — BERRY FIELD LEAN CLAY

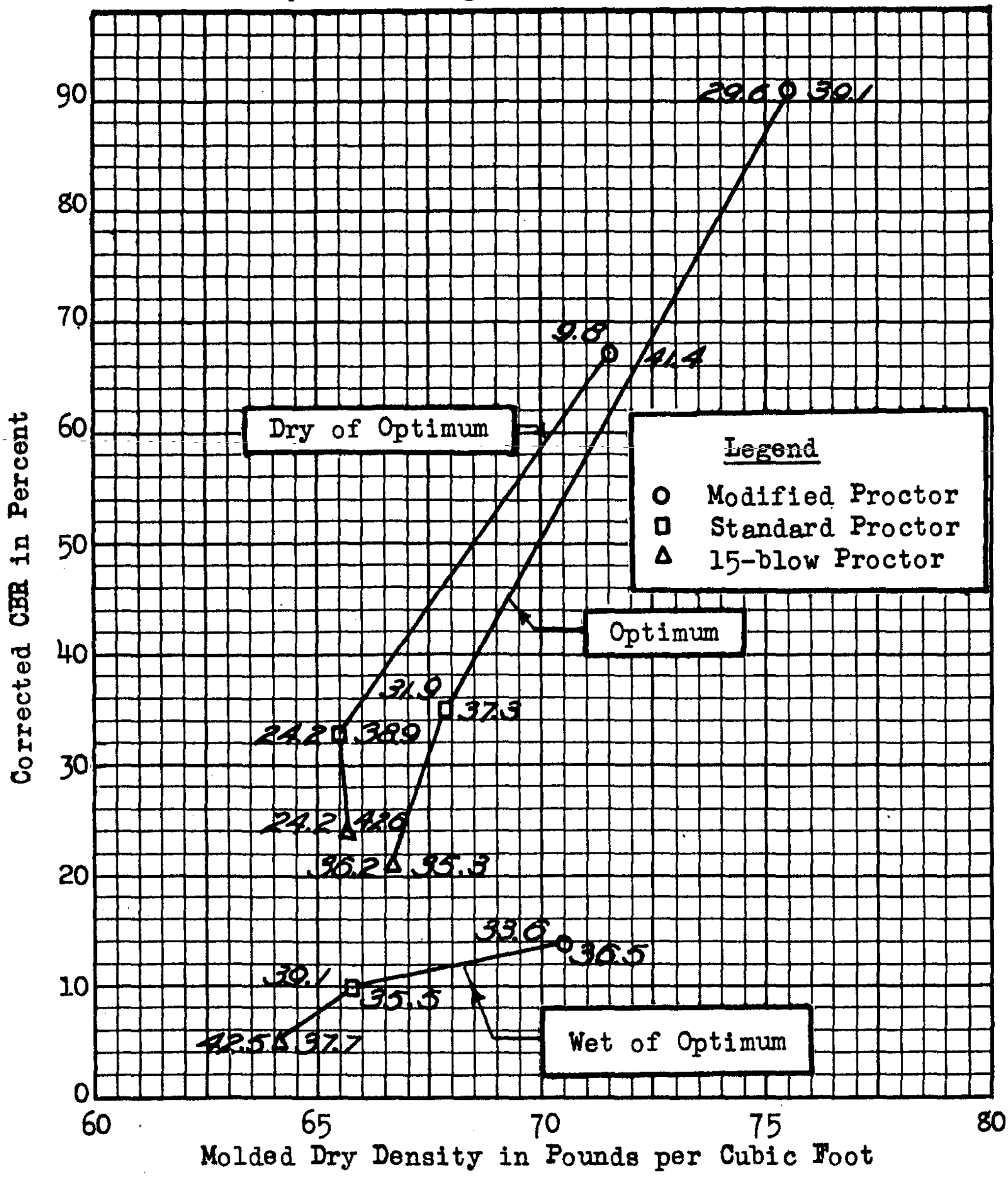


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MOLDING WATER CONTENT VS DRY DENSITY AND CBR

Sample 19 Well Graded Walesboro Gravel-Sand-Silt

Notes: Figure at left of each symbol is average molding water content. Figure at right of each symbol is water content of top inch of specimen after saturation and penetration. All points plotted are average of two tests. All specimens soaked from bottom only. Surcharges: 10 lb soaking, 0 penetration.

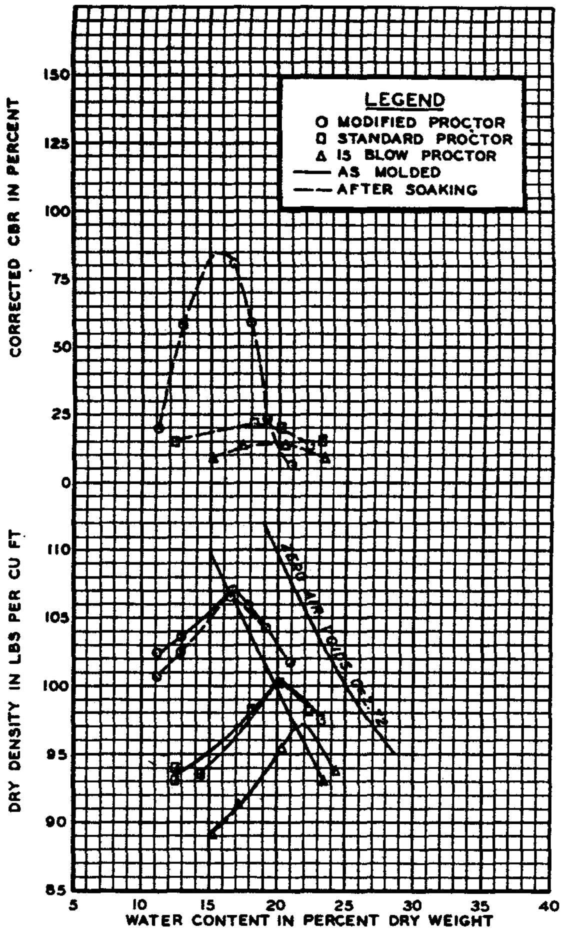


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

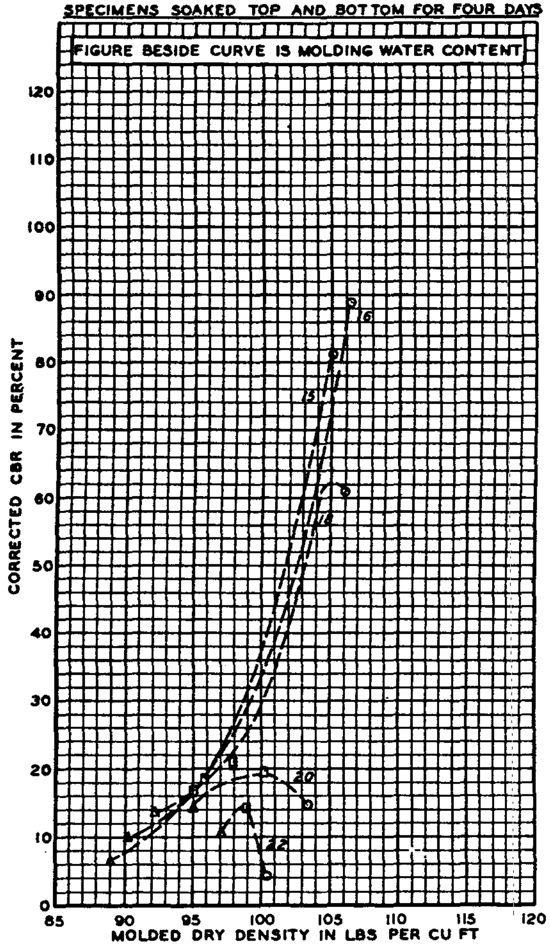
DENSITY VS CBR

Sample 21 California Pumice

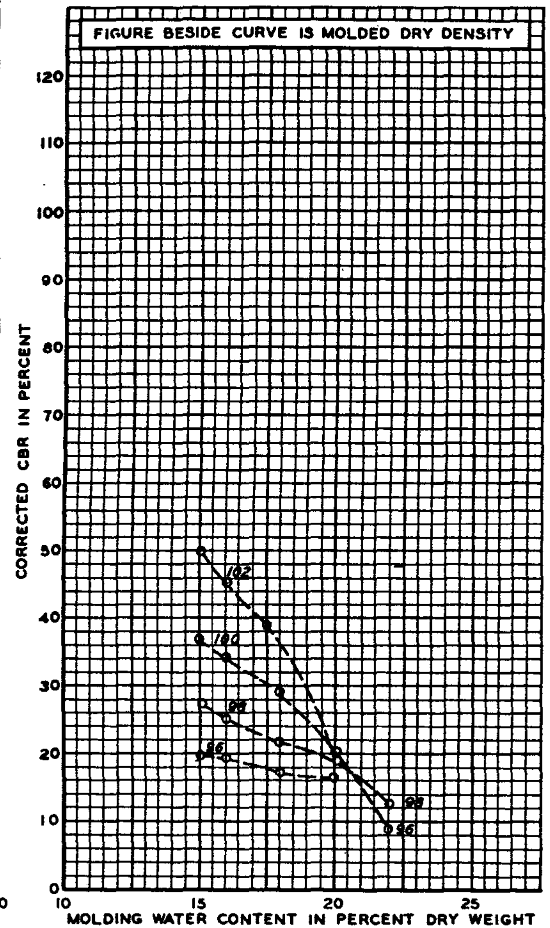
SPECIMENS SOAKED TOP AND BOTTOM FOR FOUR DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: SURCHARGE 10 LB SOAKING, 10 LB PENETRATION TESTS PERFORMED ON MATERIAL < 4 MESH

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION

CALIFORNIA BEARING RATIO INVESTIGATION

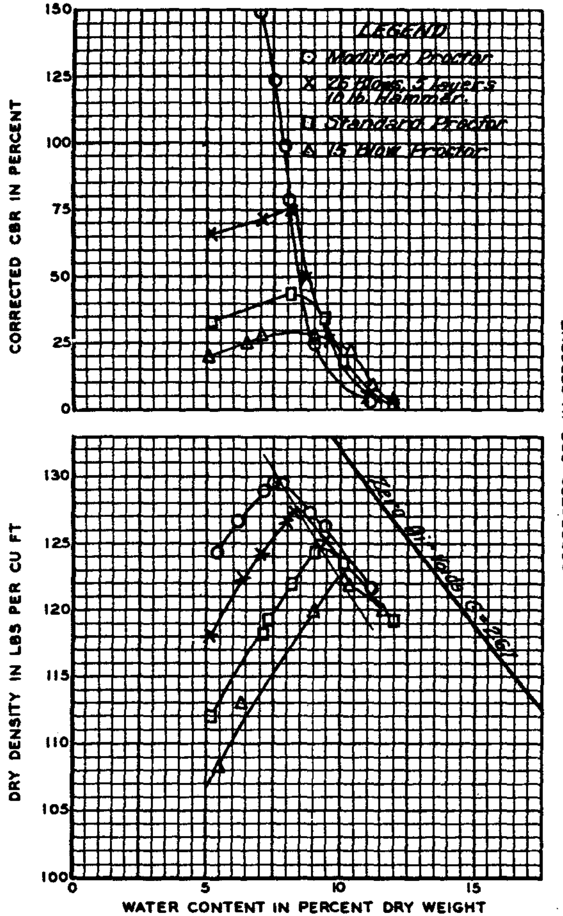
VARIATION OF CBR WITH DENSITY AND WATER CONTENT
SAMPLE 22 MISSION, TEXAS CALICHE

PLATE 96

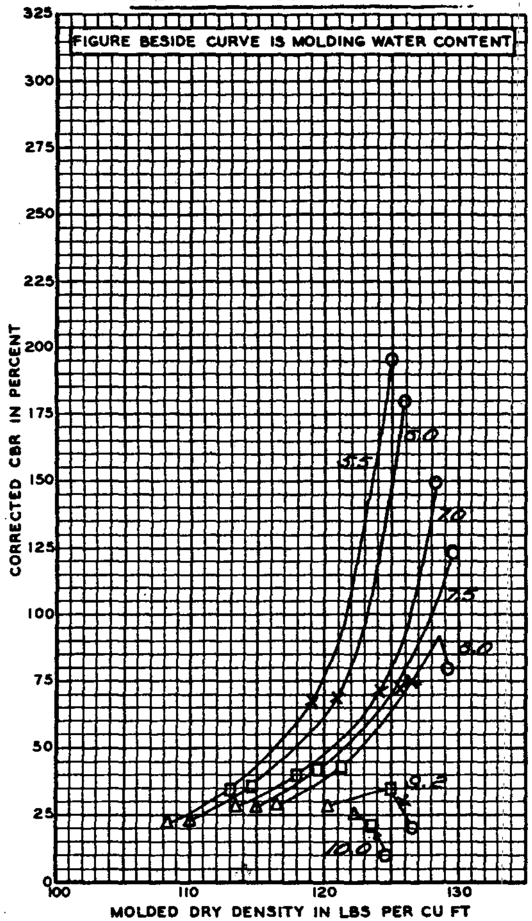
WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

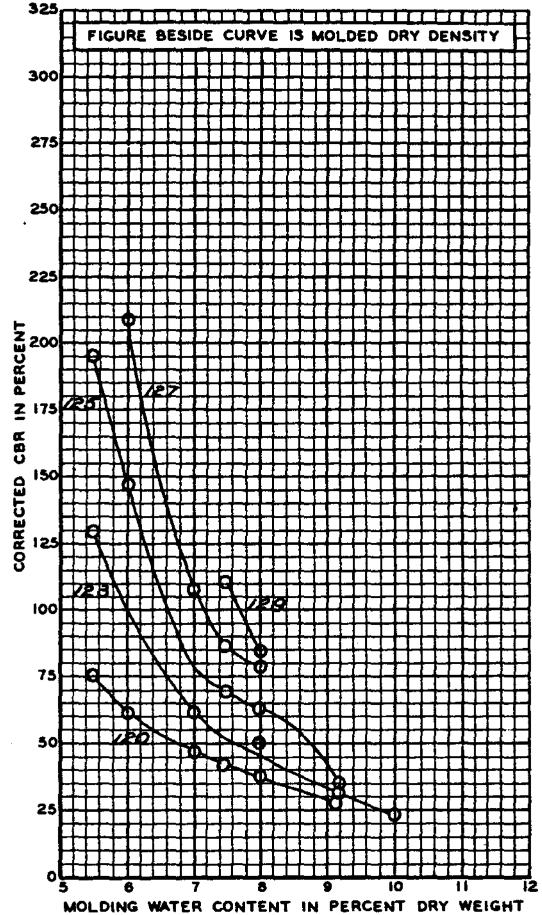
SPECIMENS TESTED AS MOLDED



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

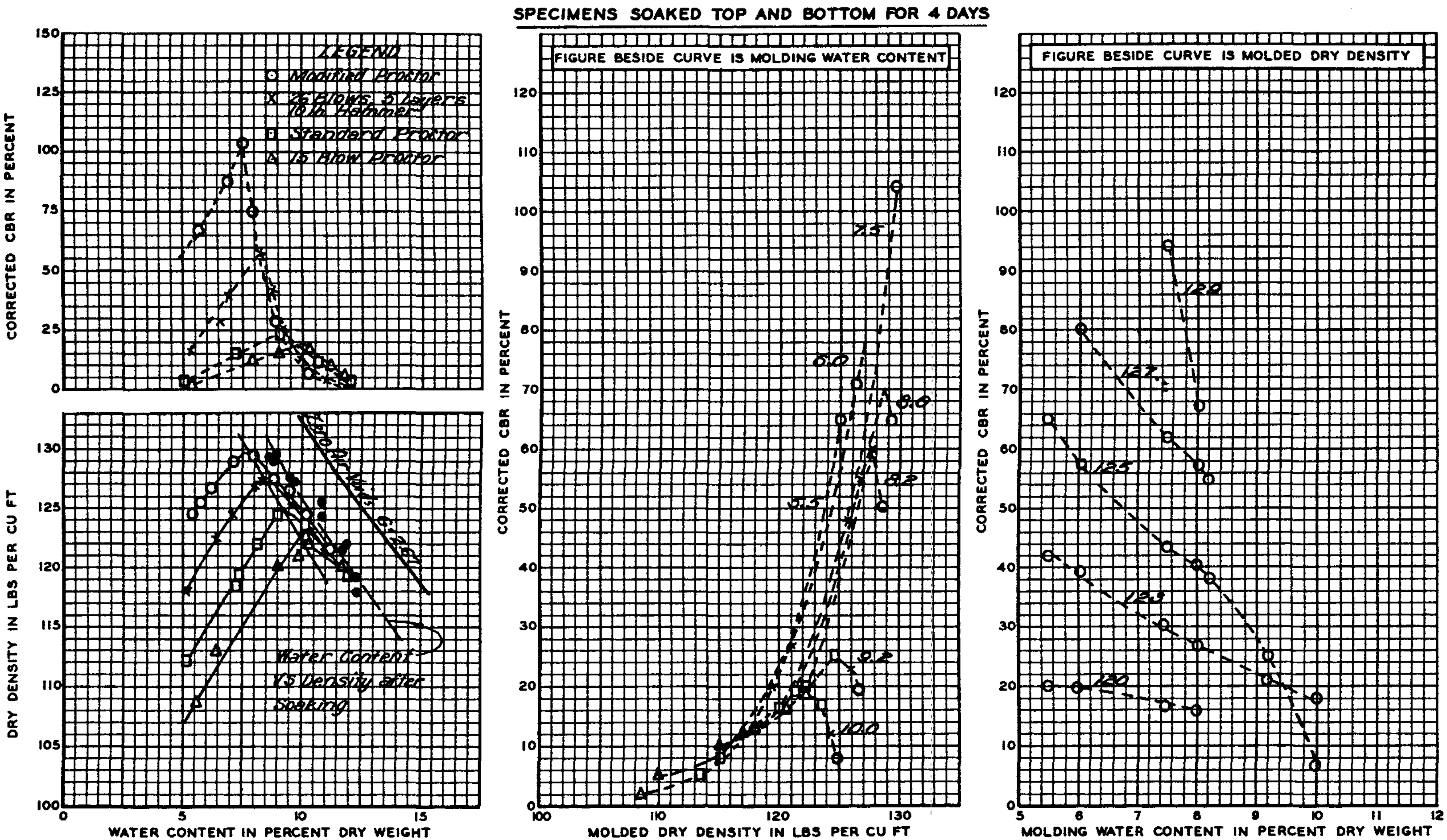
MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 23 — VICKSBURG CLAY SAND

NOTE: SURCHARGE: 10 LB. PENETRATION.

SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR

DENSITY VS CBR

WATER CONTENT VS CBR

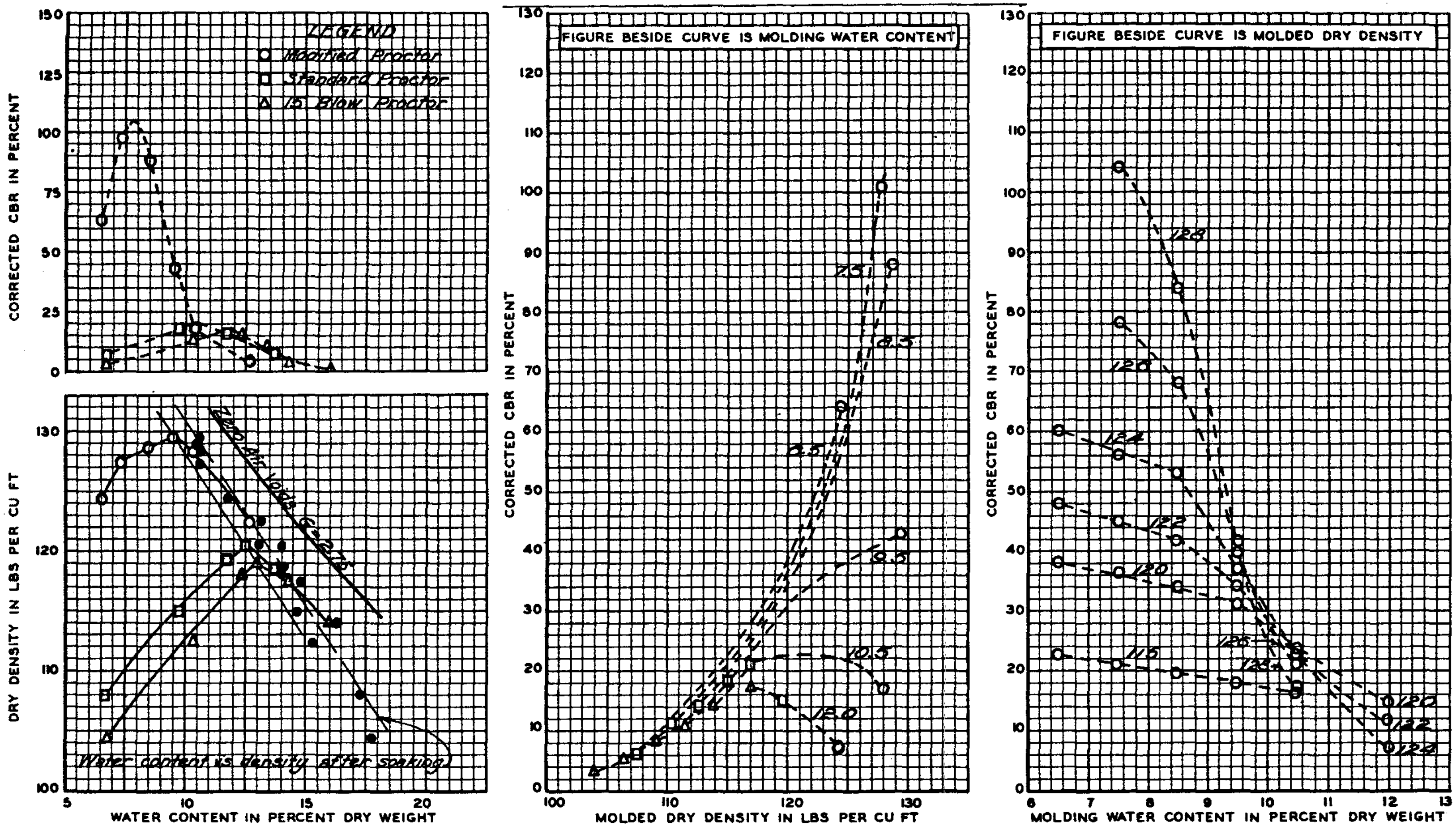
NOTES: SURCHARGES: 10 LB. SOAKING, 10 LB. PENETRATION.
NO SWELL DURING SOAKING.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

SAMPLE 23—VICKSBURG CLAY SAND

SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR

DENSITY VS CBR

WATER CONTENT VS CBR

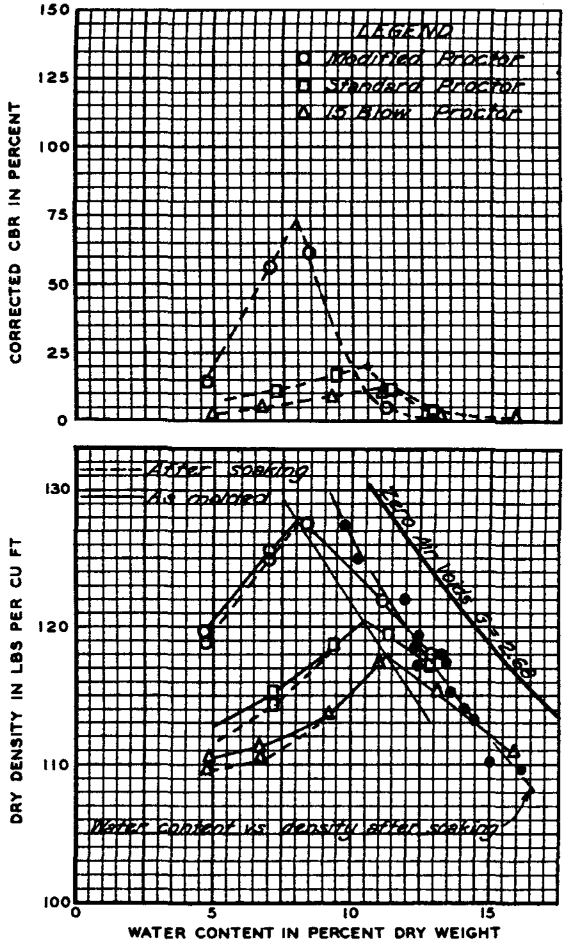
NOTES: SURCHARGE 10 LB SOAKING, 0 PENETRATION.
NO SWELL DURING SOAKING.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

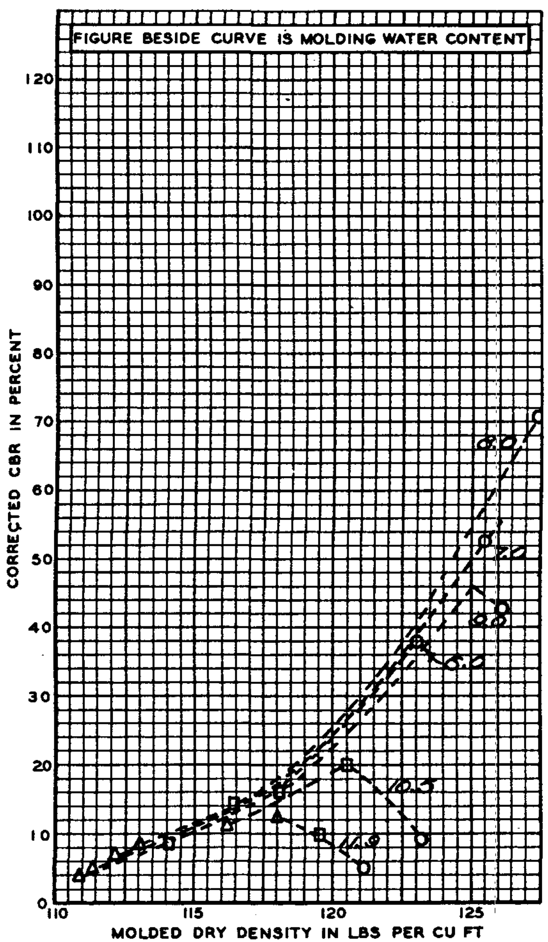
VARIATION OF CBR WITH DENSITY AND WATER CONTENT

MARIETTA MIX CLAY SAND

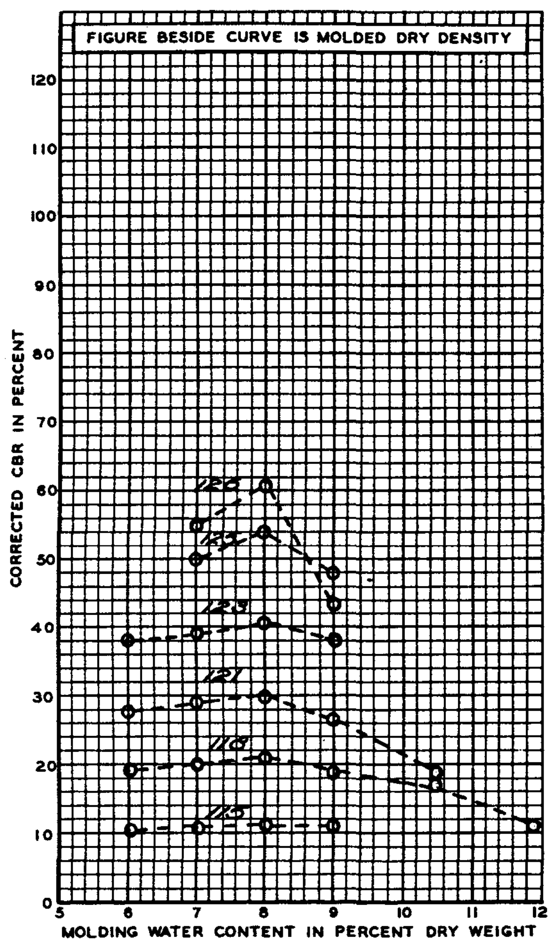
SPECIMENS SOAKED TOP AND BOTTOM FOR 2 TO 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

NOTES: SURCHARGES: 10 LB SOAKING. 10 LB PENETRATION.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT

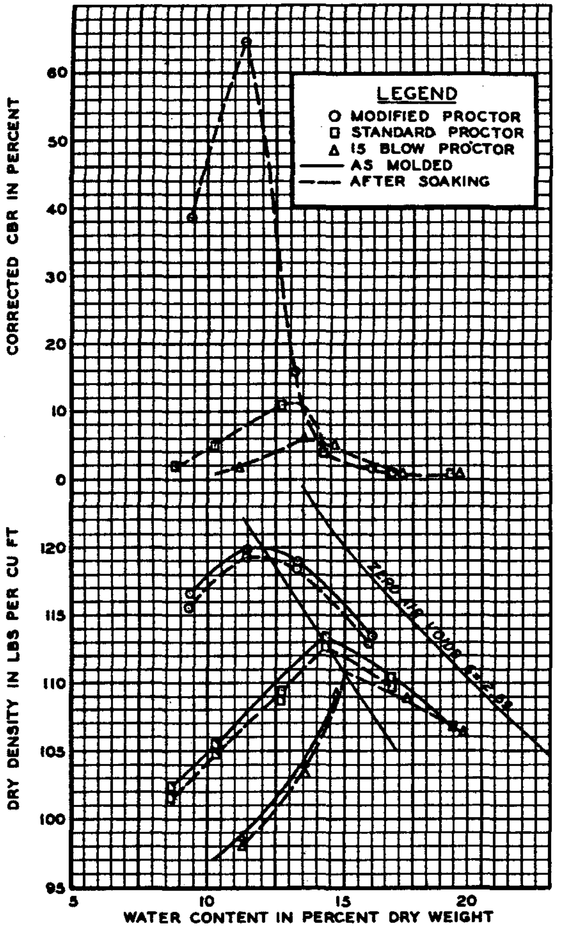
KELLOGG FIELD CLAY SAND

PLATE 100

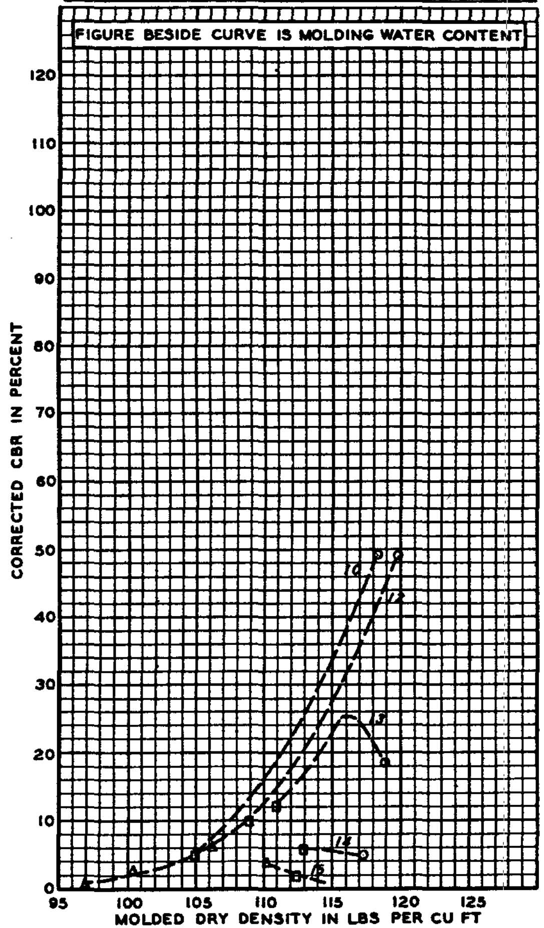
WAR DEPARTMENT

CORPS OF ENGINEERS. U.S. ARMY

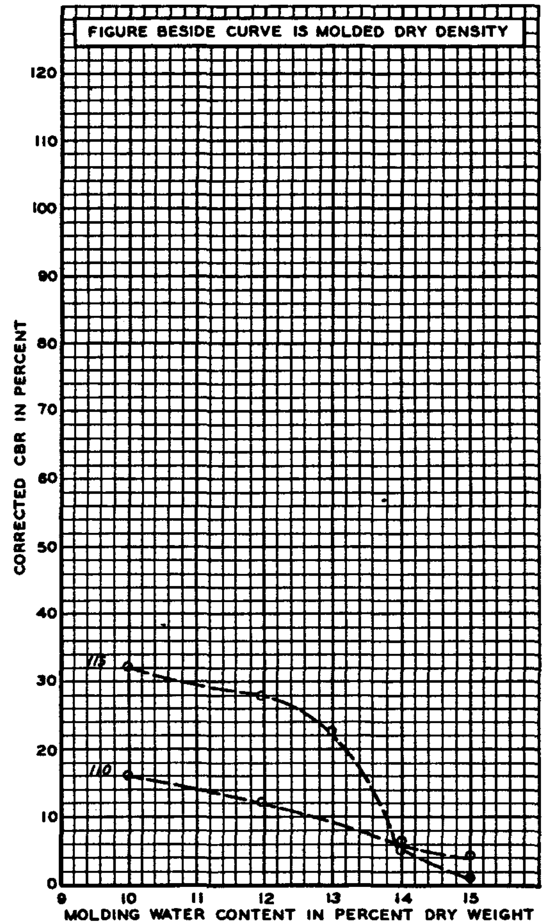
SPECIMENS SOAKED TOP AND BOTTOM FOR FOUR DAYS



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR

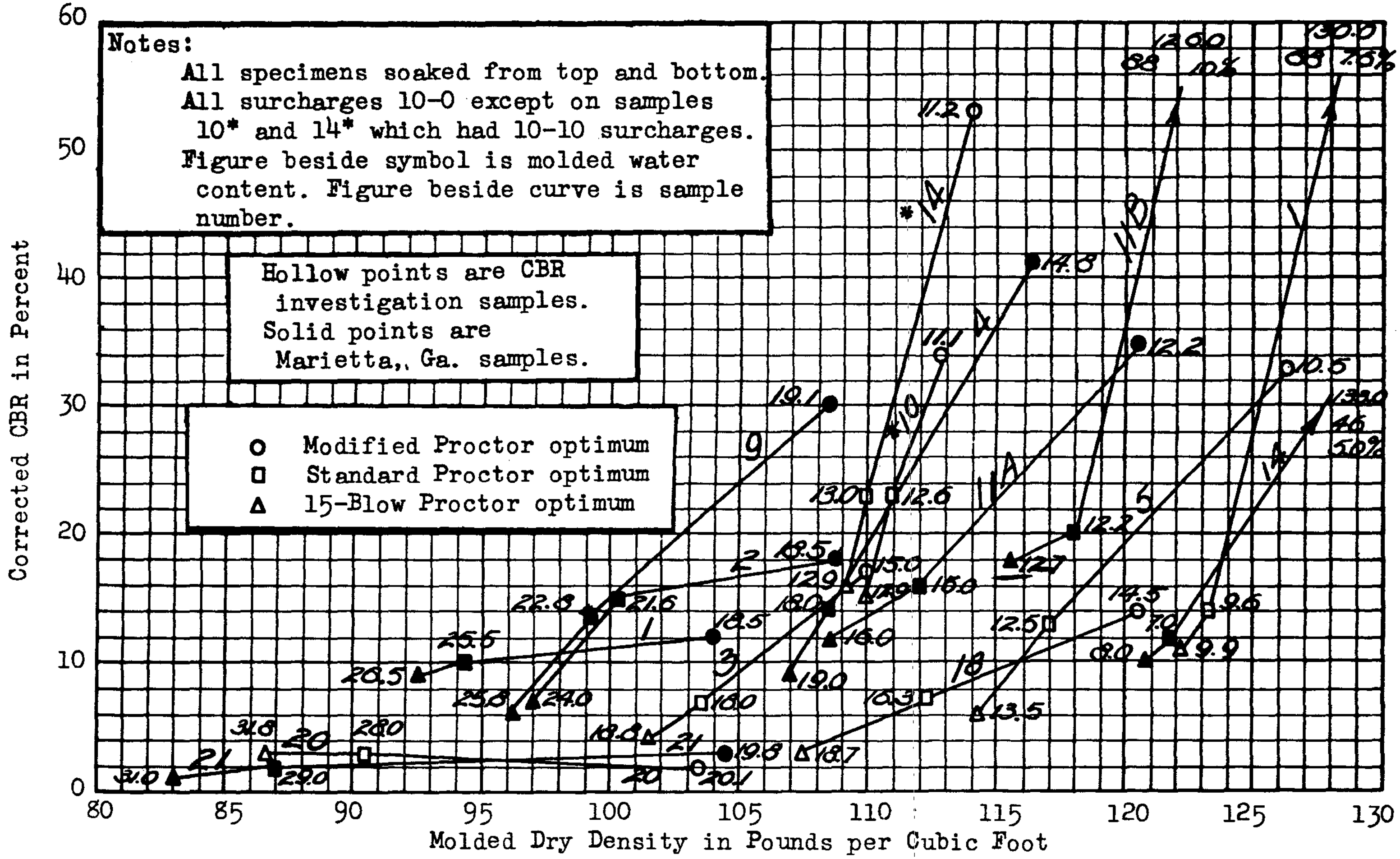


WATER CONTENT VS CBR

NOTES: SURCHARGE 10 LB SOAKING, 10 LB PENETRATION
TESTS PERFORMED ON MATERIAL < 4 MESH

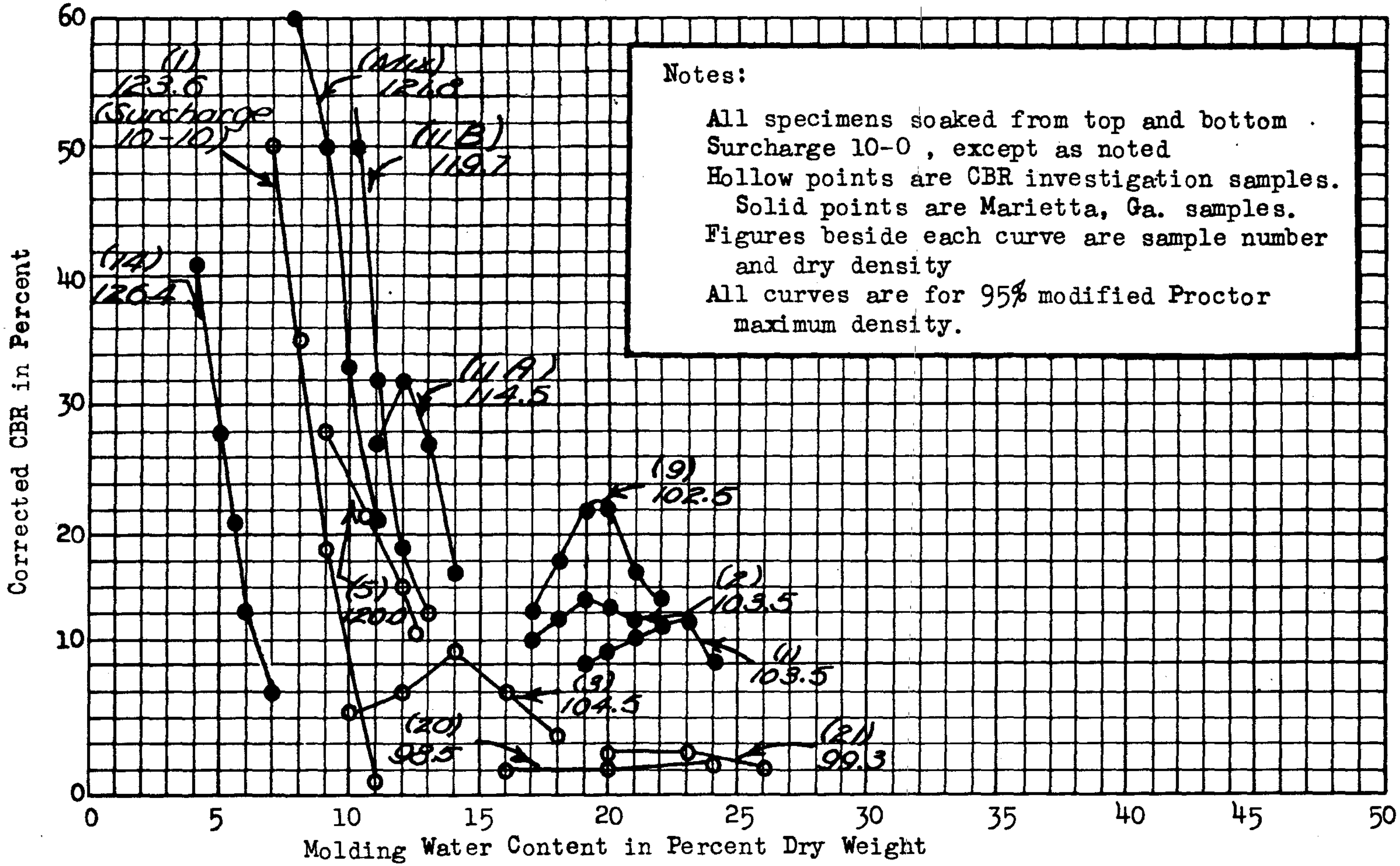
MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF CBR WITH DENSITY AND WATER CONTENT
SAMPLE 25 GEORGETOWN, TEXAS CALICHE

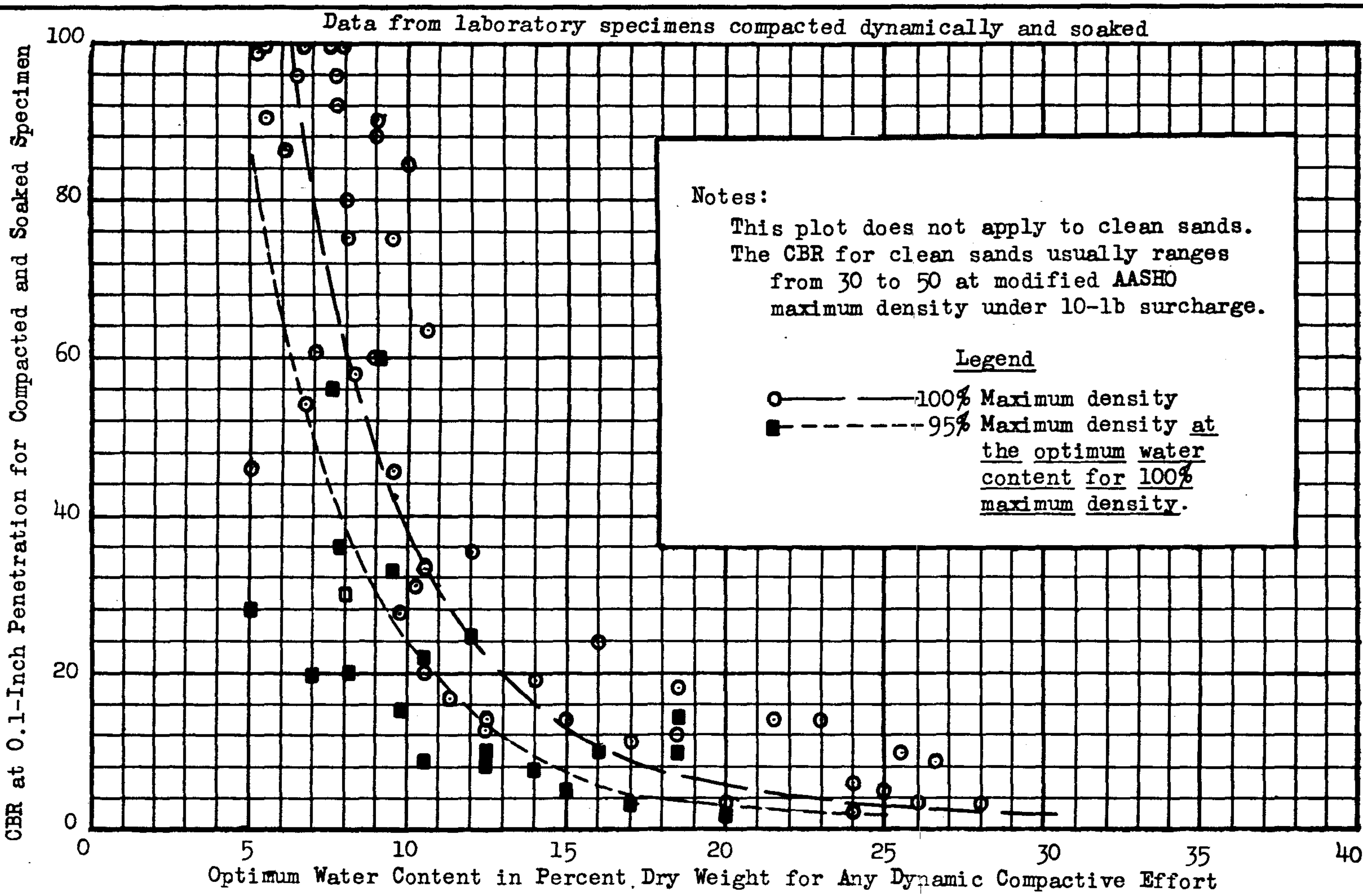


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

CBR VS MOLDED DRY DENSITY (AT OPTIMUM)

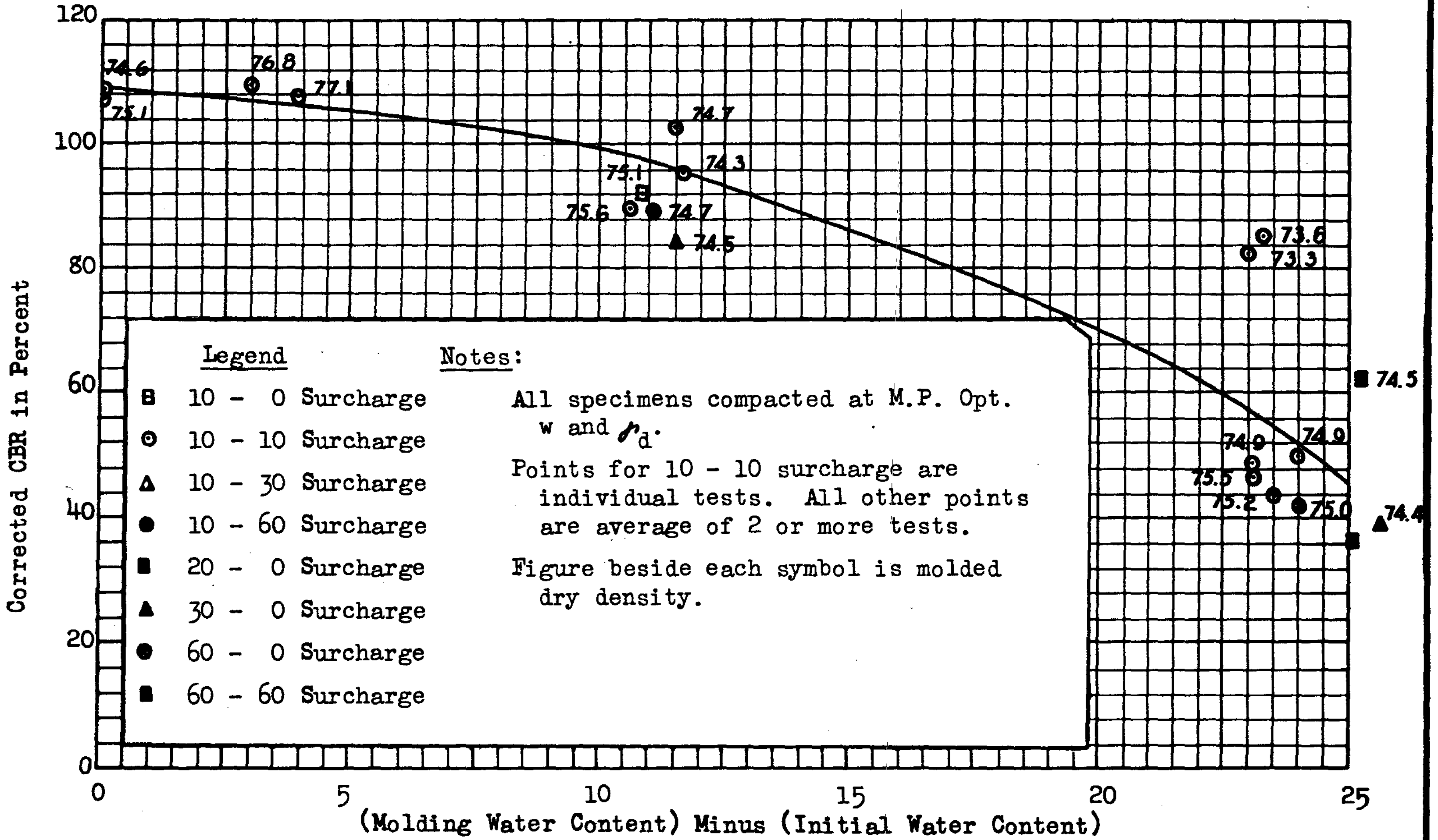


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation
MOLDING WATER CONTENT VS CBR AT 95 PERCENT MODIFIED PROCTOR MAXIMUM DENSITY



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

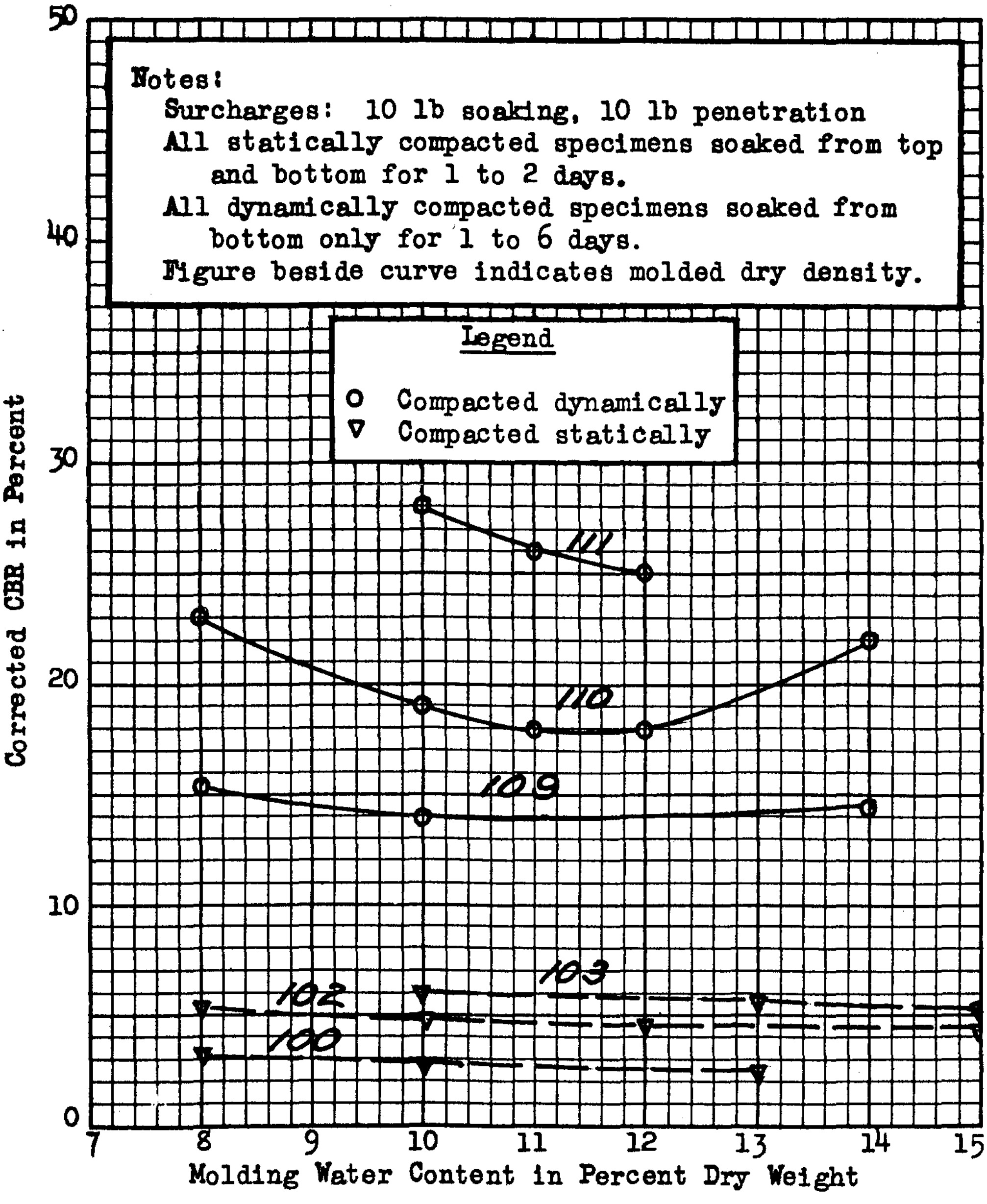
CBR VS OPTIMUM WATER CONTENT



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation
CBR VS $(\omega_m - \omega_i)$

Sample 21

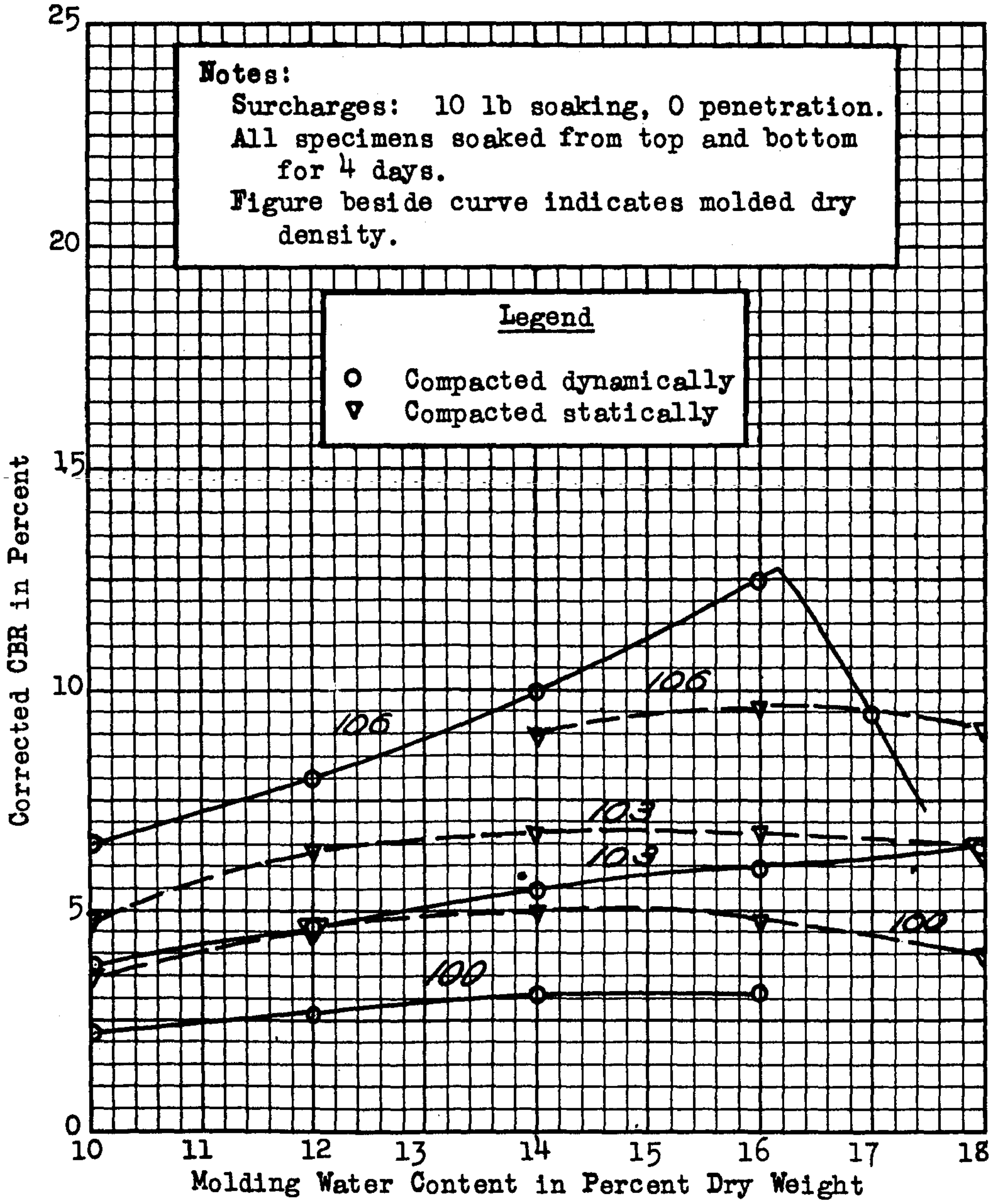
California Pumice



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPARISON OF CBR OF DYNAMICALLY COMPACTED
WITH STATICALLY COMPACTED SPECIMENS

Sample 10 Eglin Field Sand

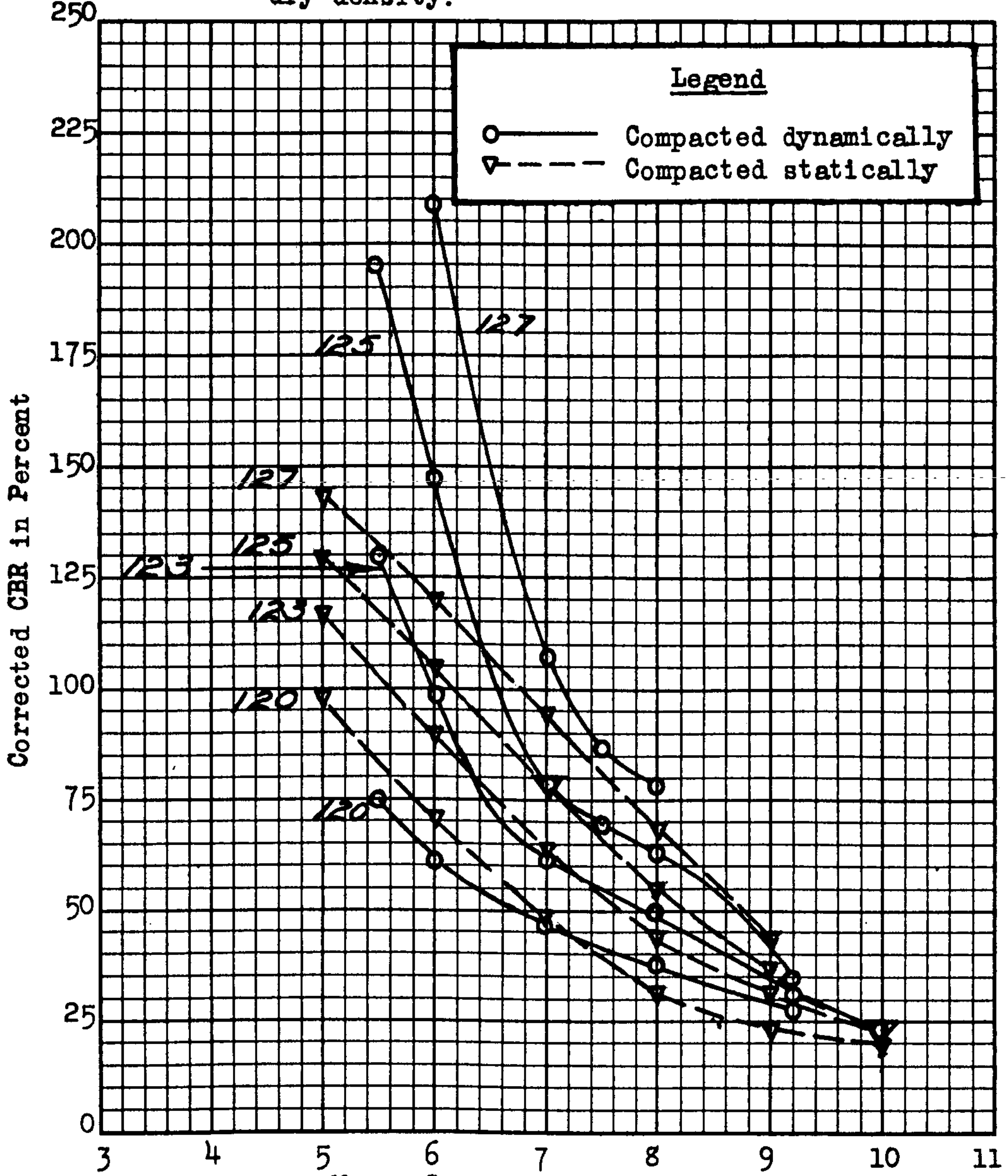


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPARISON OF CBR OF DYNAMICALLY COMPACTED WITH STATICALLY COMPACTED SPECIMENS

Sample 3 Vicksburg Loess

Notes: Specimens tested as molded.
Surcharges: 10 lb penetration.
Figure beside curve indicates molded dry density.

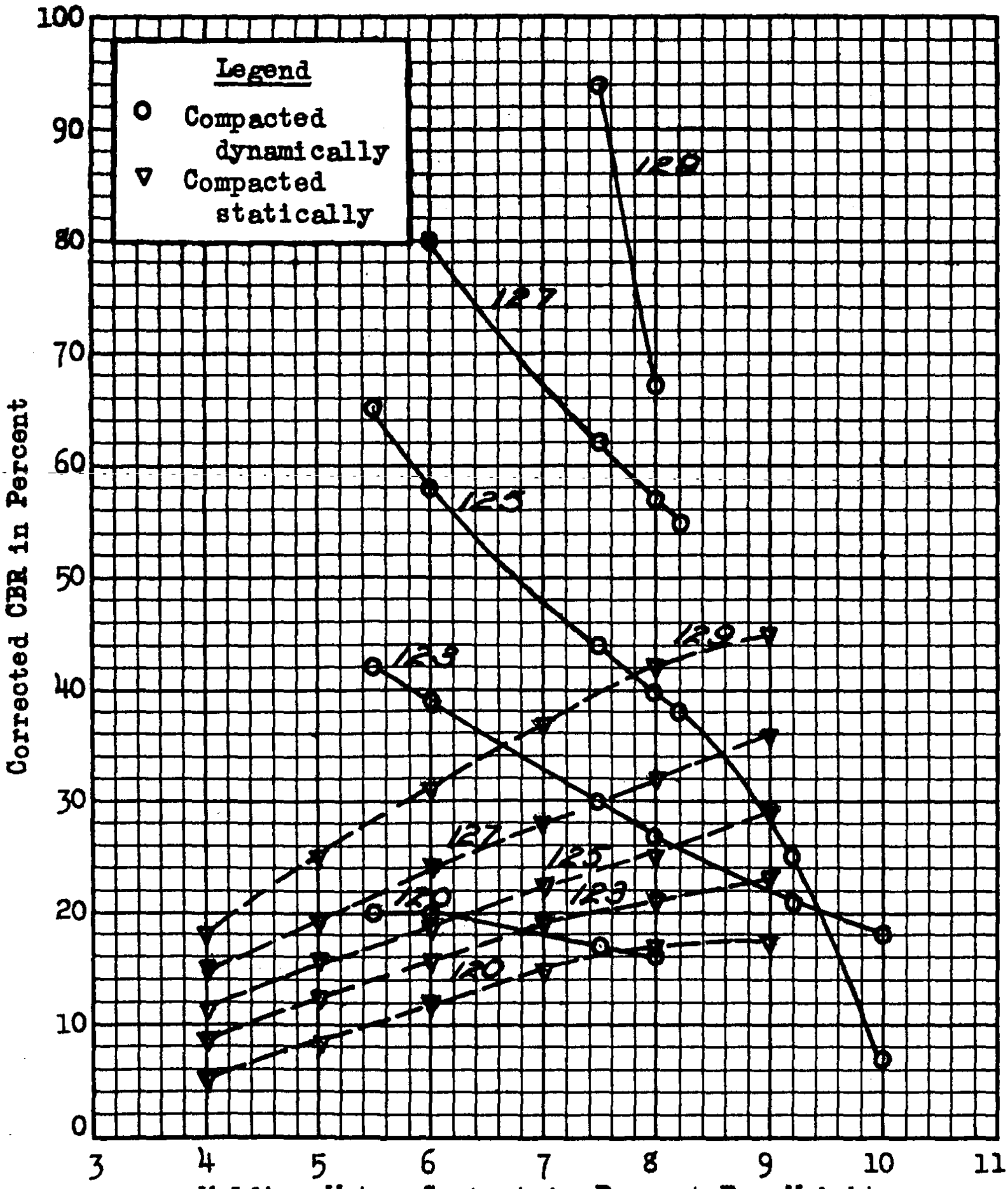


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPARISON OF CBR OF DYNAMICALLY COMPACTED WITH STATICALLY COMPACTED SPECIMENS

Sample 23 Vicksburg Clay Sand

Notes: Specimens soaked top and bottom for 4 days.
Surcharges: 10 lb soaking, 10 lb penetration.
Figure beside curve indicates molded dry density.



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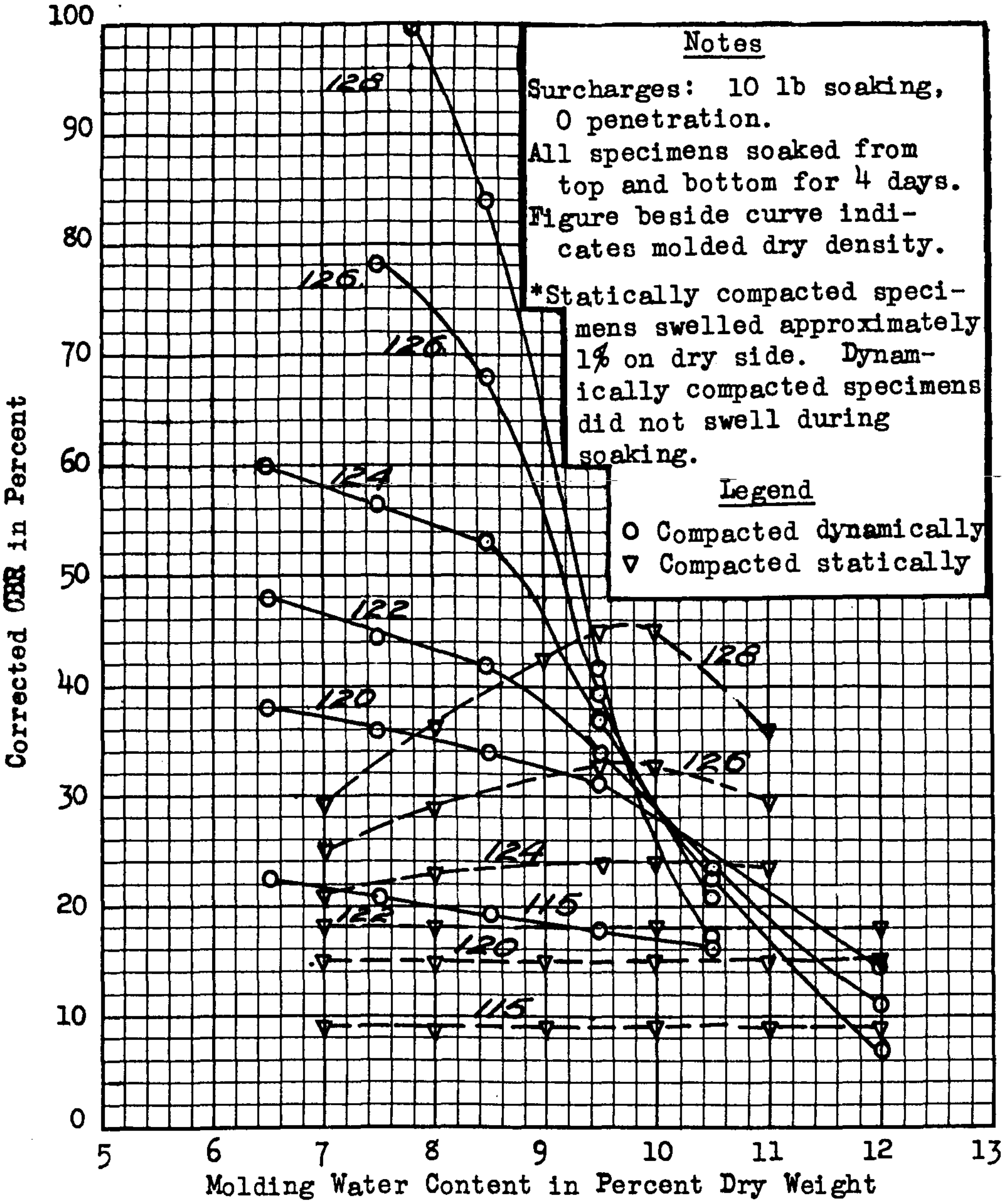
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPARISON OF CBR OF DYNAMICALLY COMPACTED
WITH STATICALLY COMPACTED SPECIMENS

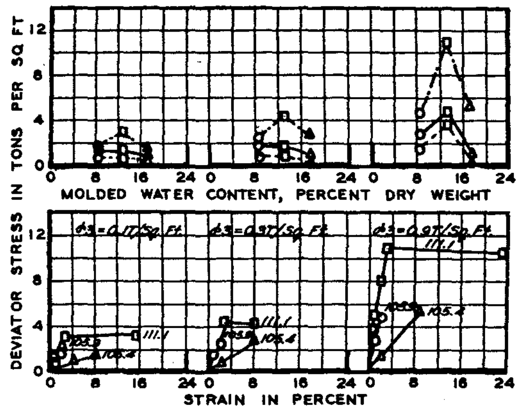
Sample 23 Vicksburg Clay Sand



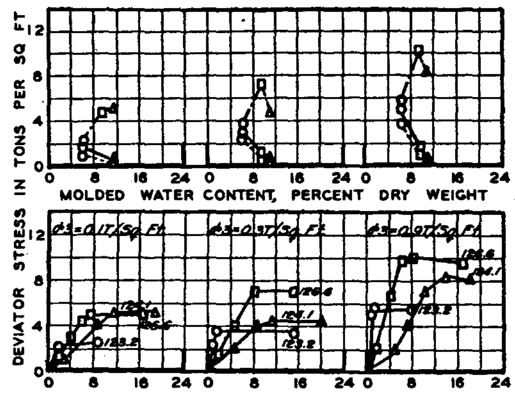
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPARISON OF CBR OF DYNAMICALLY COMPACTED
WITH STATICALLY COMPACTED SPECIMENS

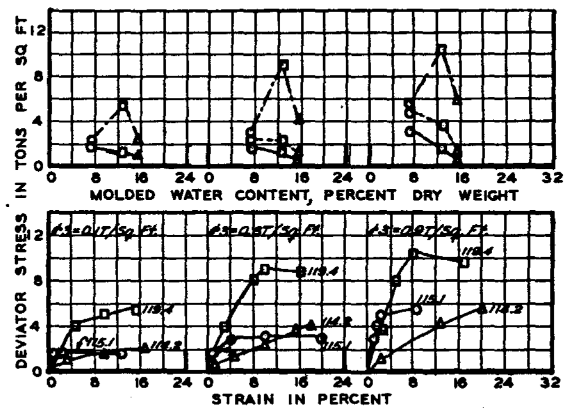
Marietta mix Clay Sand



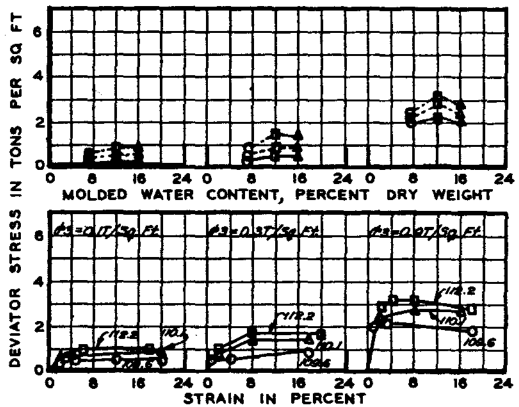
SAMPLE 3 VICKSBURG LOESS



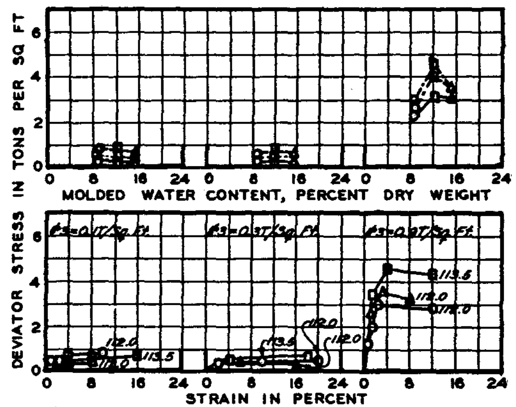
SAMPLE 5 DE RIDDER SAND CLAY (LOAM)



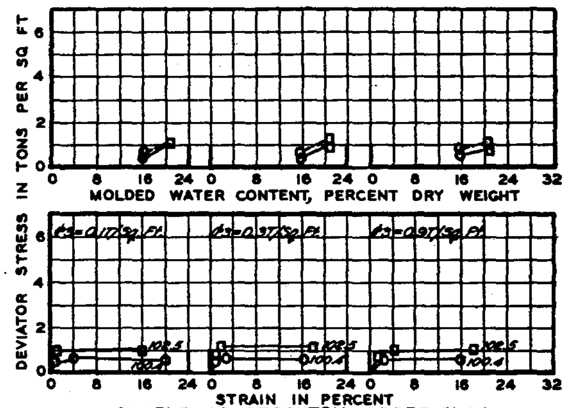
SAMPLE 18 BERRY FIELD LEAN CLAY



SAMPLE 10 EGLIN FIELD SAND



SAMPLE 14 MANCHESTER SAND



SAMPLE 20 STOCKTON ADOBE CLAY

LEGEND

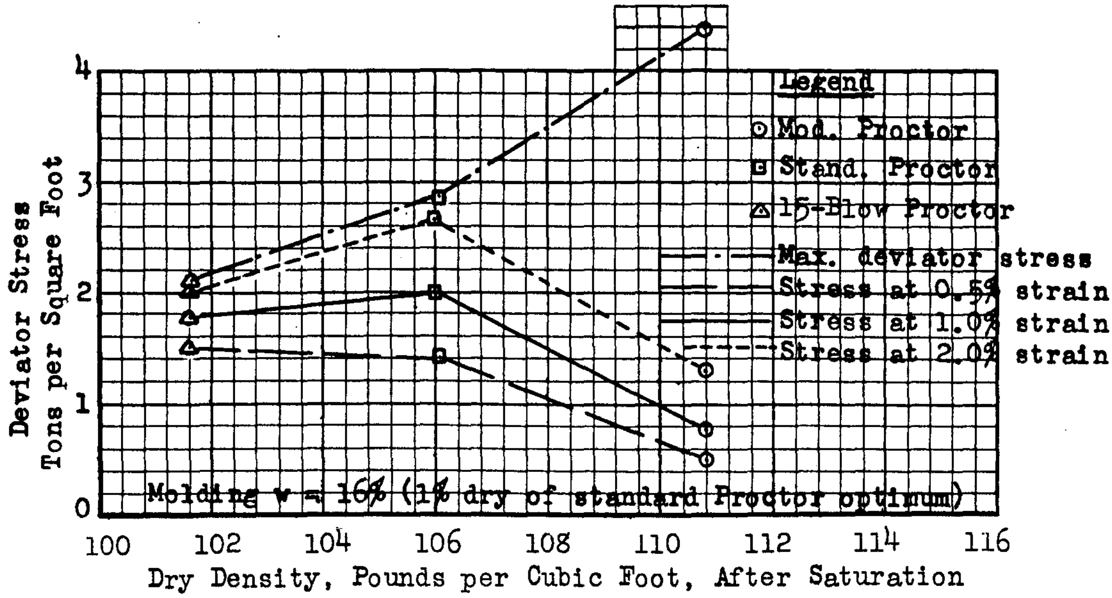
- O DRY OF OPTIMUM
- □ OPTIMUM
- Δ WET OF OPTIMUN
- — — — STRESS AT 0.5 % STRAIN
- ——— STRESS AT 1.0 % STRAIN
- - - - - - STRESS AT 2.0 % STRAIN
- —·— MAXIMUM STRESS

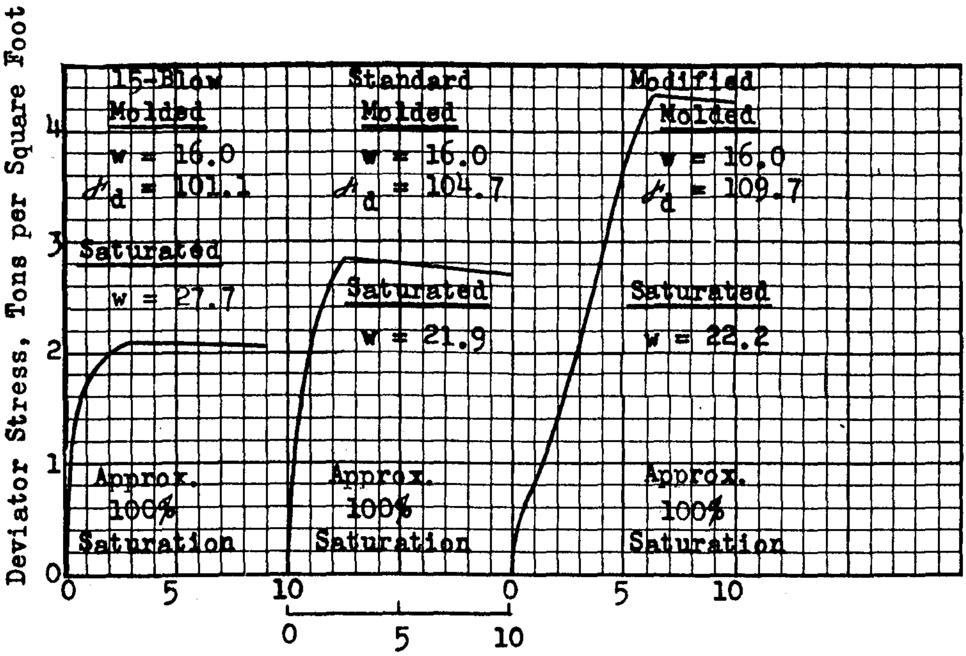
NOTE: MODIFIED PROCTOR COMPACTION FIGURE BESIDE CURVE INDICATES MOLDED DRY DENSITY.

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

DEVIATOR STRESS VS STRAIN AND MOLDED WATER CONTENT FOR Q_c SATURATED TRIAXIAL COMPRESSION TESTS

SAMPLES 3, 5, 10, 14, 18 AND 20





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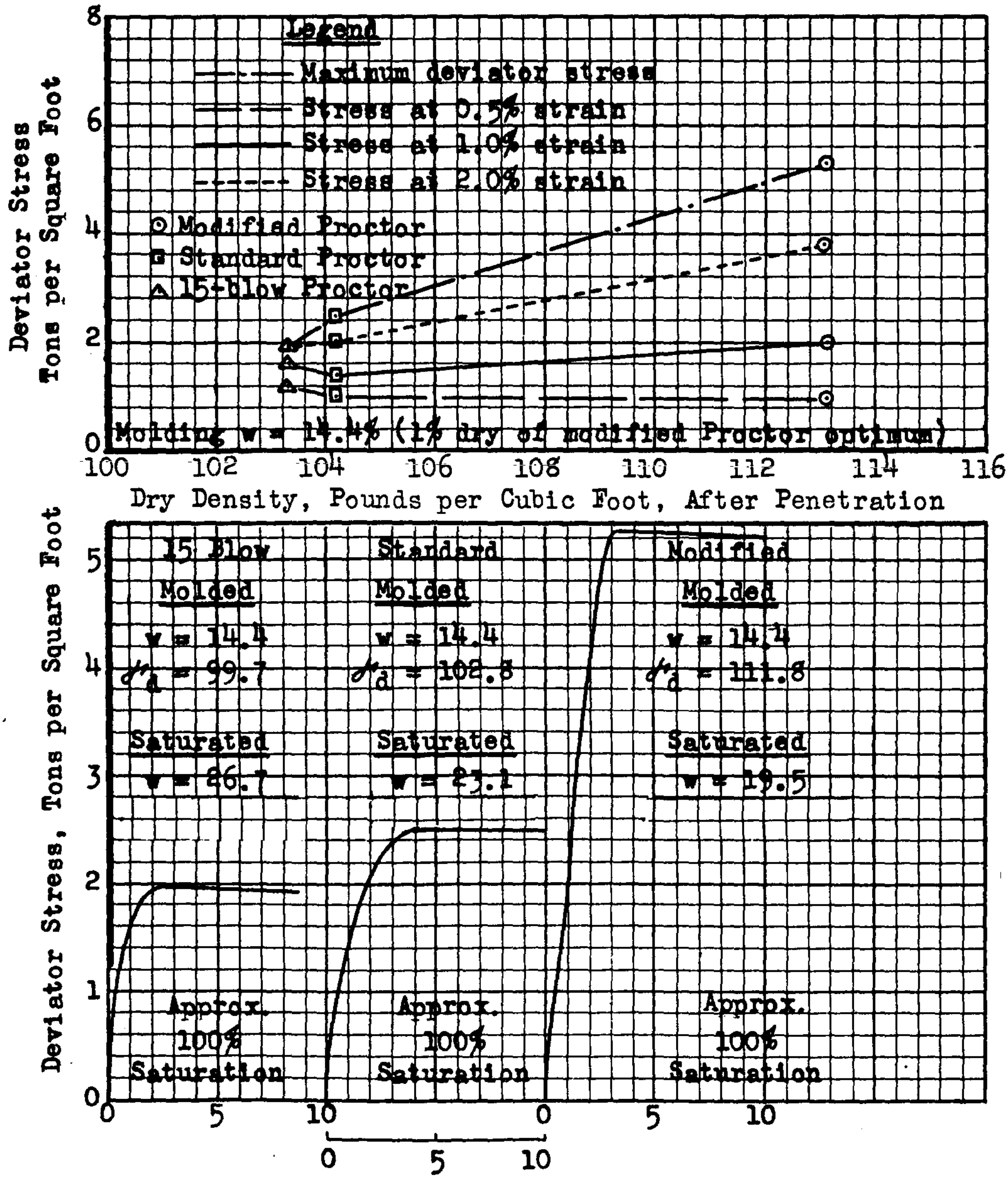
California Bearing Ratio Investigation

DEVIATOR STRESS VS STRAIN AND DRY DENSITY FOR Q_c SATURATED TRIAXIAL COMPRESSION TESTS

$\sigma_3 = 0.3\ kg/cm^2$

Sample 3 Vicksburg Loess



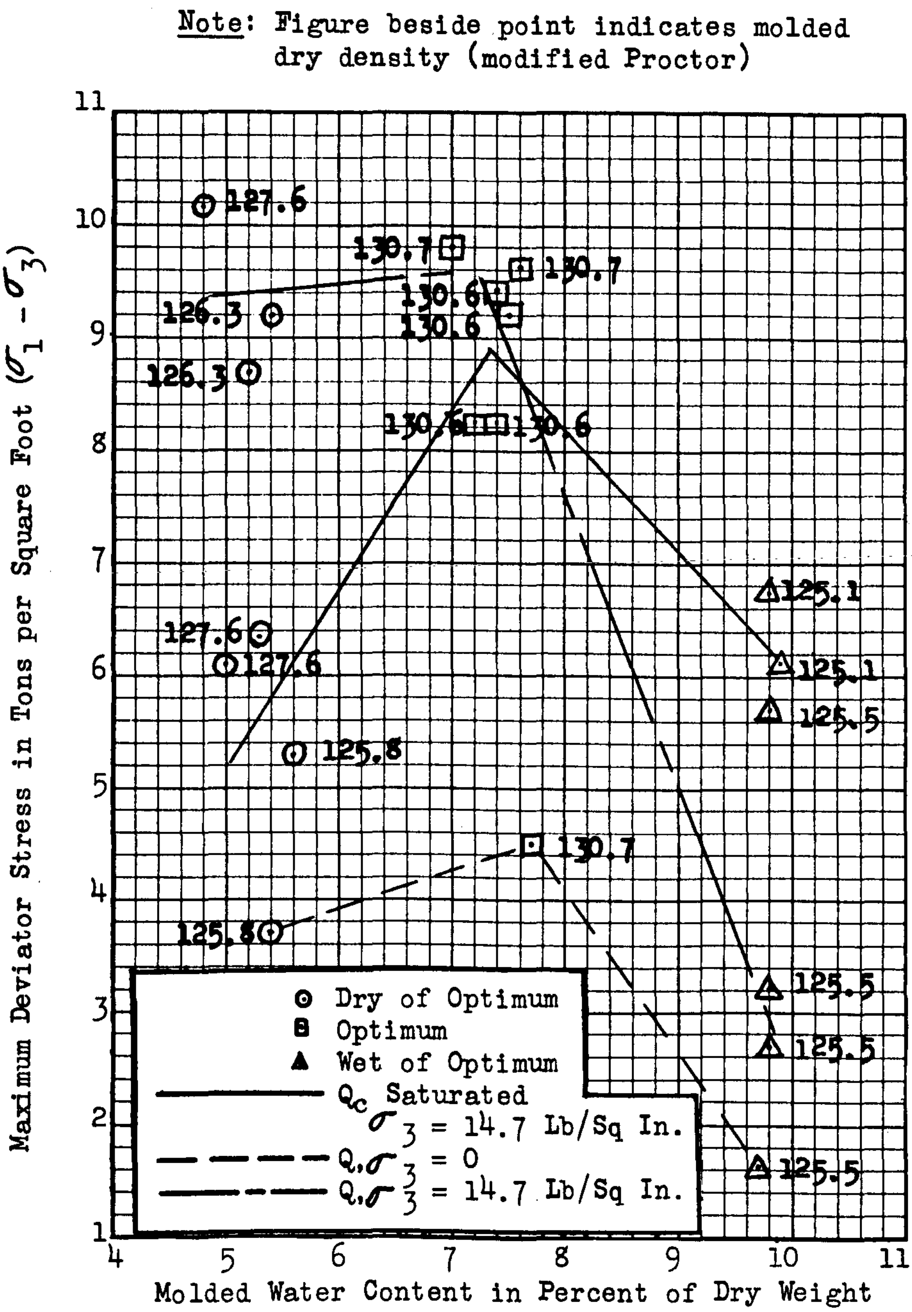
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

DEVIATOR STRESS VS STRAIN AND DRY DENSITY FOR Q_c SATURATED

TRIAXIAL COMPRESSION TESTS

$\sigma_3 = 0.3\ kg/cm^2$

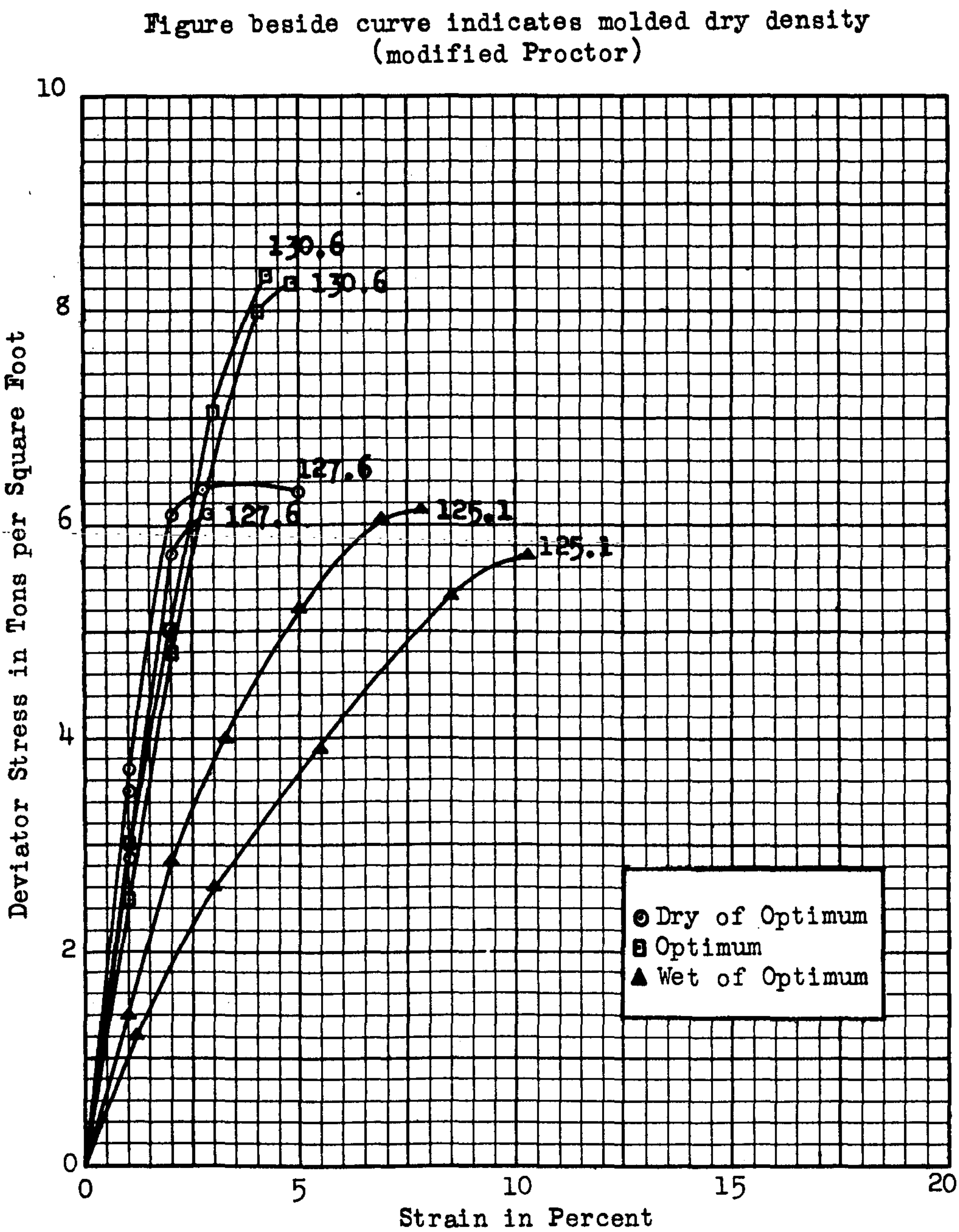
Sample 3 Vicksburg Loess



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MAXIMUM DEVIATOR STRESS VS MOLDED WATER CONTENT
Triaxial Compression Test

Sample 1 DeRidder Clay Sand



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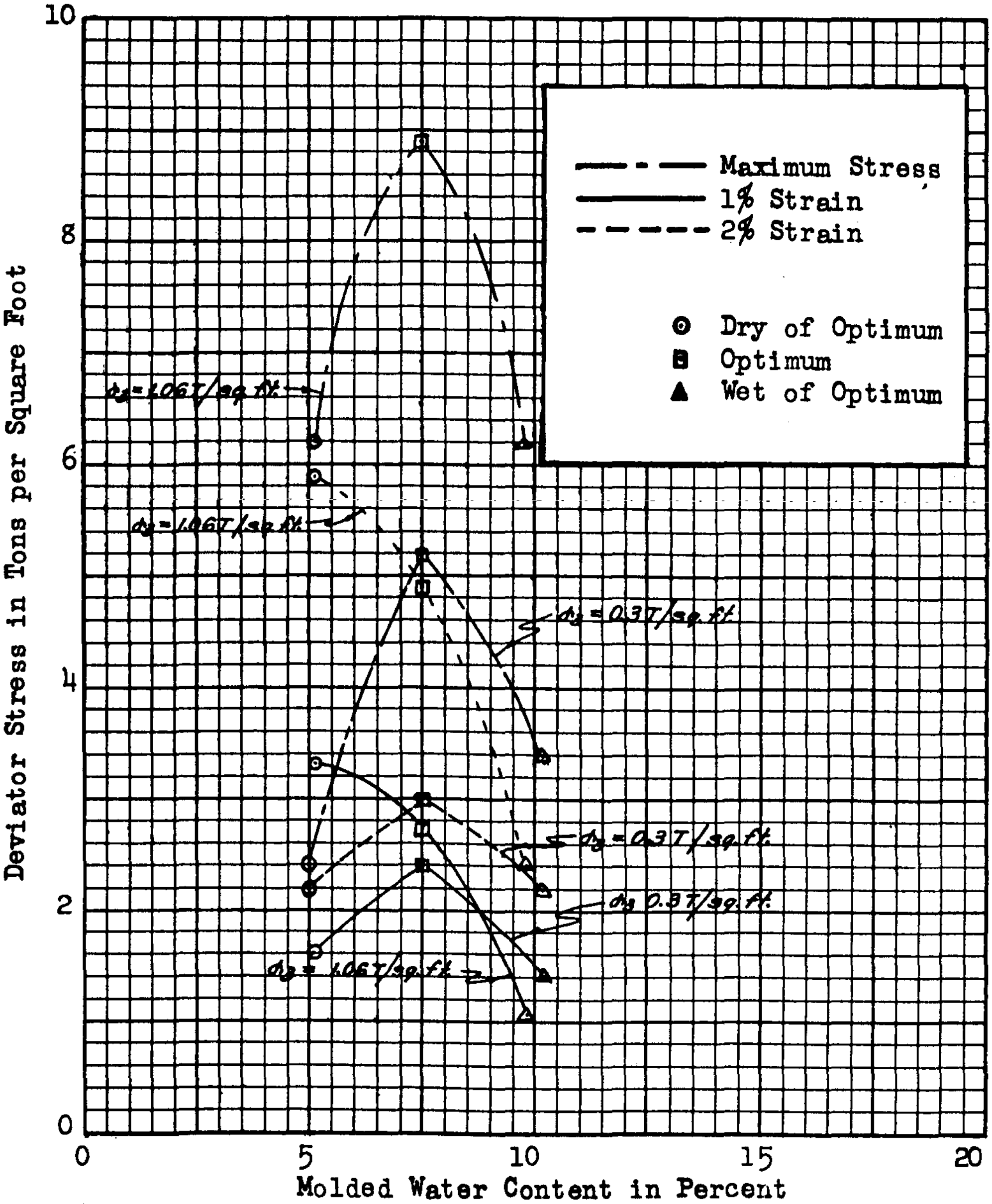
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

STRESS VS STRAIN CURVES FOR Q_c SATURATED TRIAXIAL COMPRESSION TESTS
(σ_3 = 14.7 Lb per Sq In.)

Sample 1 DeRidder Clay Sand

Specimens compacted by modified Proctor method



Mississippi River Commission
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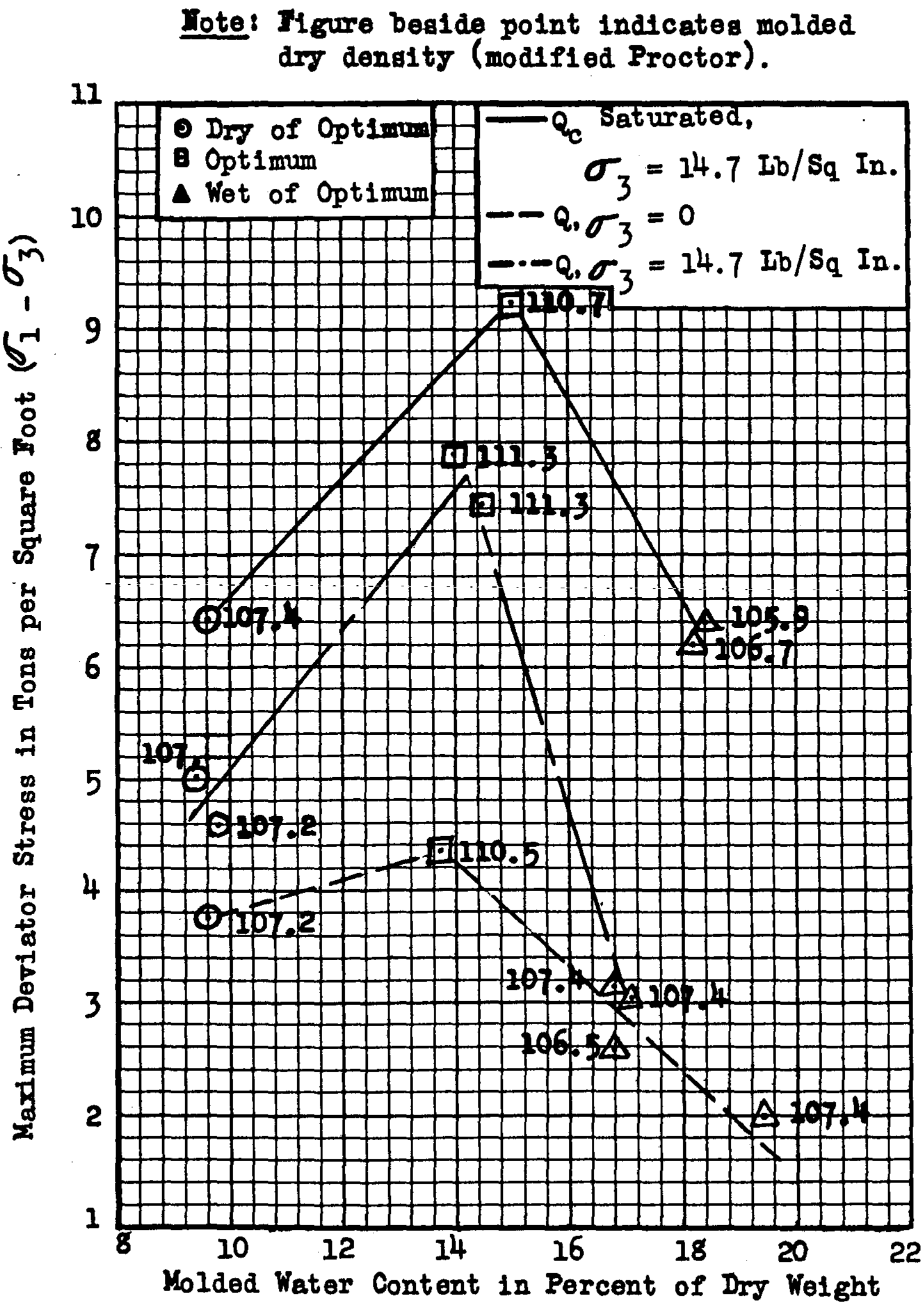
California Bearing Ratio Investigation

DEVIATOR STRESS VS MOLDED WATER CONTENT FOR Q_c SATURATED TRIAXIAL COMPRESSION TESTS

Sample 1 DeRidder Clay Sand



Mississippi River Commission
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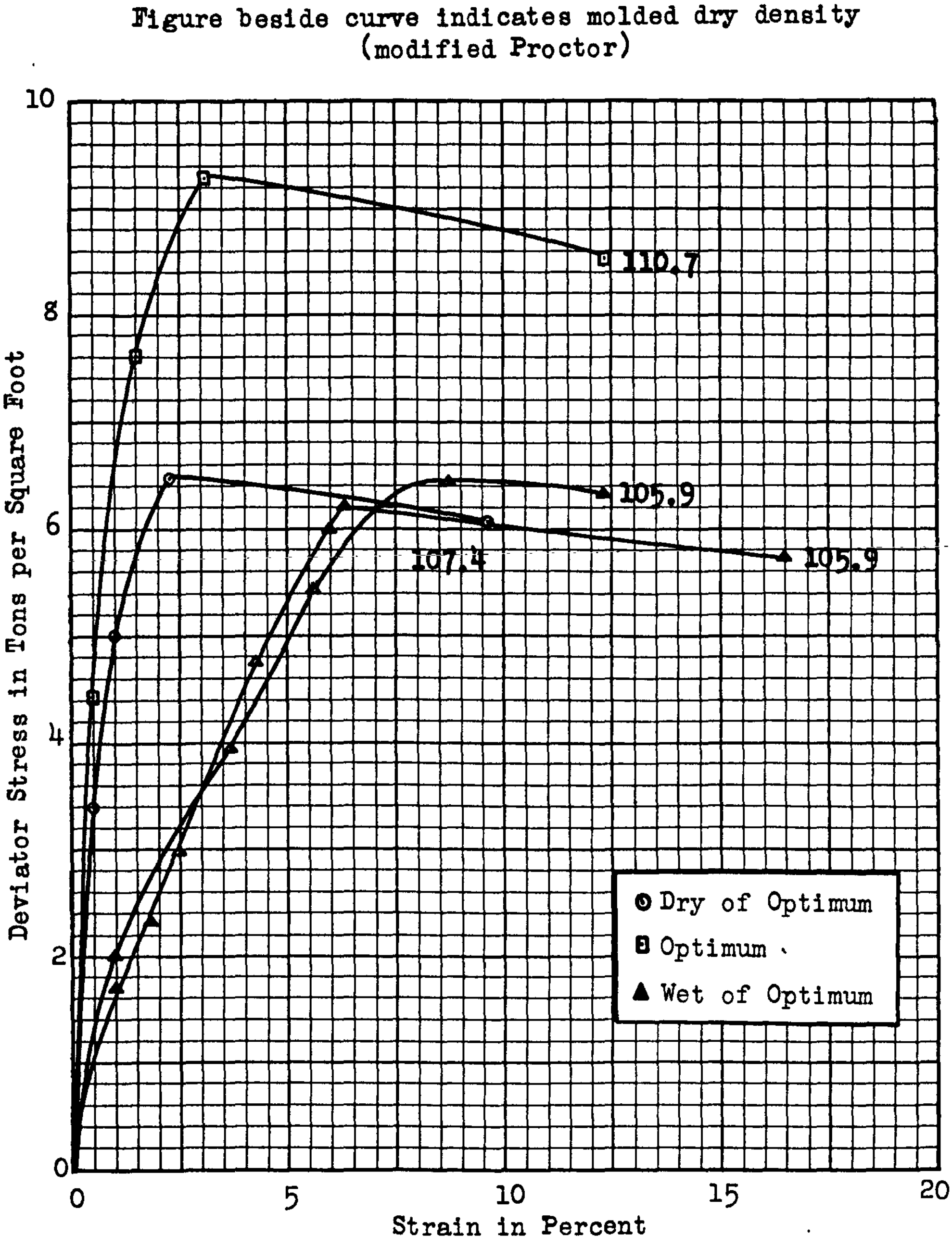
MAXIMUM DEVIATOR STRESS VS MOLDED WATER CONTENT
Triaxial Compression Tests
Sample 3 Vicksburg Loess



Mississippi River Commission
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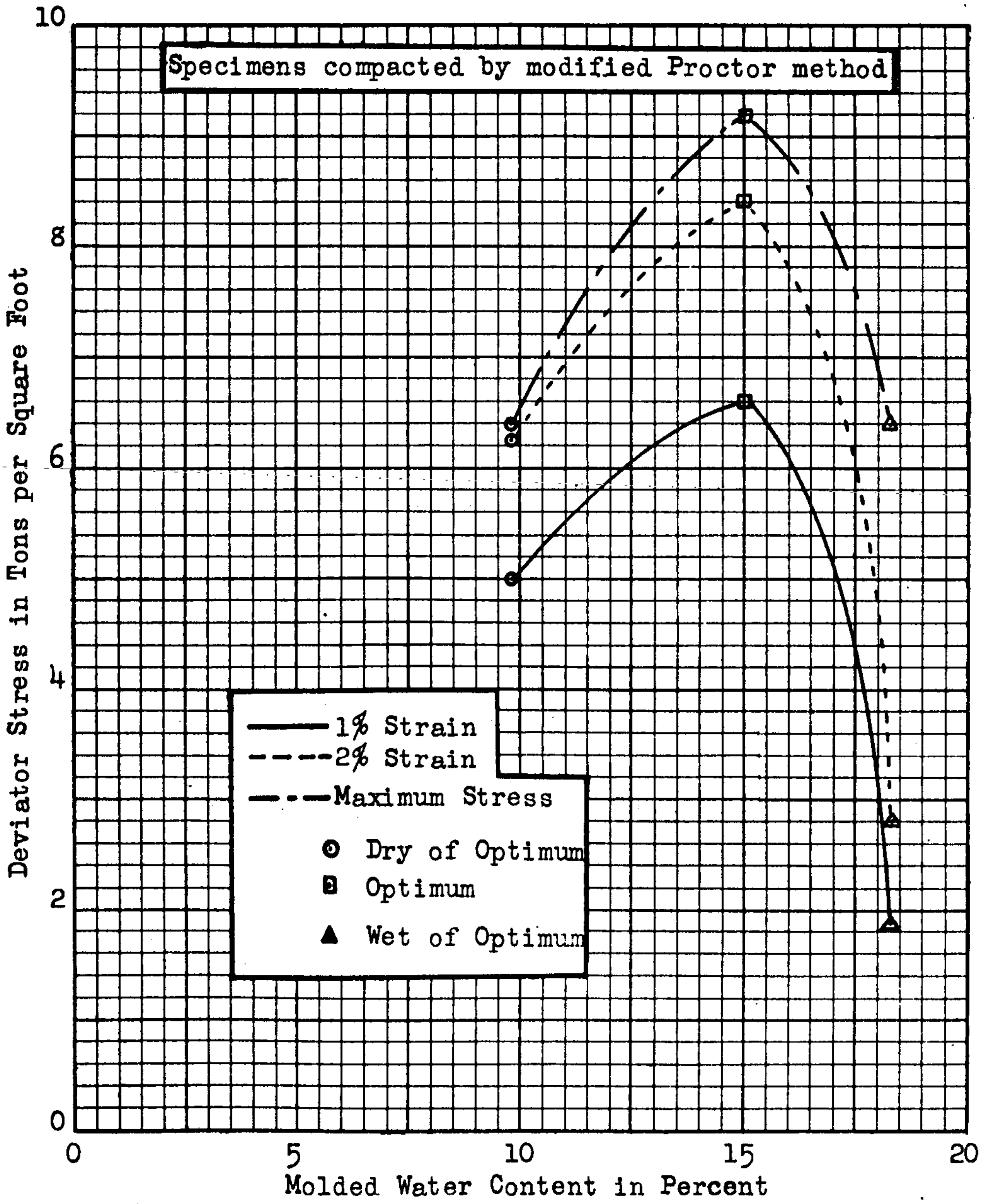
California Bearing Ratio Investigation

STRESS VS STRAIN CURVES FOR Q_c SATURATED TRIAXIAL COMPRESSION TESTS
(σ_3 = 14.7 Lb per Sq In.)

Sample 3 Vicksburg Loess



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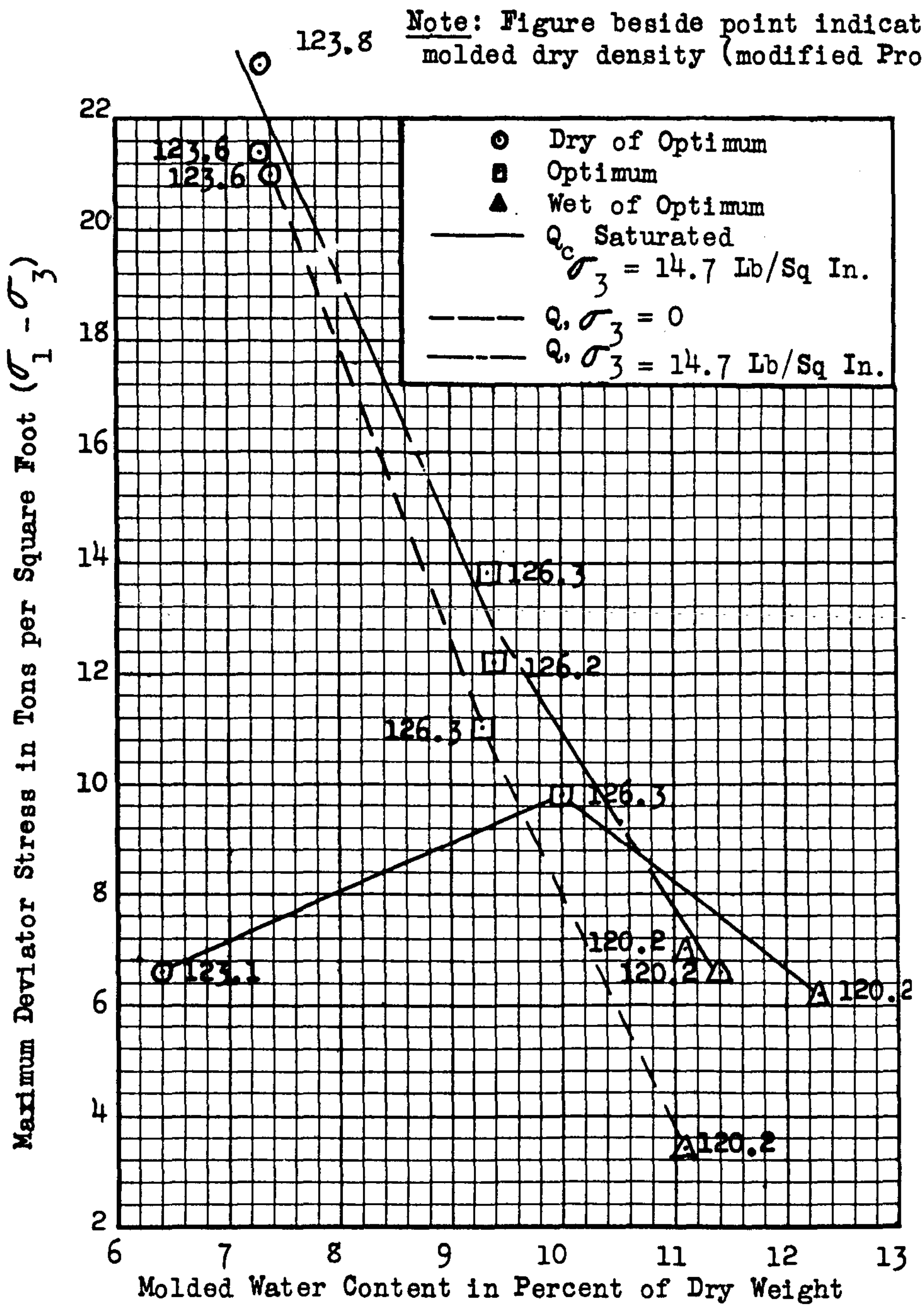
California Bearing Ratio Investigation

DEVIATOR STRESS VS MOLDED WATER CONTENT FOR
Q_c SATURATED TRIAXIAL COMPRESSION TESTS
(σ_3 = 14.7 Lb per Sq In.)

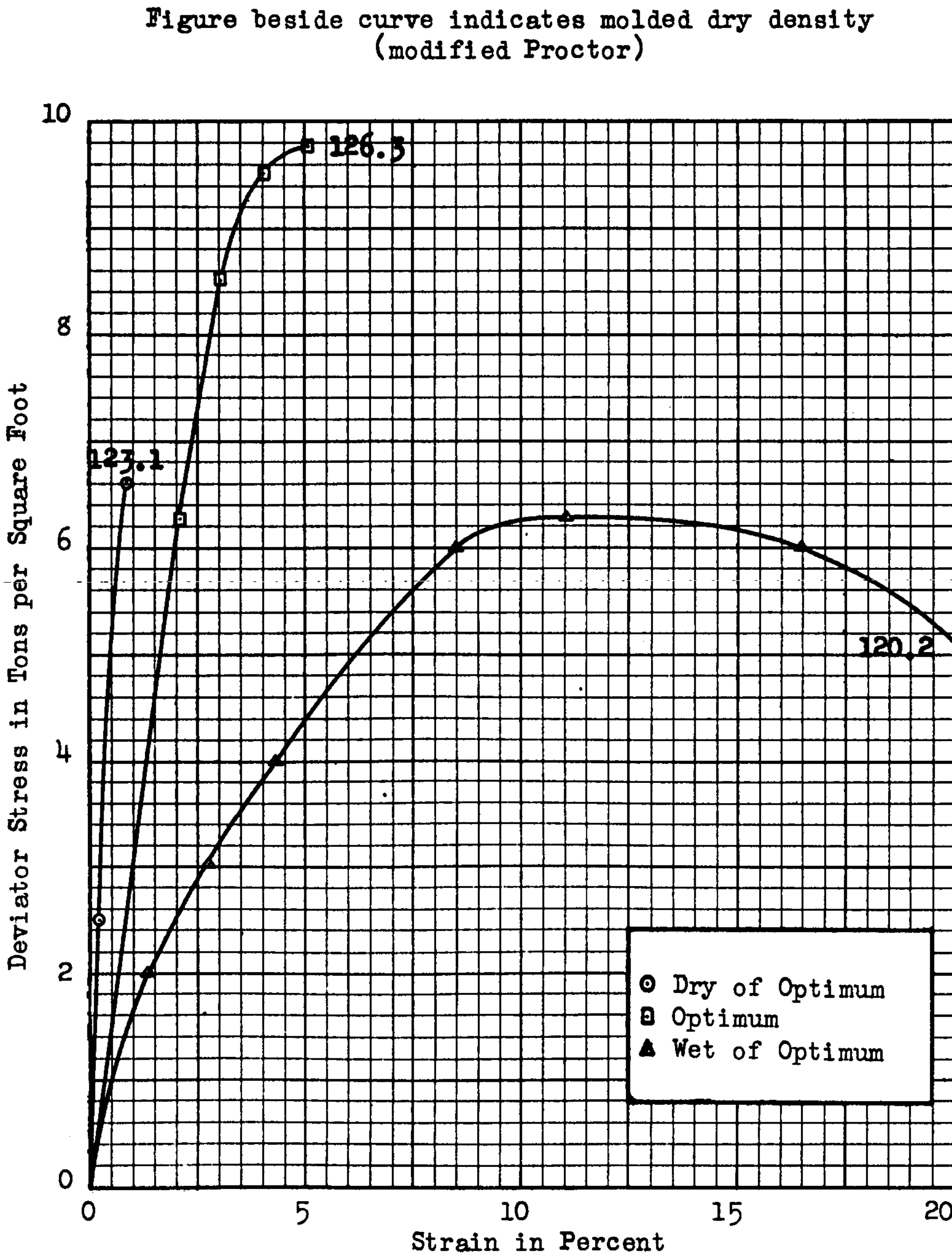
Sample 3 Vicksburg Loess



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

MAXIMUM DEVIATOR STRESS VS MOLDED WATER CONTENT
Triaxial Compression Tests

Sample 5 DeRidder Sand Clay (Loam)



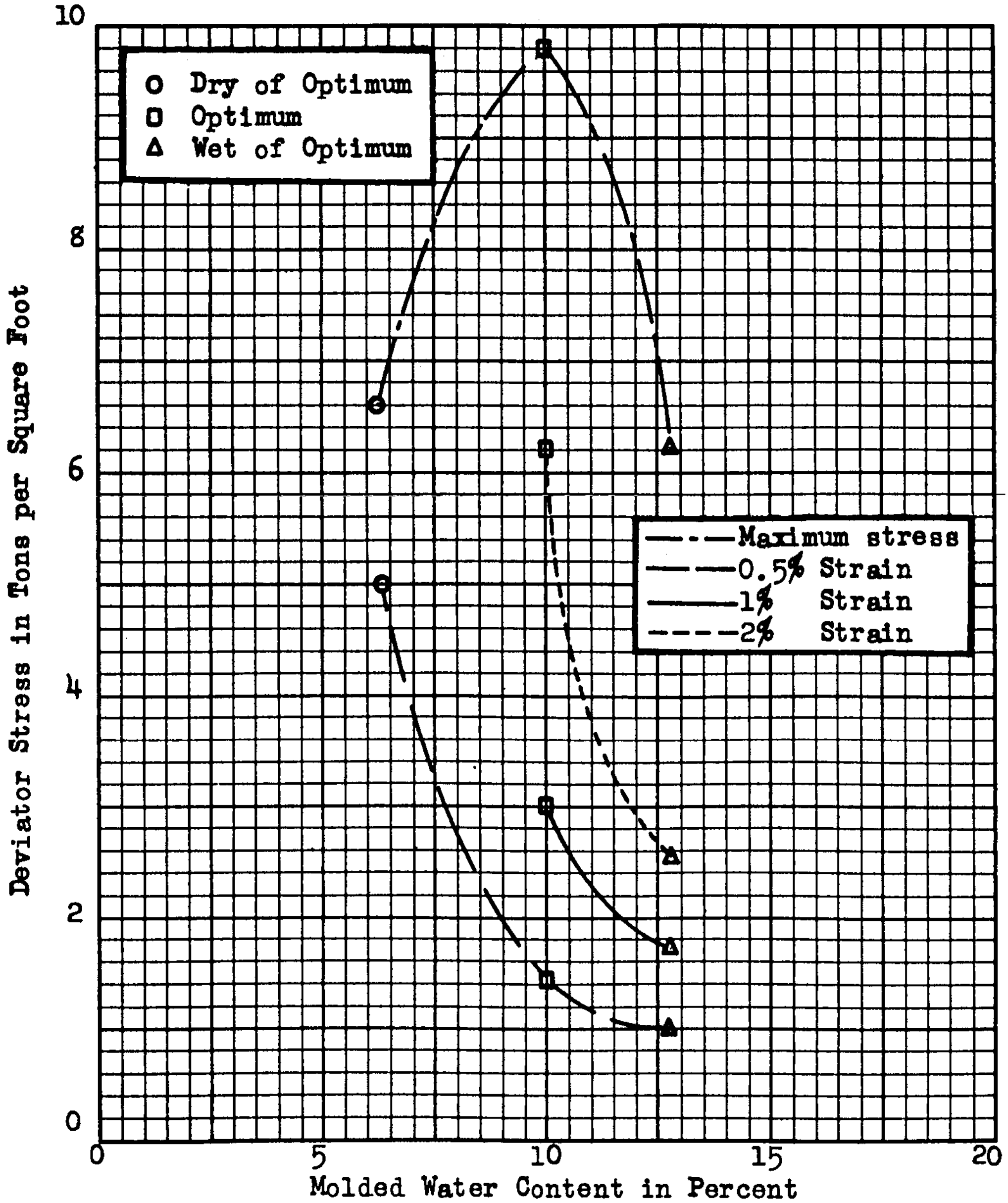
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

STRESS VS STRAIN CURVES FOR Q_c SATURATED TRIAXIAL COMPRESSION TESTS

(σ_3 = 14.7 Lb per Sq In.)

Sample 5 DeRidder Sand Clay (Loam)

Specimens compacted by modified Proctor method

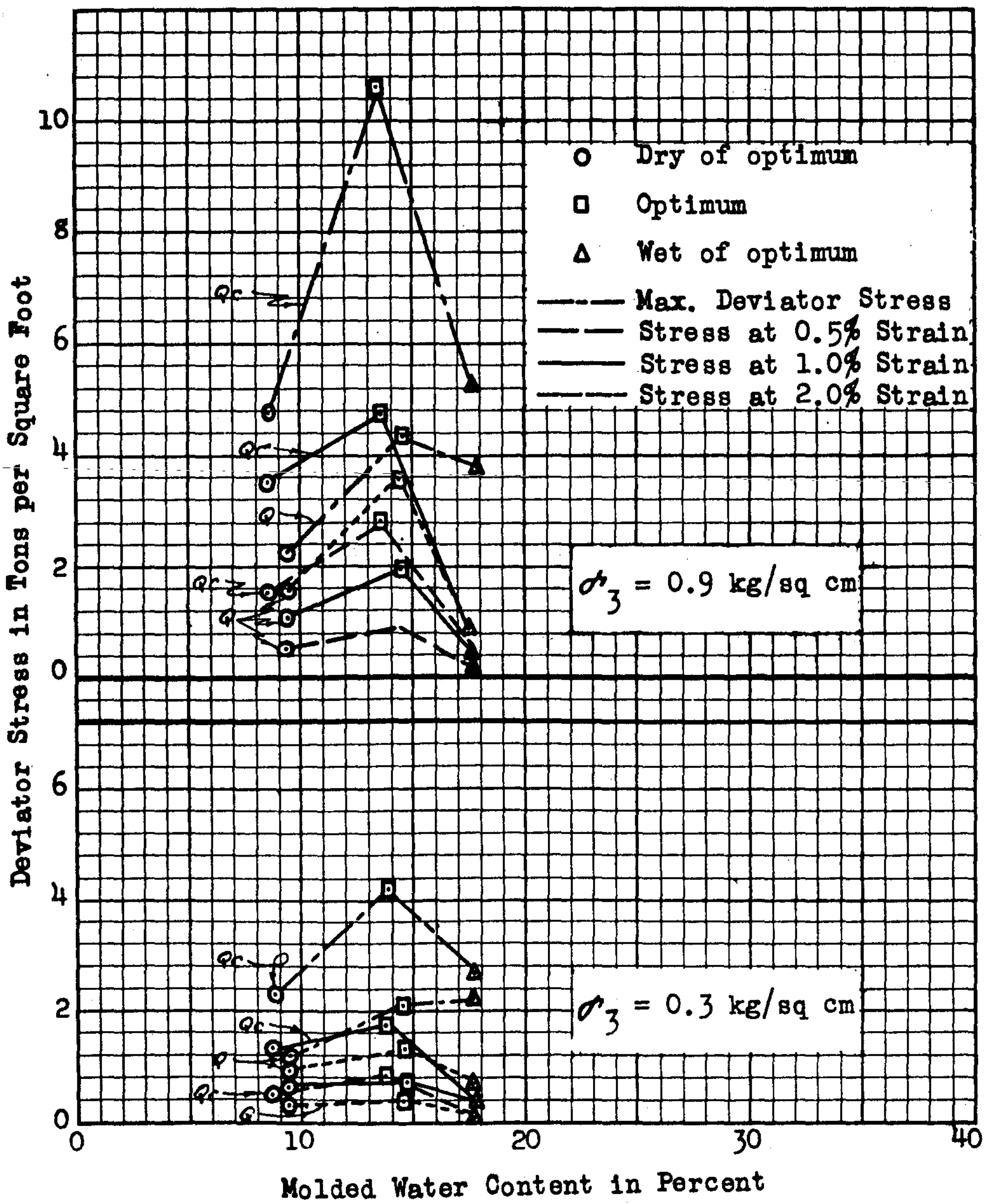


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

DEVIATOR STRESS VS MOLDED WATER CONTENT FOR

Q_c SATURATED TRIAXIAL COMPRESSION TESTS
(σ_3 = 14.7 Lb per Sq In.)

Sample 5 DeRidder Sand Clay (Loam)

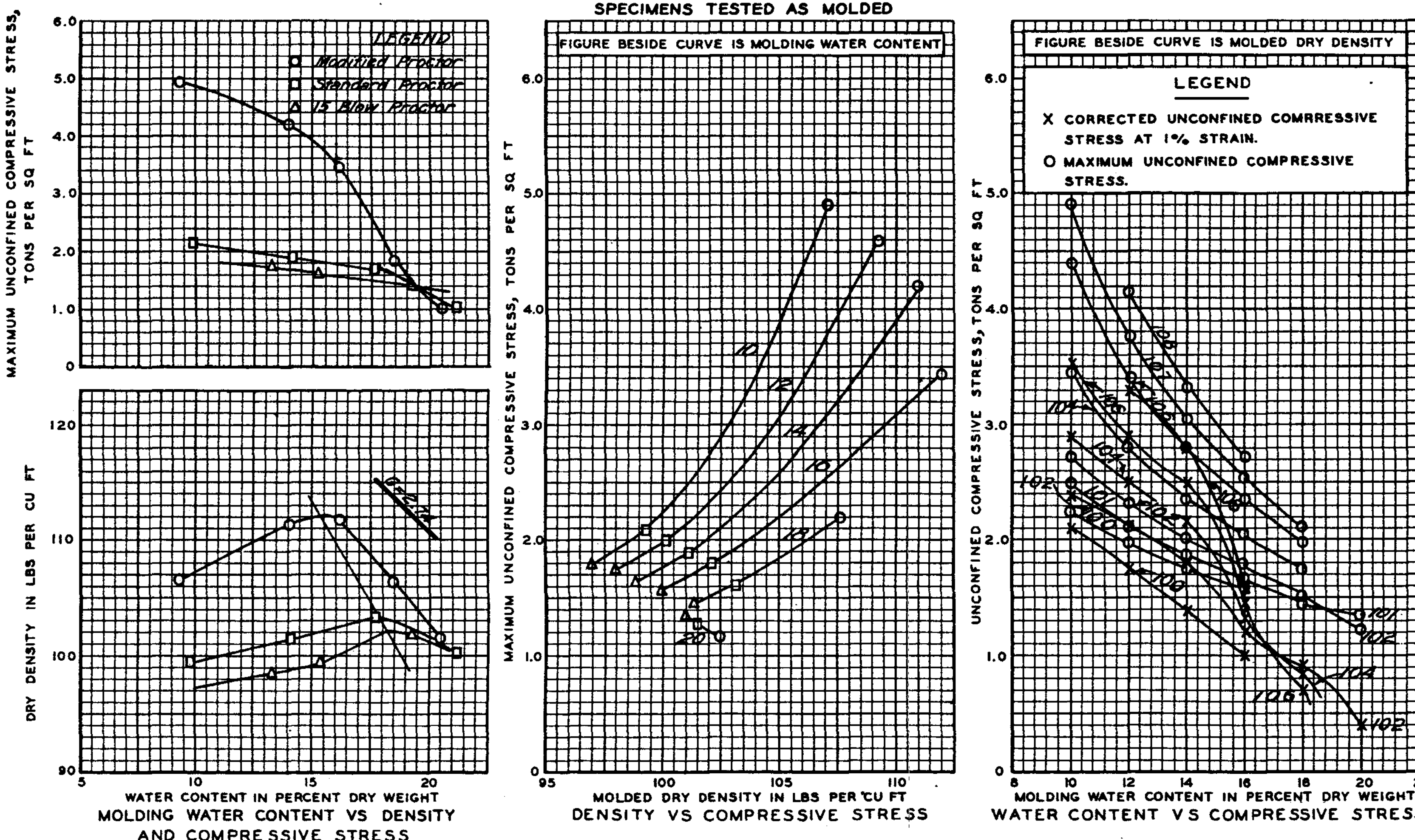


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

DEVIATOR STRESS AND STRAIN VS MOLDED WATER CONTENT FOR Q AND Q_c SATURATED TRIAXIAL COMPRESSION TESTS

Sample 3 Vicksburg Loess

SPECIMENS TESTED AS MOLDED

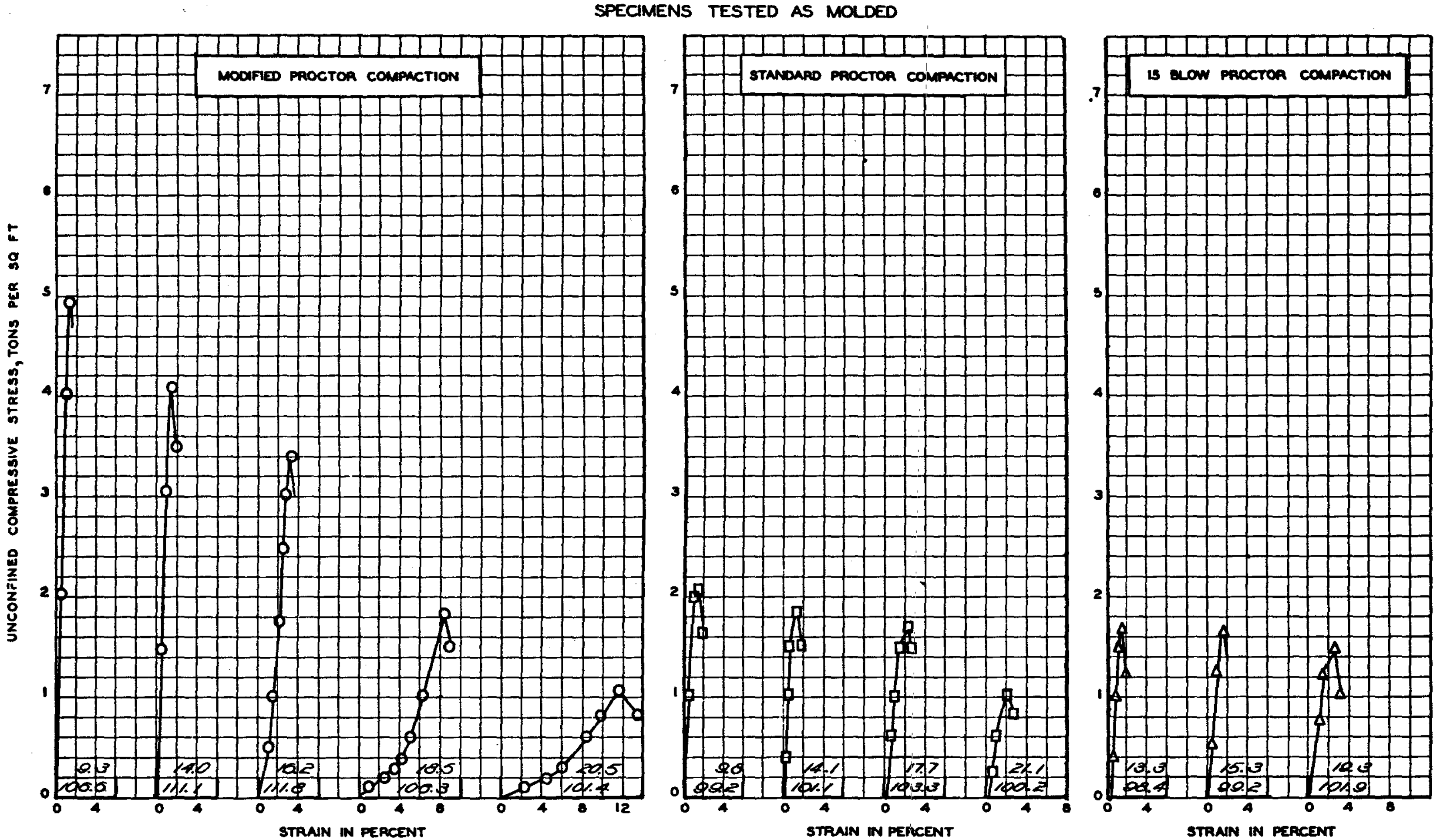


NOTES: SPECIMENS MOLDED IN 2 - INCH DIAMETER MOLD.
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF COMPRESSIVE STRESS WITH DENSITY AND WATER CONTENT

SAMPLE 3 VICKSBURG LOESS



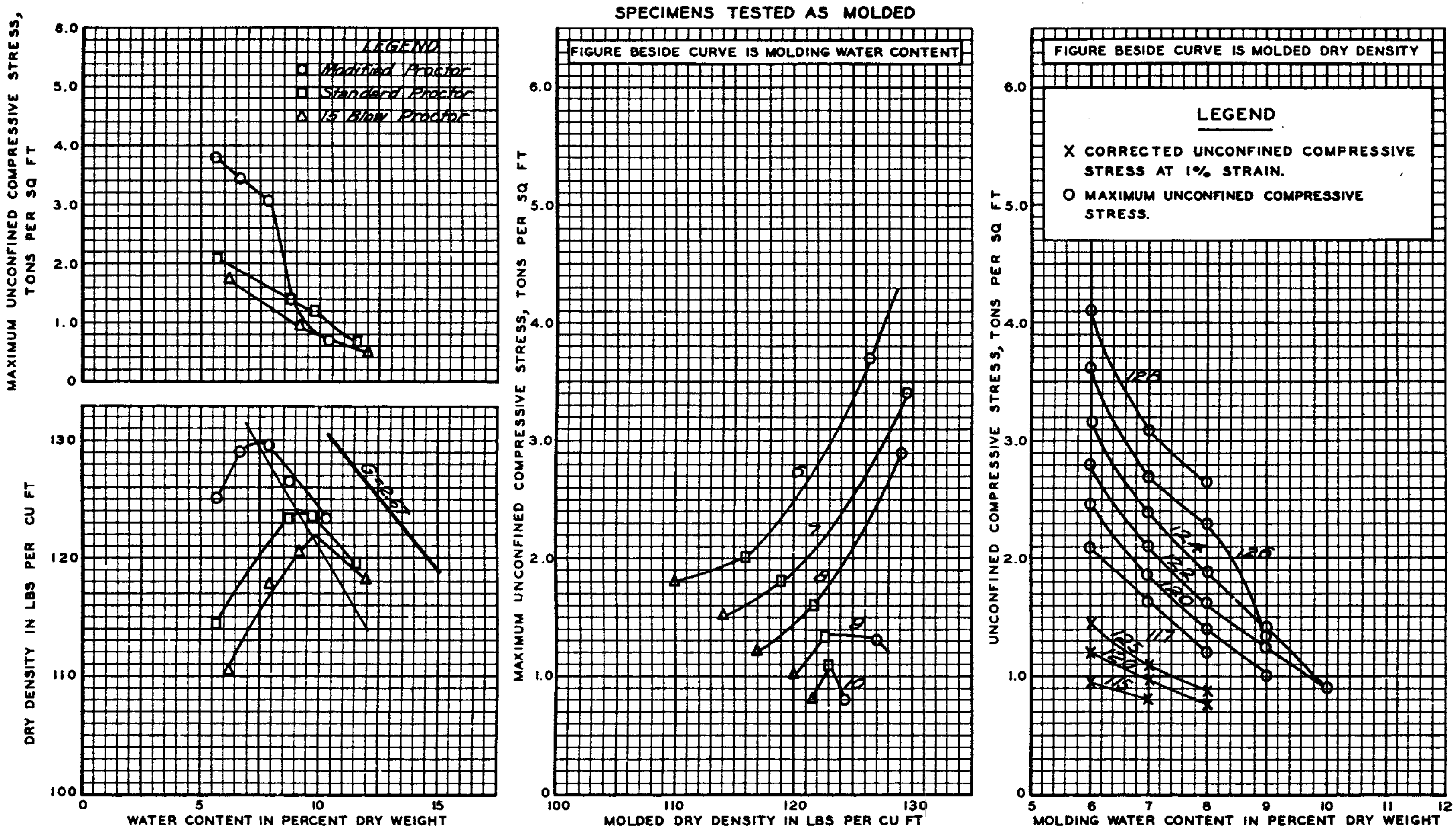
NOTES: SPECIMENS MOLDED IN 2-INCH DIAMETER MOLD.
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD.
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY.

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

UNCONFINED COMPRESSION STRESS-STRAIN CURVES

SAMPLE 3 VICKSBURG LOESS

SPECIMENS TESTED AS MOLDED



MOLDING WATER CONTENT VS DENSITY AND COMPRESSIVE STRESS

DENSITY VS COMPRESSIVE STRESS

WATER CONTENT VS COMPRESSIVE STRESS

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF COMPRESSIVE STRESS WITH DENSITY AND WATER CONTENT

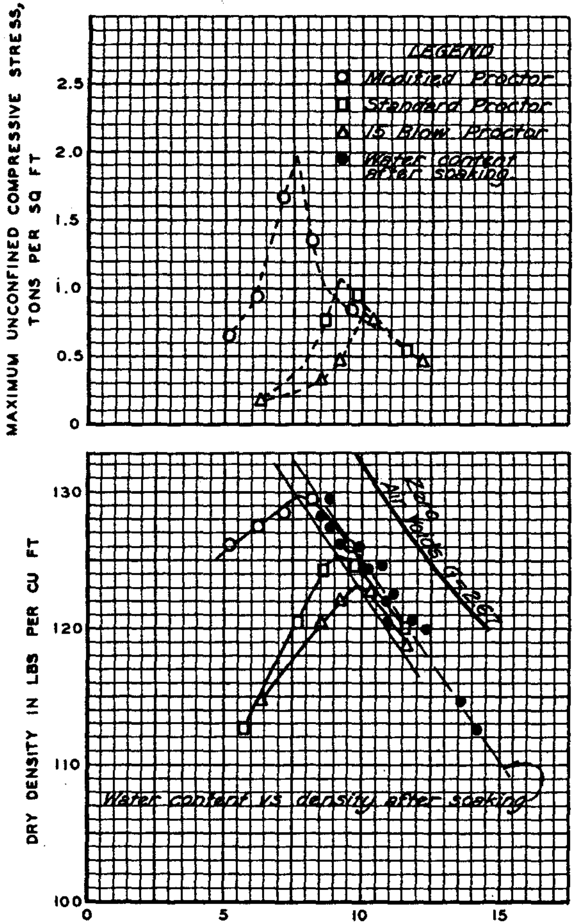
SAMPLE 23 VICKSBURG CLAY SAND

NOTES: TEST SPECIMENS CUT FROM SPECIMENS MOLDED IN CBR MOLD.
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD.

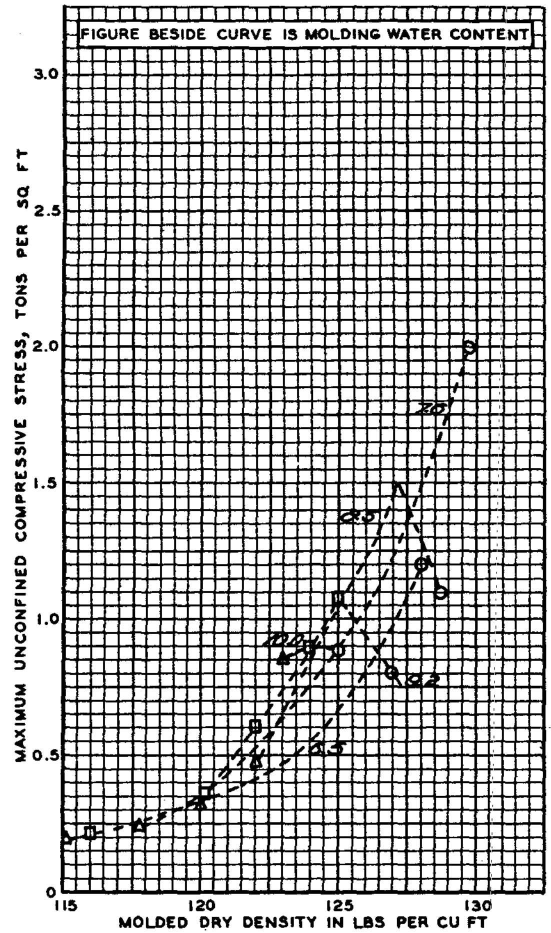
WAR DEPARTMENT

CORPS OF ENGINEERS. U.S. ARMY

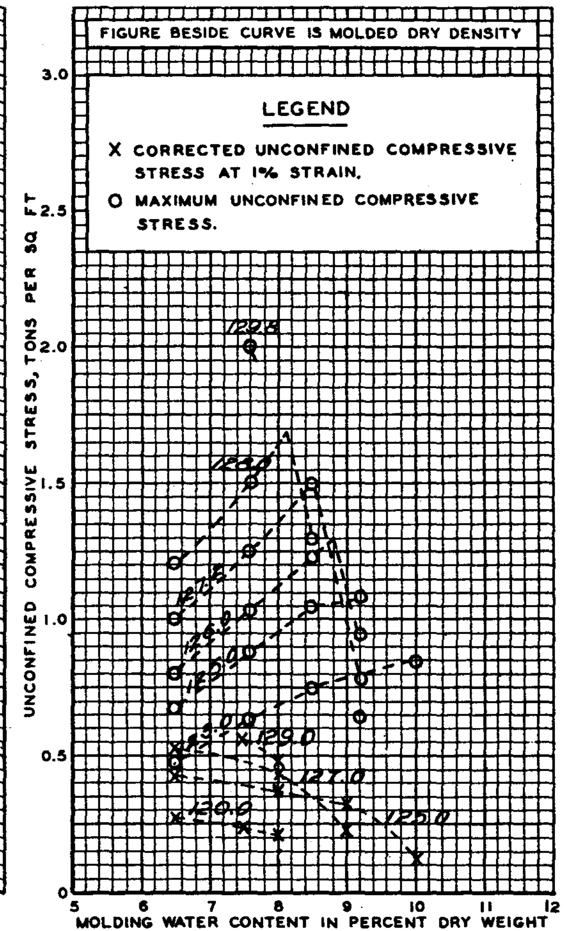
SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND COMPRESSIVE STRESS



DENSITY VS COMPRESSIVE STRESS



WATER CONTENT VS COMPRESSIVE STRESS

NOTES: TEST SPECIMENS CUT FROM SPECIMENS MOLDED AND SOAKED IN CBR MOLD.
NO SWELL DURING SOAKING.
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD.

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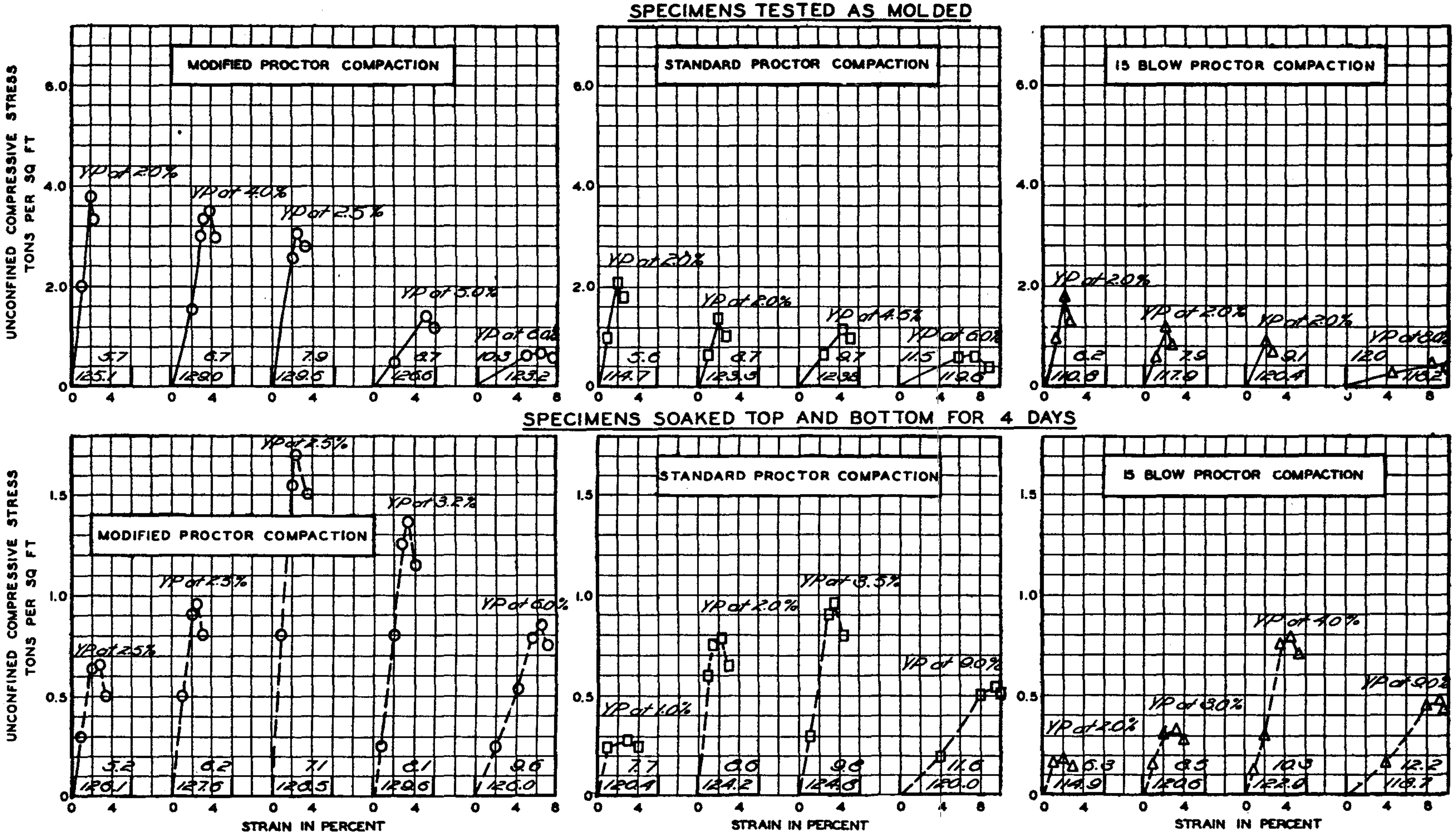
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF COMPRESSIVE STRESS WITH DENSITY AND WATER CONTENT

SAMPLE 23 VICKSBURG CLAY SAND



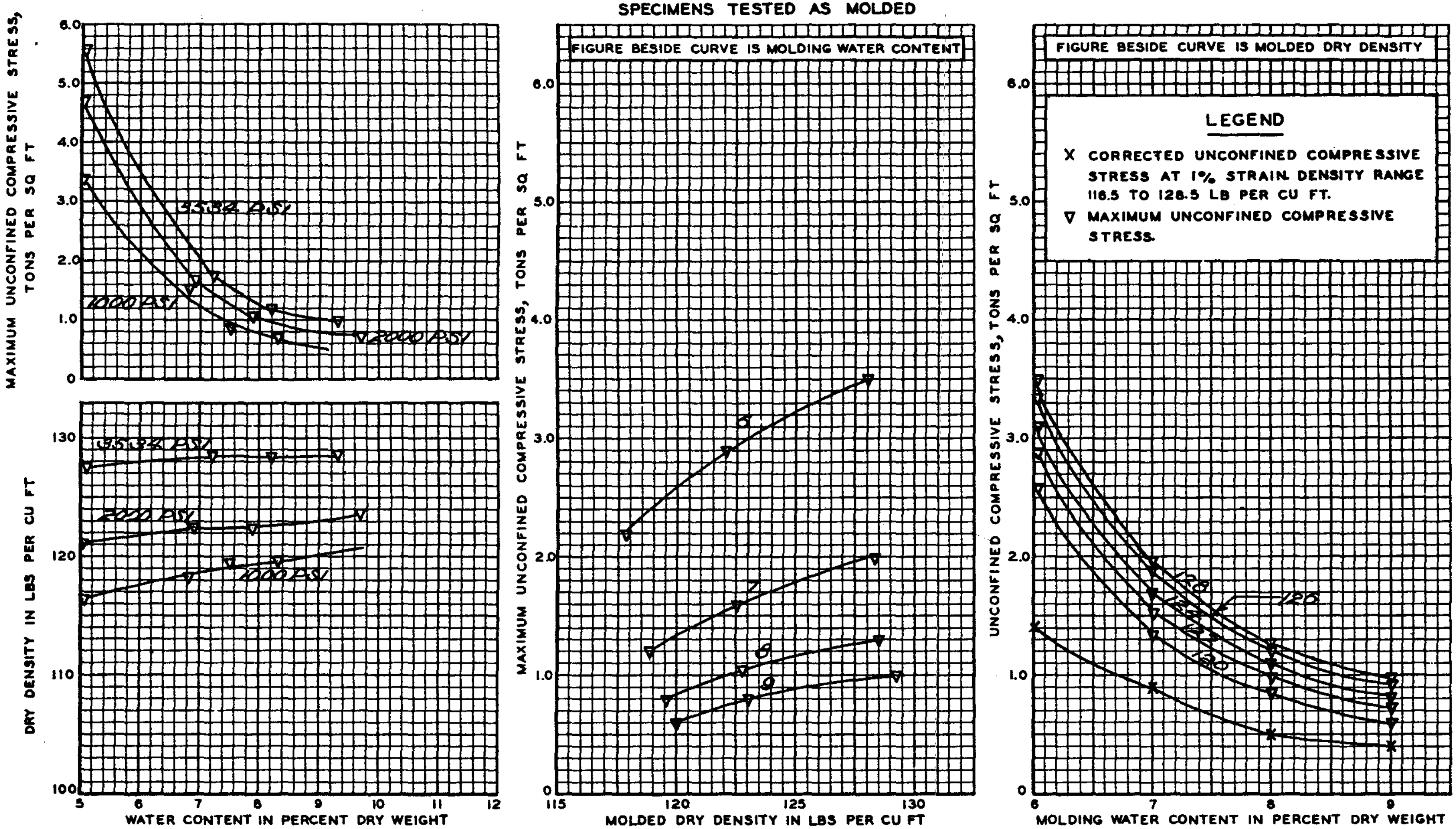
NOTES: TEST SPECIMENS CUT FROM SPECIMENS MOLDED OR MOLDED AND SOAKED IN CBR MOLD
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY
YP = YIELD POINT

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

UNCONFINED COMPRESSION STRESS-STRAIN CURVES

SAMPLE 23 VICKSBURG CLAY SAND

SPECIMENS TESTED AS MOLDED



MOLDING WATER CONTENT VS DENSITY AND COMPRESSIVE STRESS

DENSITY VS COMPRESSIVE STRESS

WATER CONTENT VS COMPRESSIVE STRESS

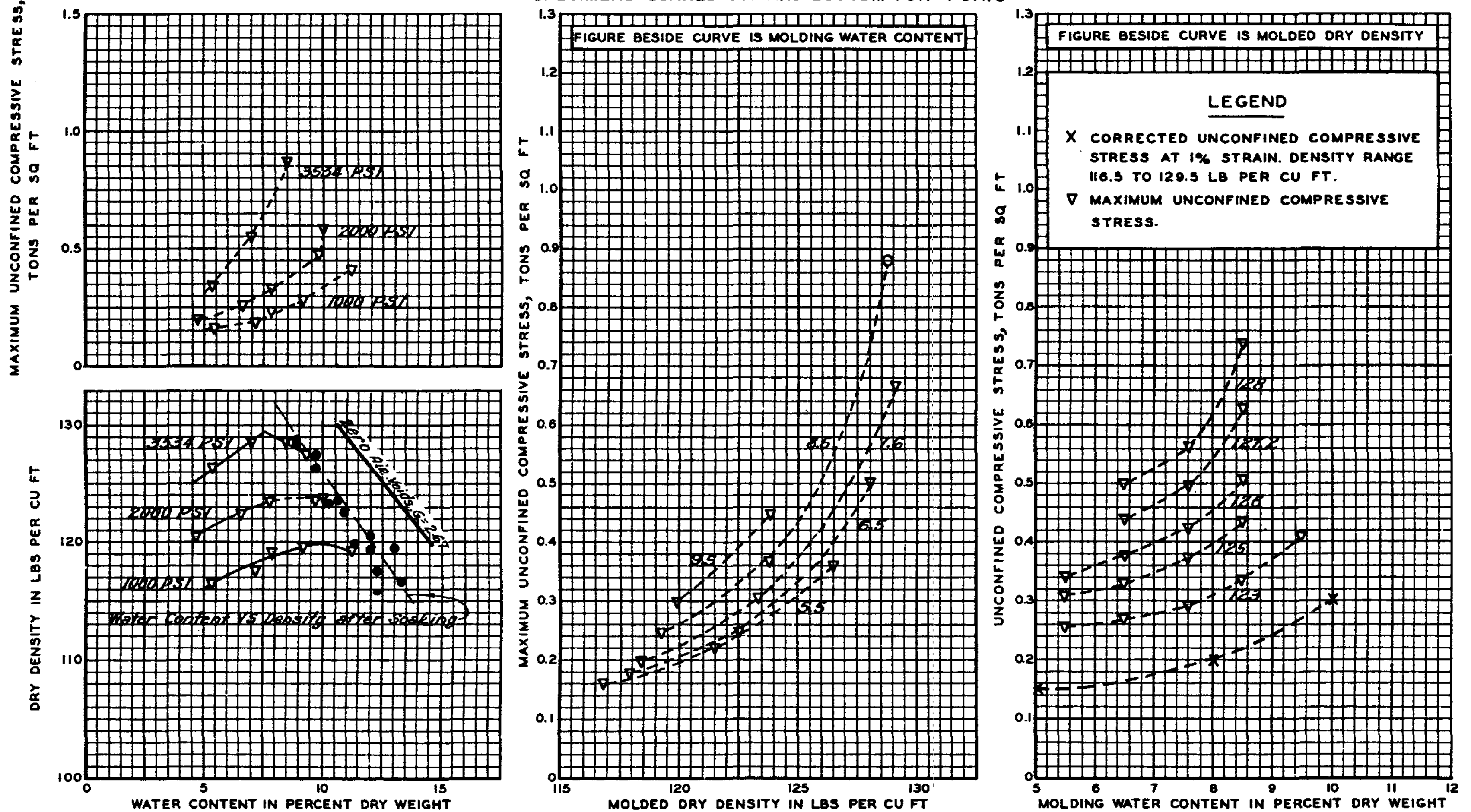
NOTES: STATIC COMPACTION.
TEST SPECIMENS CUT FROM SPECIMENS MOLDED IN CBR MOLD.
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD.

MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

VARIATION OF COMPRESSIVE STRESS WITH DENSITY AND WATER CONTENT

SAMPLE 23 VICKSBURG CLAY SAND

SPECIMENS SOAKED TOP AND BOTTOM FOR 4 DAYS



MOLDING WATER CONTENT VS DENSITY AND COMPRESSIVE STRESS

DENSITY VS COMPRESSIVE STRESS

WATER CONTENT VS COMPRESSIVE STRESS

NOTES: STATIC COMPACTION.
TEST SPECIMENS CUT FROM SPECIMENS MOLDED AND SOAKED IN CBR MOLD.
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD.

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION

CALIFORNIA BEARING RATIO INVESTIGATION

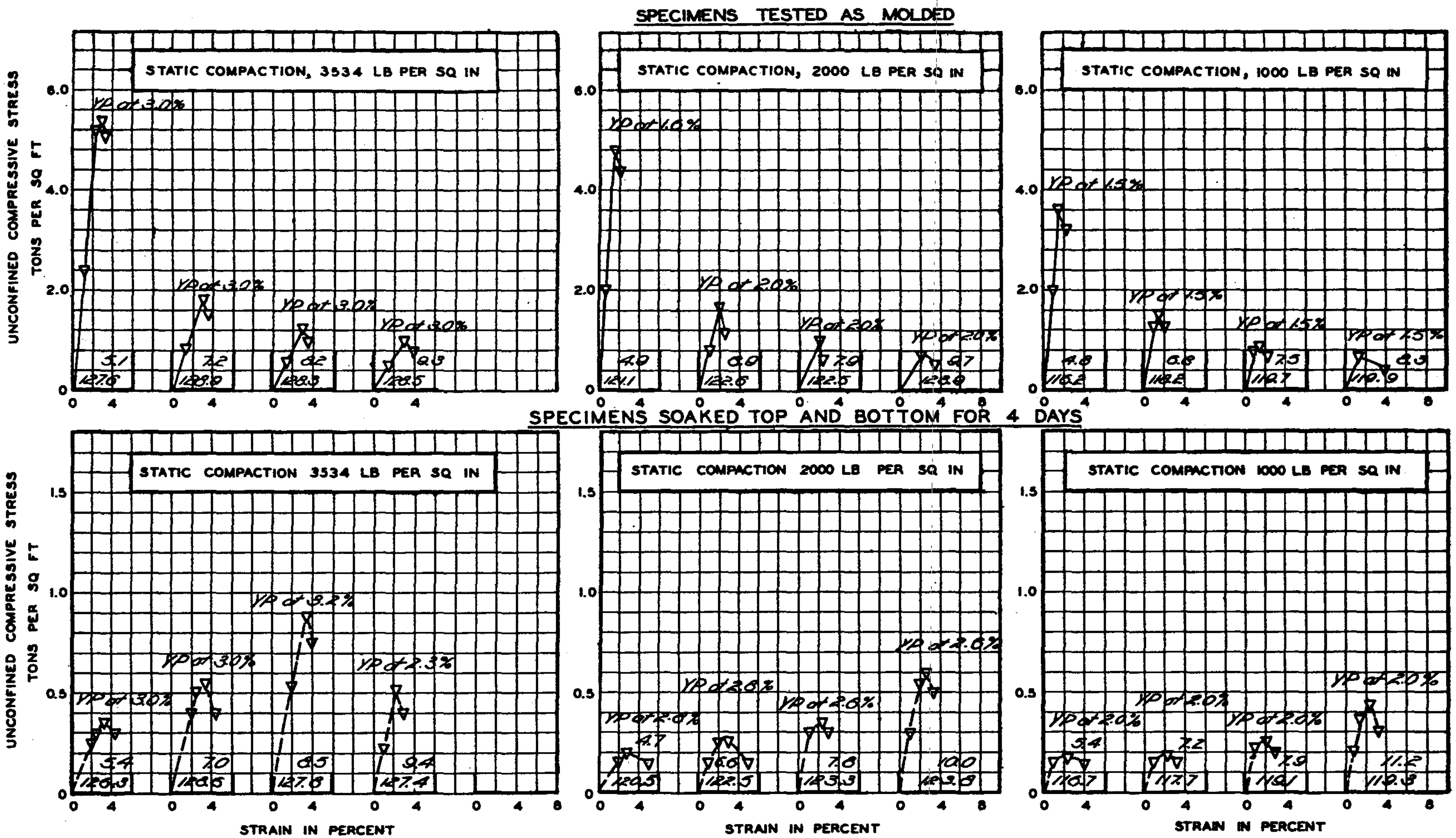
VARIATION OF COMPRESSIVE STRESS WITH DENSITY AND WATER CONTENT

SAMPLE 23—VICKSBURG CLAY SAND

PLATE 129

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY



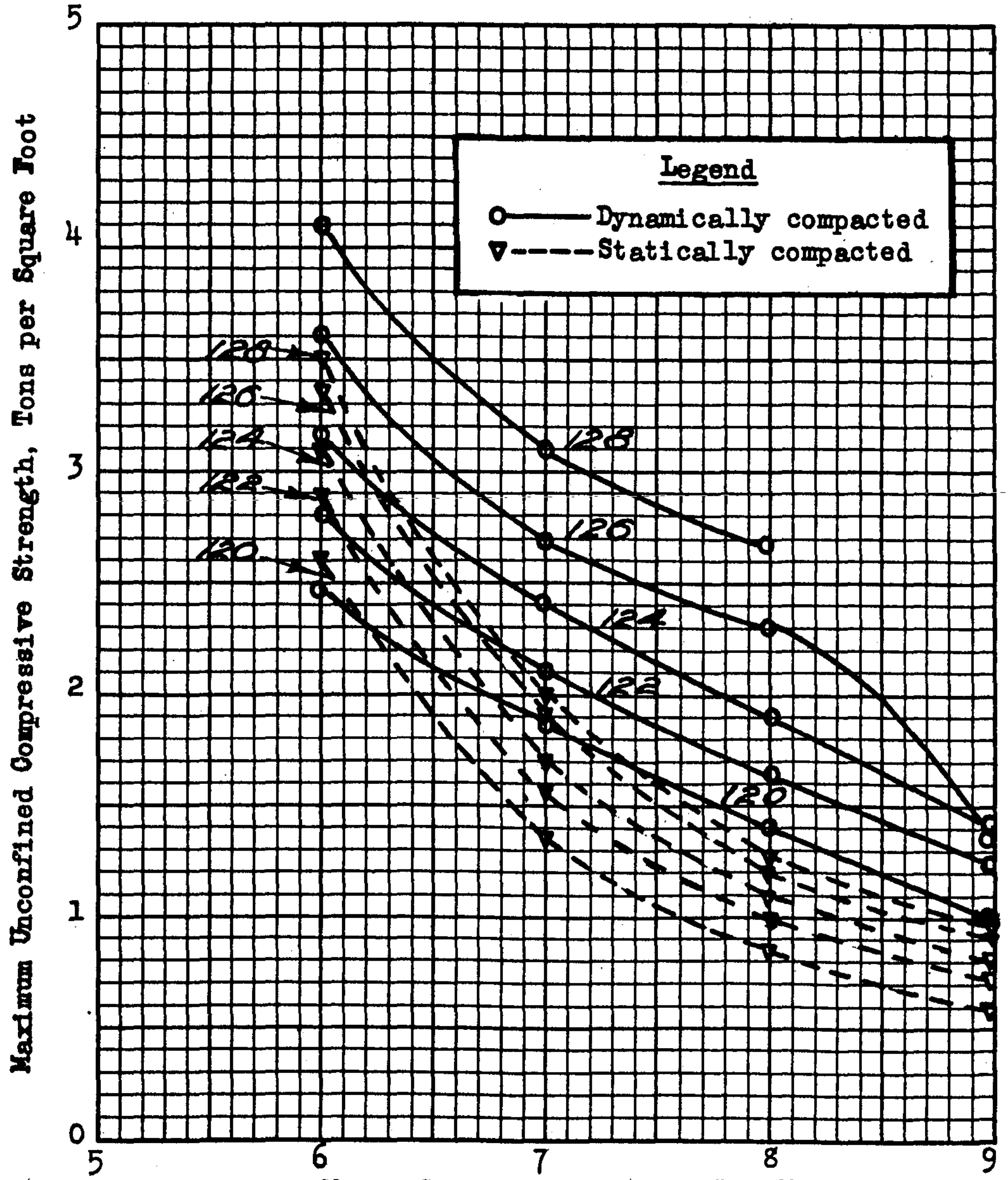
NOTES: TEST SPECIMENS CUT FROM SPECIMENS MOLDED OR MOLDED AND SOAKED IN CBR MOLD
DENSITY DETERMINATION MADE BEFORE REMOVING SPECIMEN FROM MOLD
FIGURES BESIDE CURVES INDICATE MOLDING WATER CONTENT AND DRY DENSITY RESPECTIVELY
YP=YIELD POINT

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

UNCONFINED COMPRESSION STRESS-STRAIN CURVES

SAMPLE 23 VICKSBURG CLAY SAND

Notes:
Specimens tested as molded.
Figure beside curve is molded dry density.



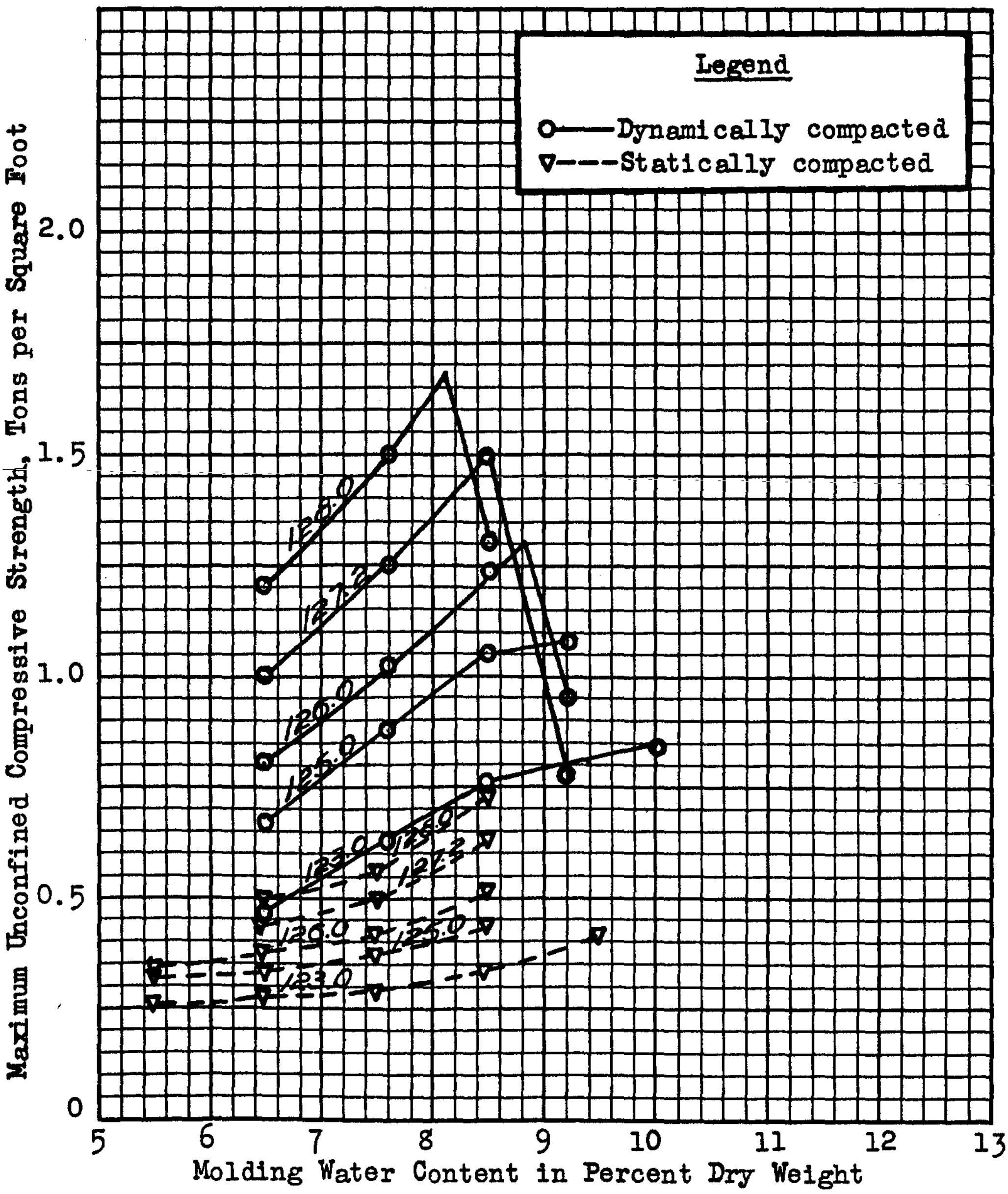
Molding Water Conuent in Percent Dry Weight

Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPARISON OF UNCONFINED COMPRESSIVE STRENGTHS OF DYNAMICALLY COMPACTED WITH STATICALLY COMPACTED SPECIMENS

Sample 23 Vicksburg Clay Sand

Notes:
Specimens soaked top and bottom for 4 days.
Figure beside curve is molded dry density.

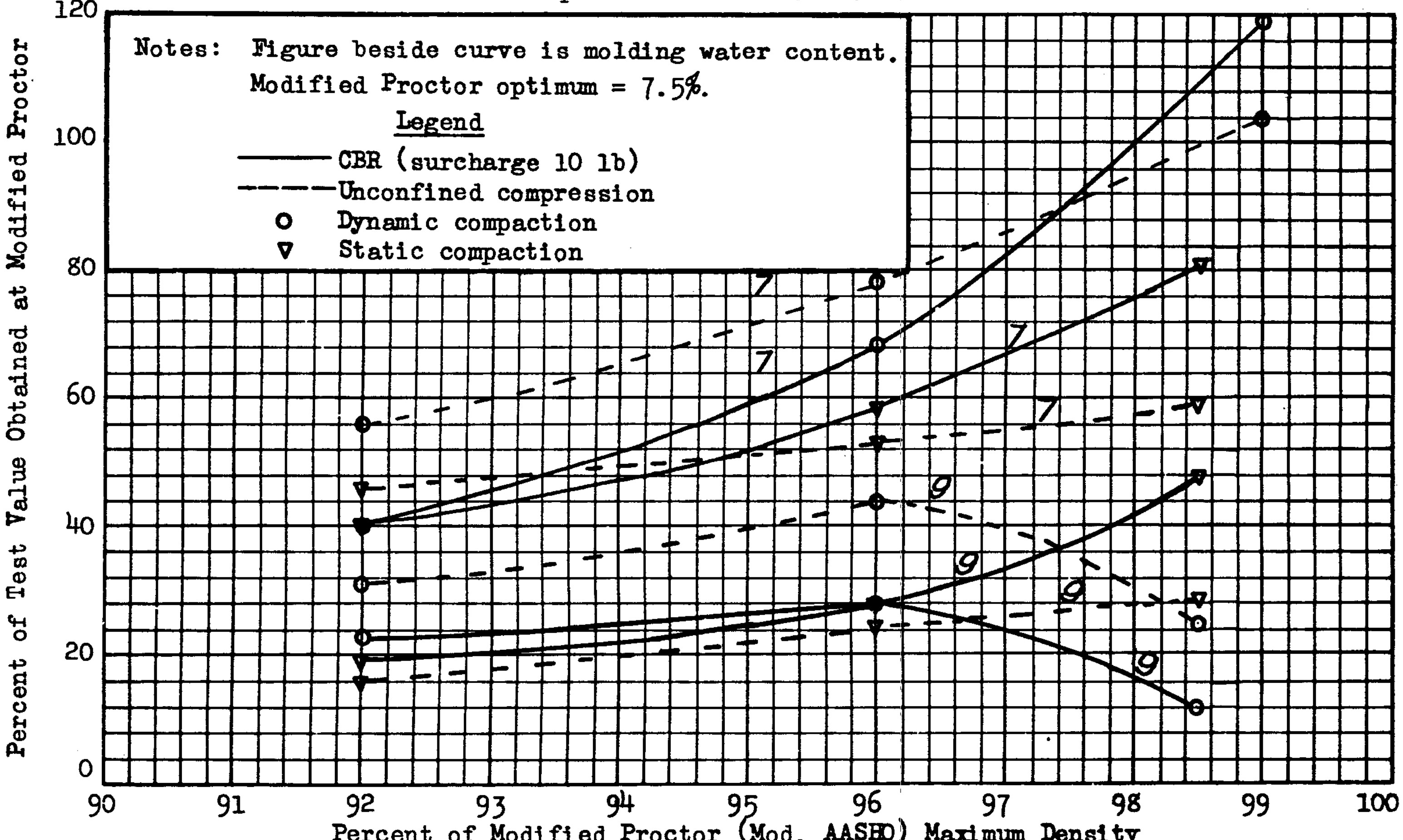


Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

COMPARISON OF UNCONFINED COMPRESSIVE STRENGTHS OF DYNAMICALLY COMPACTED WITH STATICALLY COMPACTED SPECIMENS

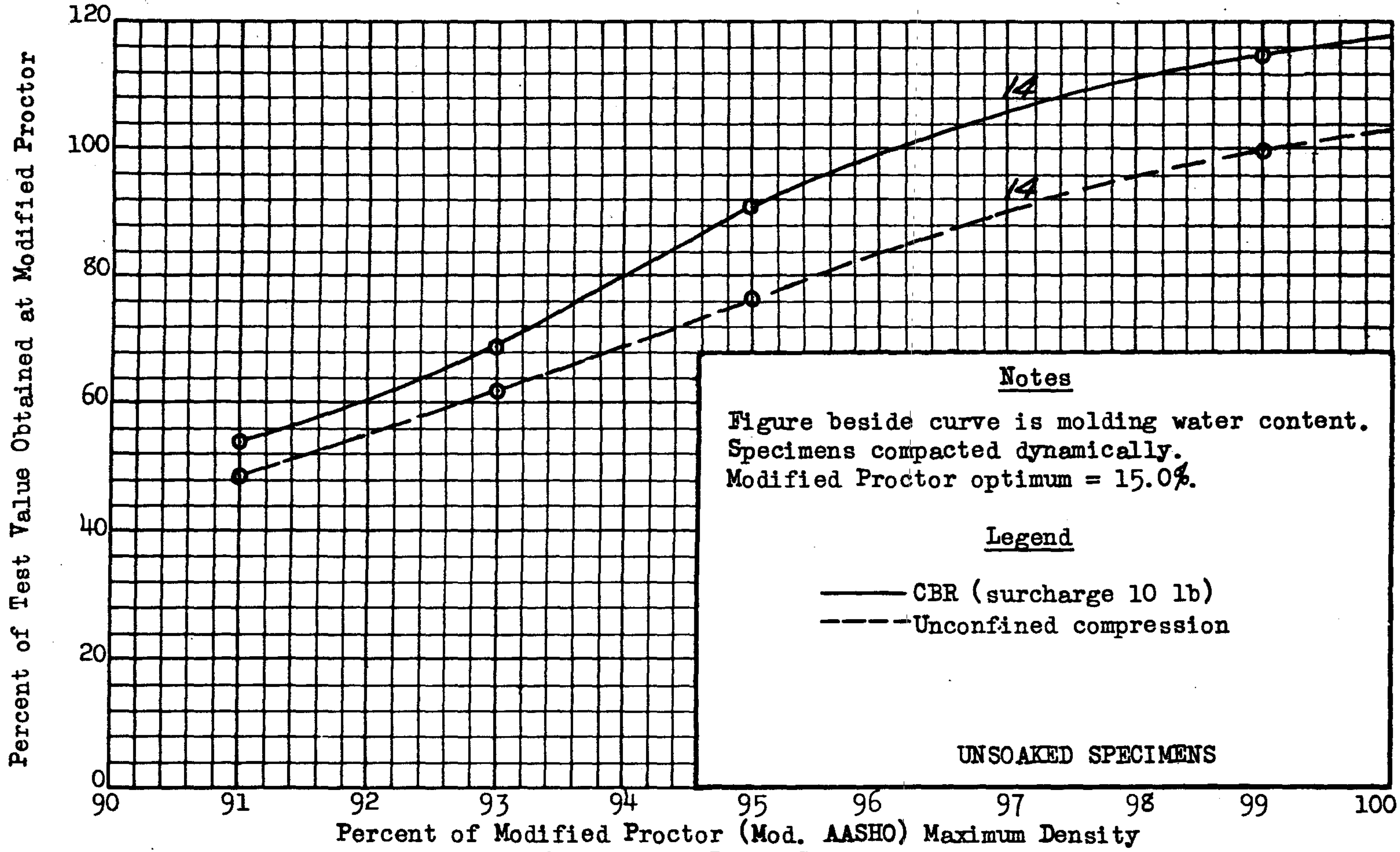
Sample 23 Vicksburg Clay Sand

Specimens tested as molded



Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation
COMPARISON OF THE SENSITIVITY (TO DENSITY AND STRUCTURE) OF
THE CBR AND UNCONFINED COMPRESSION TEST

Sample 23 Vicksburg Clay Sand



Mississippi River Commission
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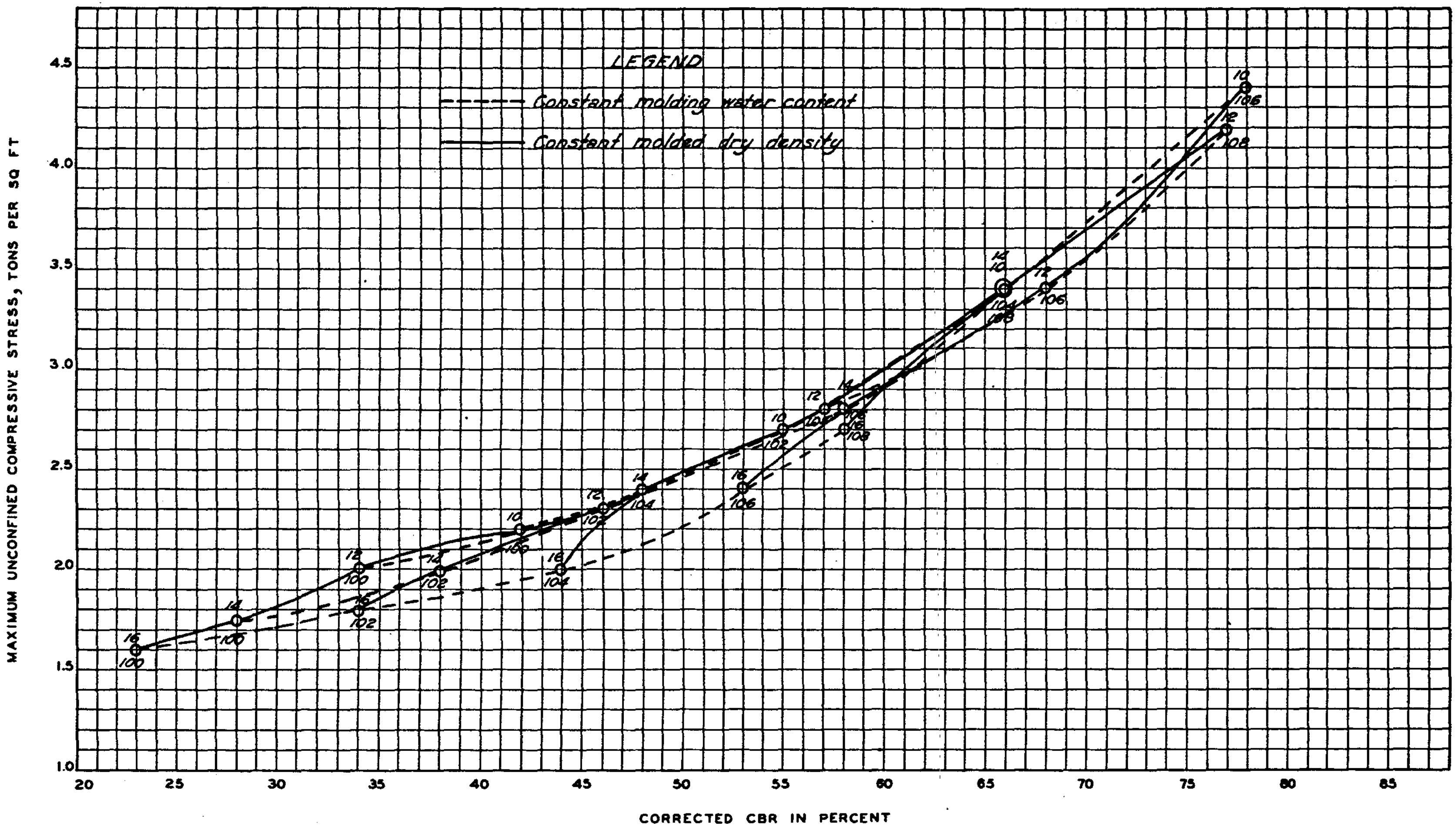
California Bearing Ratio Investigation

COMPARISON OF THE SENSITIVITY (TO DENSITY AND STRUCTURE) OF THE CBR AND UNCONFINED COMPRESSION TEST

Sample 3

Vicksburg Loess



NOTES: SPECIMENS TESTED AS MOLDED
FIGURE ABOVE POINT INDICATES MOLDING WATER CONTENT
FIGURE BELOW POINT INDICATES MOLDED DRY DENSITY
ALL SPECIMENS COMPACTED DYNAMICALLY

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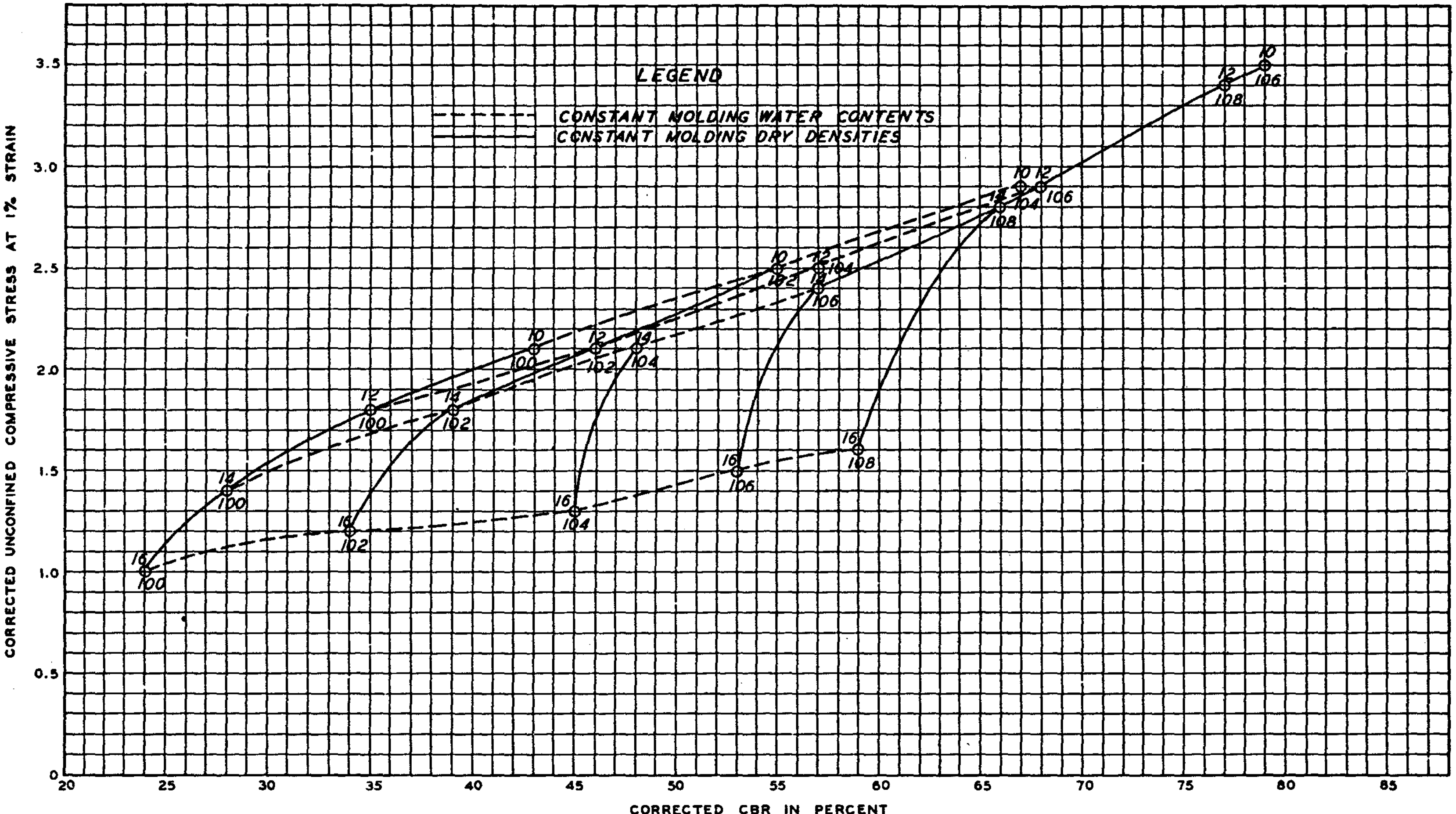
CALIFORNIA BEARING RATIO INVESTIGATION

CBR VS UNCONFINED COMPRESSIVE STRESS

SAMPLE 3 VICKSBURG LOESS

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

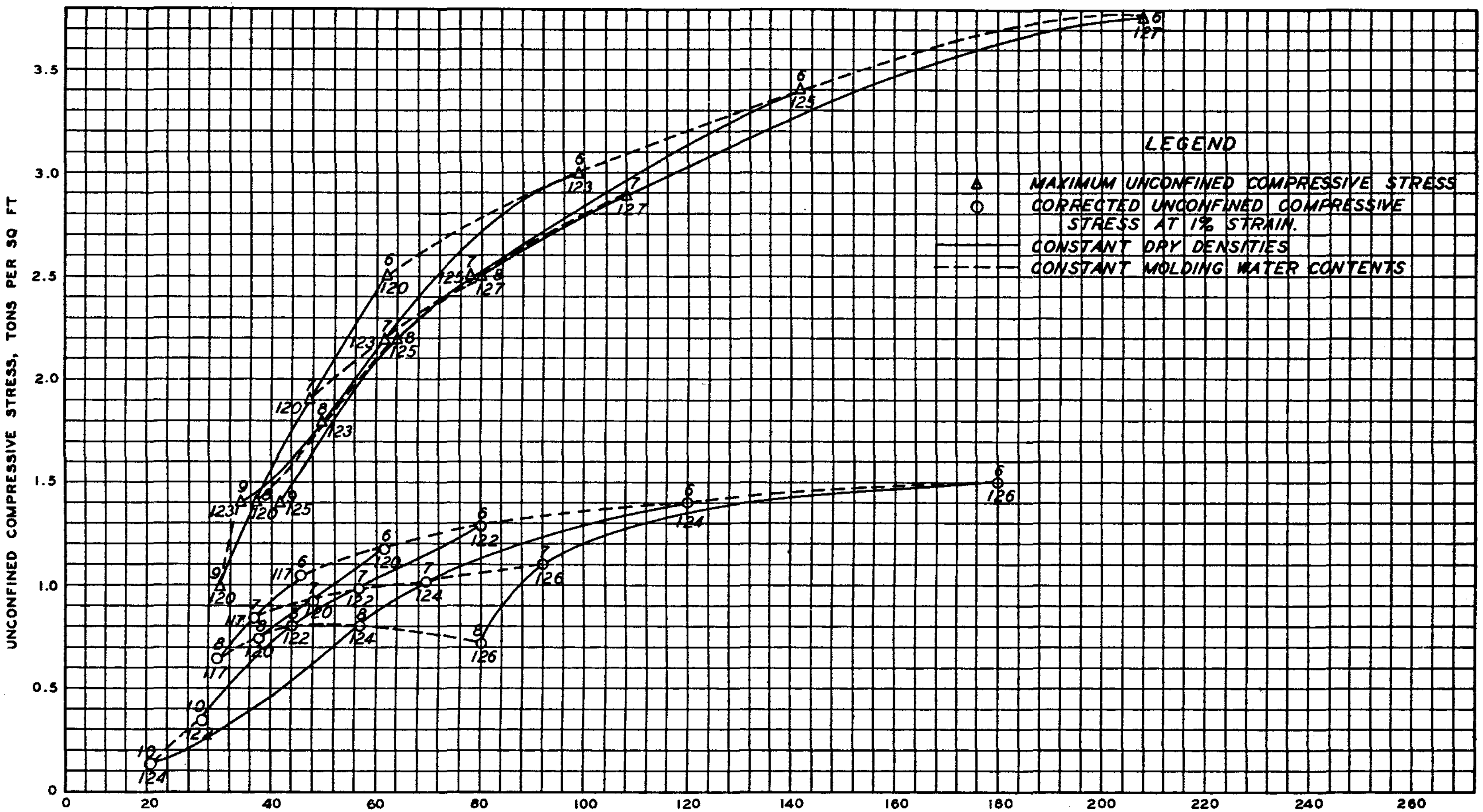


NOTES: SPECIMENS TESTED AS MOLDED.
ALL SPECIMENS COMPACTED DYNAMICALLY.
FIGURE ABOVE POINT INDICATES MOLDING WATER CONTENT.
FIGURE BELOW POINT INDICATES MOLDED DRY DENSITY.
DATA TAKEN FROM FAMILIES OF CURVES FOR CBR AND UNCONFINED COMPRESSIVE STRESS FOR VARIATION IN WATER CONTENT AND DENSITY.

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

CBR VS UNCONFINED COMPRESSIVE STRESS

SAMPLE 3 VICKSBURG LOESS

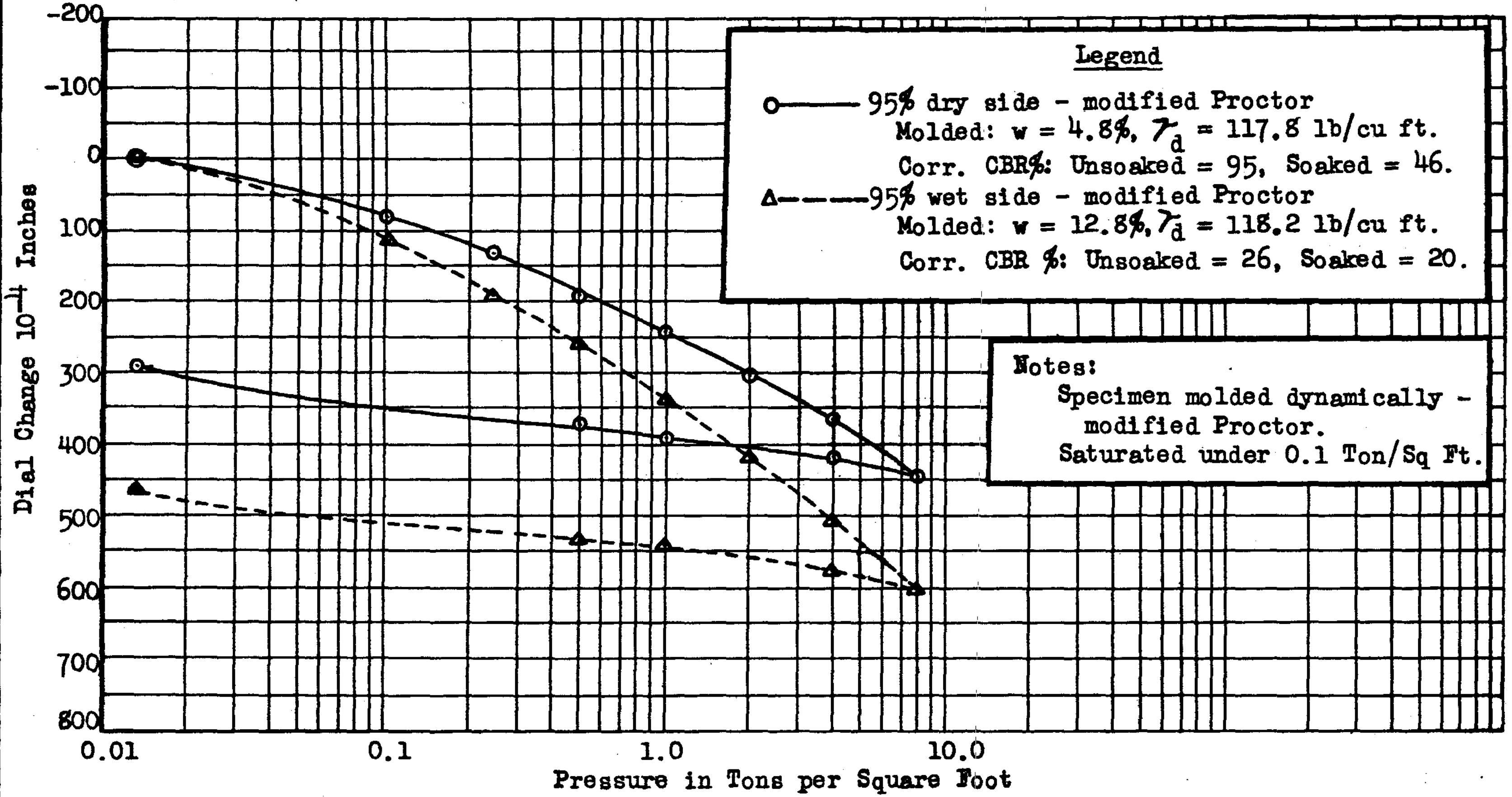


NOTES: SPECIMENS TESTED AS MOLDED.
ALL SPECIMENS COMPACTED DYNAMICALLY.
FIGURE ABOVE POINT INDICATES MOLDING WATER CONTENT.
FIGURE BELOW POINT INDICATES MOLDED DRY DENSITY.
DATA TAKEN FROM FAMILIES OF CURVES FOR CBR AND UNCONFINED COMPRESSIVE STRESS FOR VARIATION IN WATER CONTENT AND DENSITY.

MISSISSIPPI RIVER COMMISSION
U. S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION

CBR VS UNCONFINED COMPRESSIVE STRESS

SAMPLE 23 CLAY SAND



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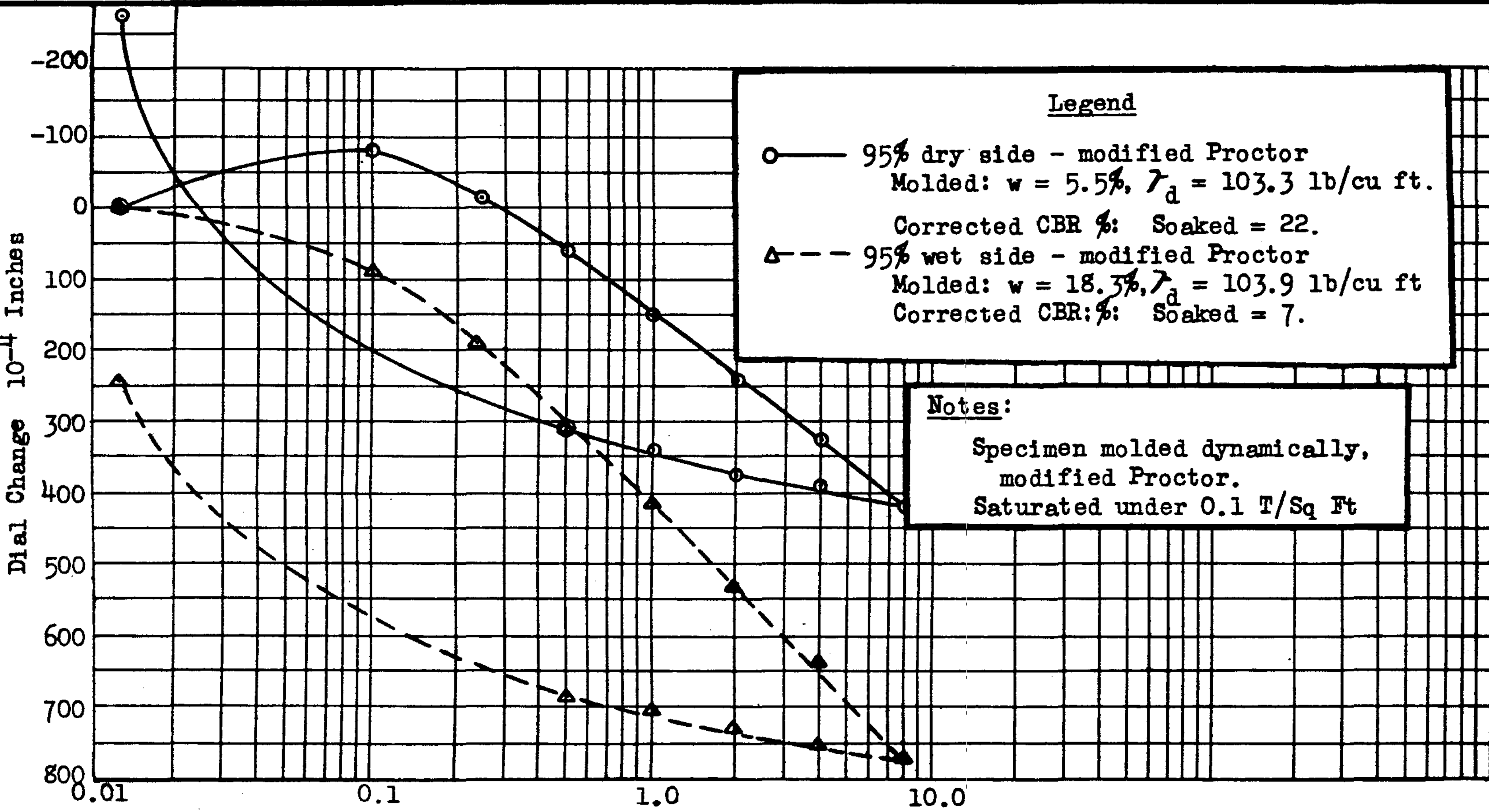
California Bearing Ratio Investigation

CONSOLIDATION TEST
(Dial Change vs Pressure)

Sample 1 (2nd Batch)

DeRidder Clay Sand

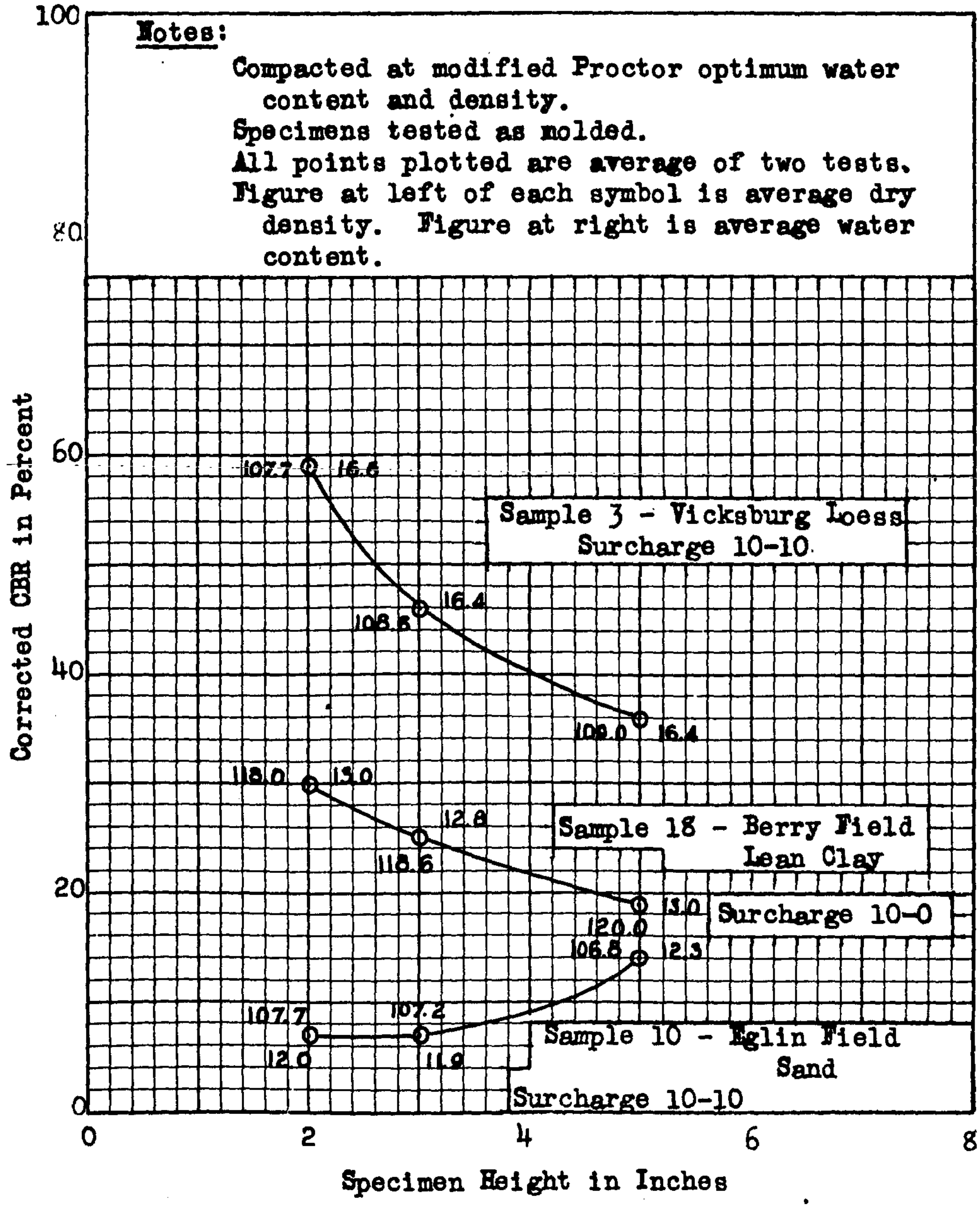


Pressure in Tons per Square Foot

Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

CONSOLIDATION TEST
(Dial Change vs Pressure)

Sample 3 Vicksburg Loess



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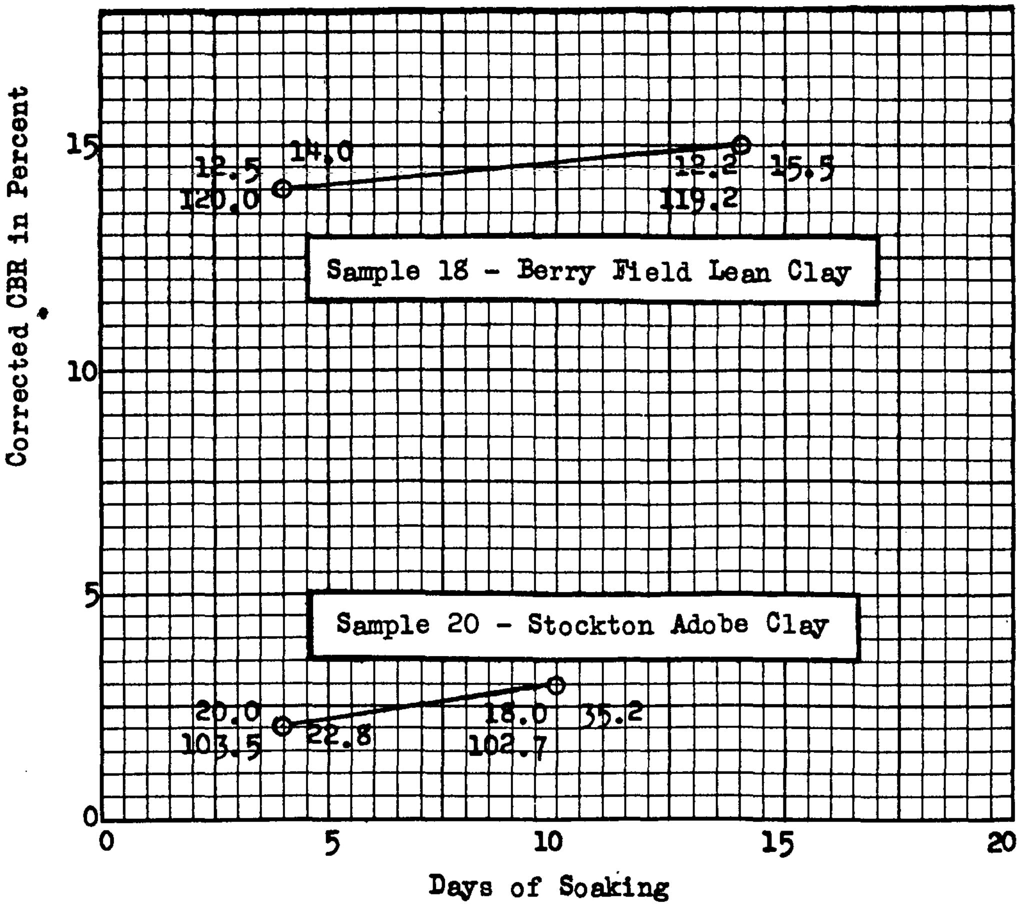
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

CBR VS HEIGHT OF SPECIMEN

Samples 3, 10, 18

Notes:

Specimens compacted approximately at modified Proctor optimum.
Figures at left of point indicate molding water content and dry density. Figure at right indicates water content of top inch after soaking.
Soaked top and bottom.
Surcharges 10-0.



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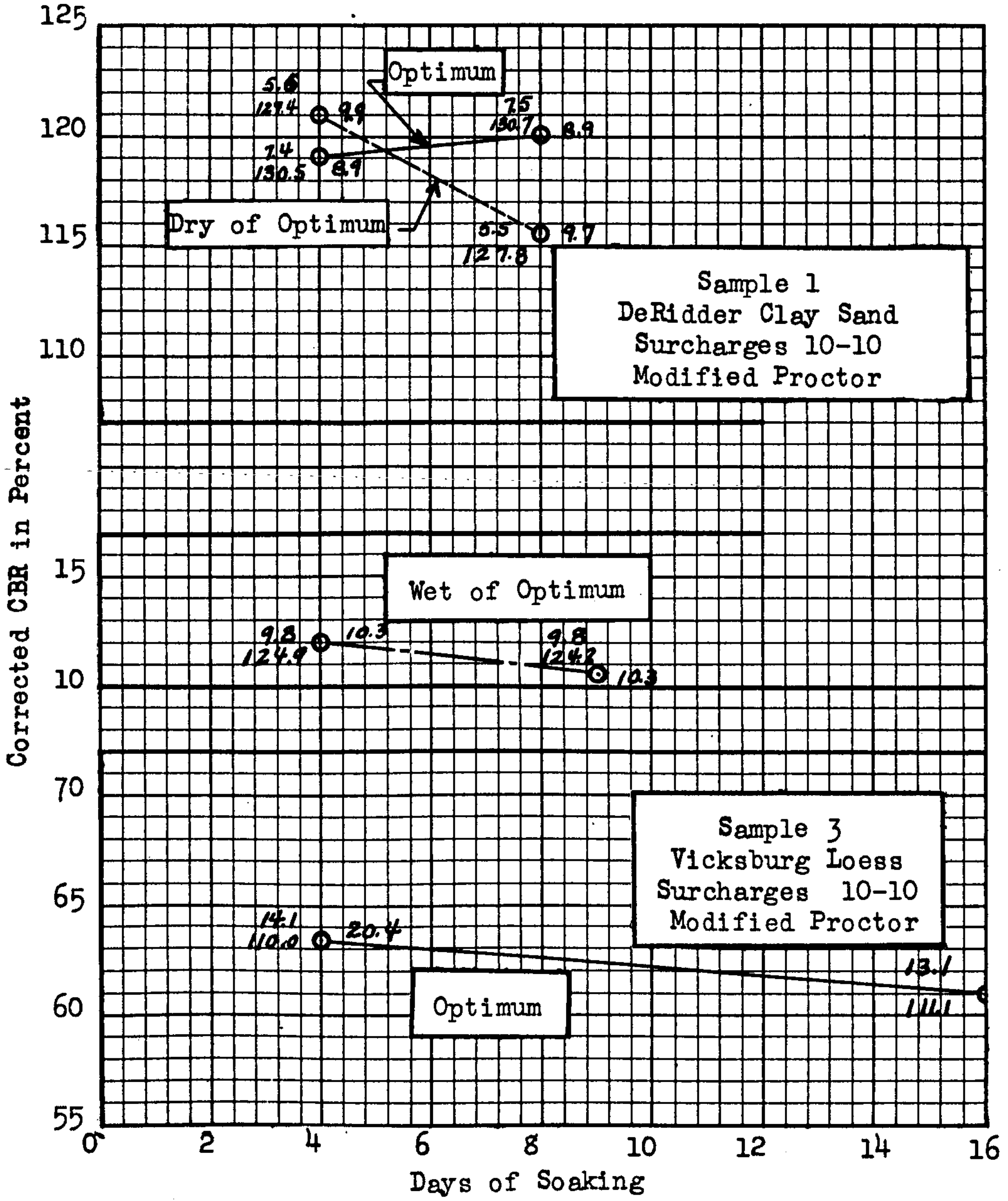
CBR VS DAYS OF SOAKING

Note: Numbers at left of point indicate molding water content and dry density. Number at right indicates water content of top inch after soaking.



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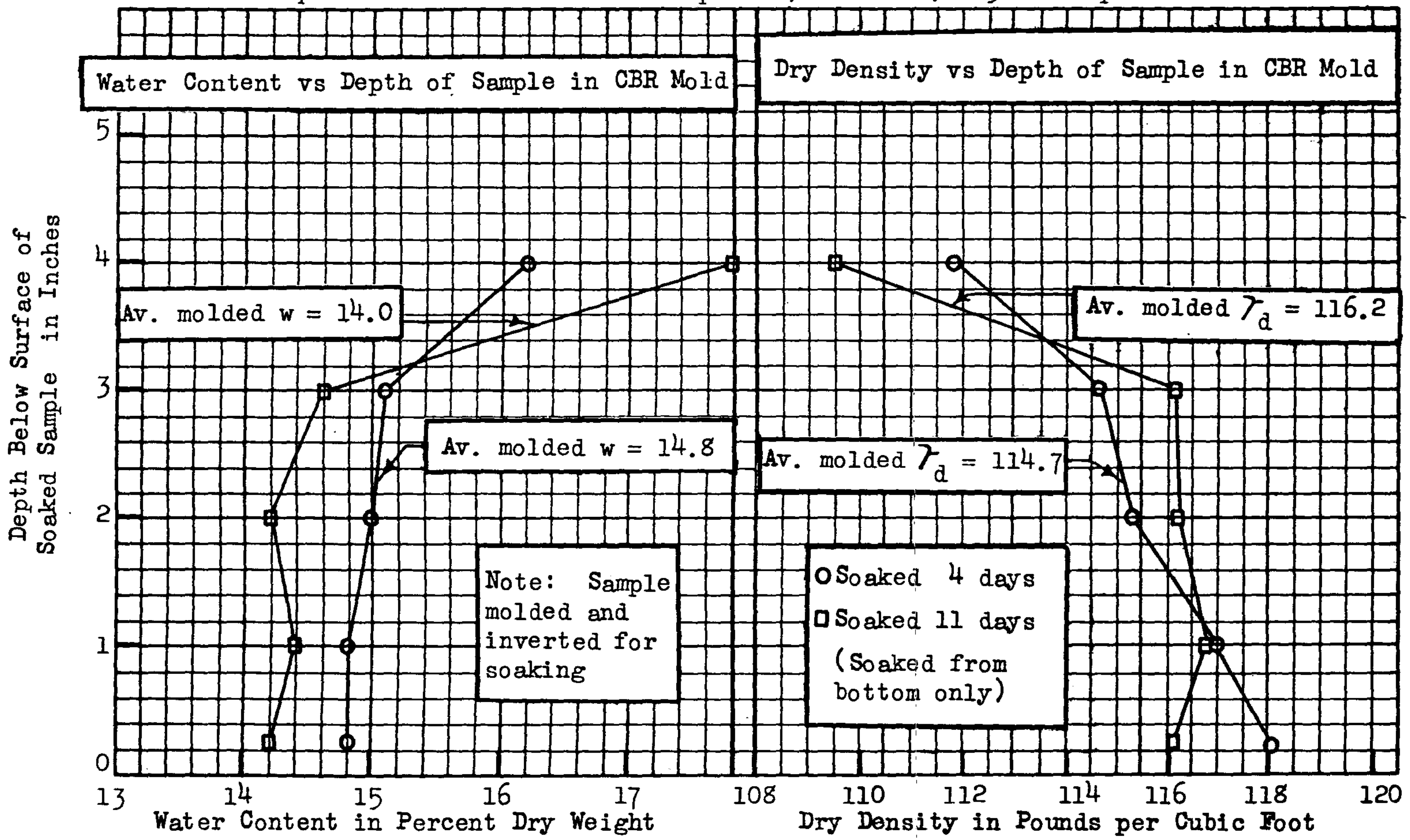
California Bearing Ratio Investigation

CBR VS DAYS OF SOAKING
(Specimens soaked from bottom only)

Compacted at modified Proctor optimum, CBR mold, 4.5-inch specimen



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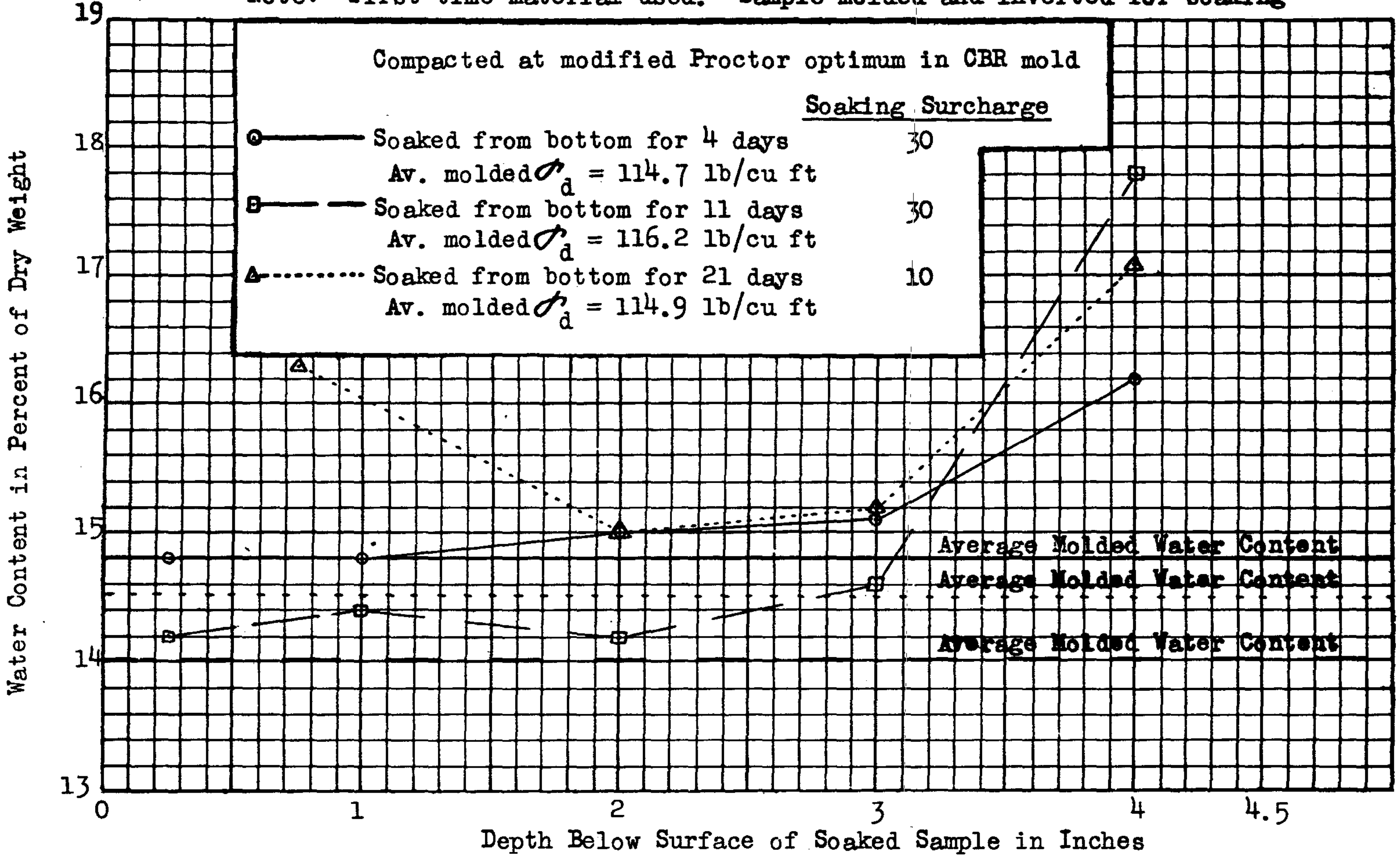
WATER CONTENT AND DRY DENSITY VS DEPTH OF SAMPLE IN CBR MOLD

Sample 18

Berry Field
Lean Clay

Note: First time material used. Sample molded and inverted for soaking



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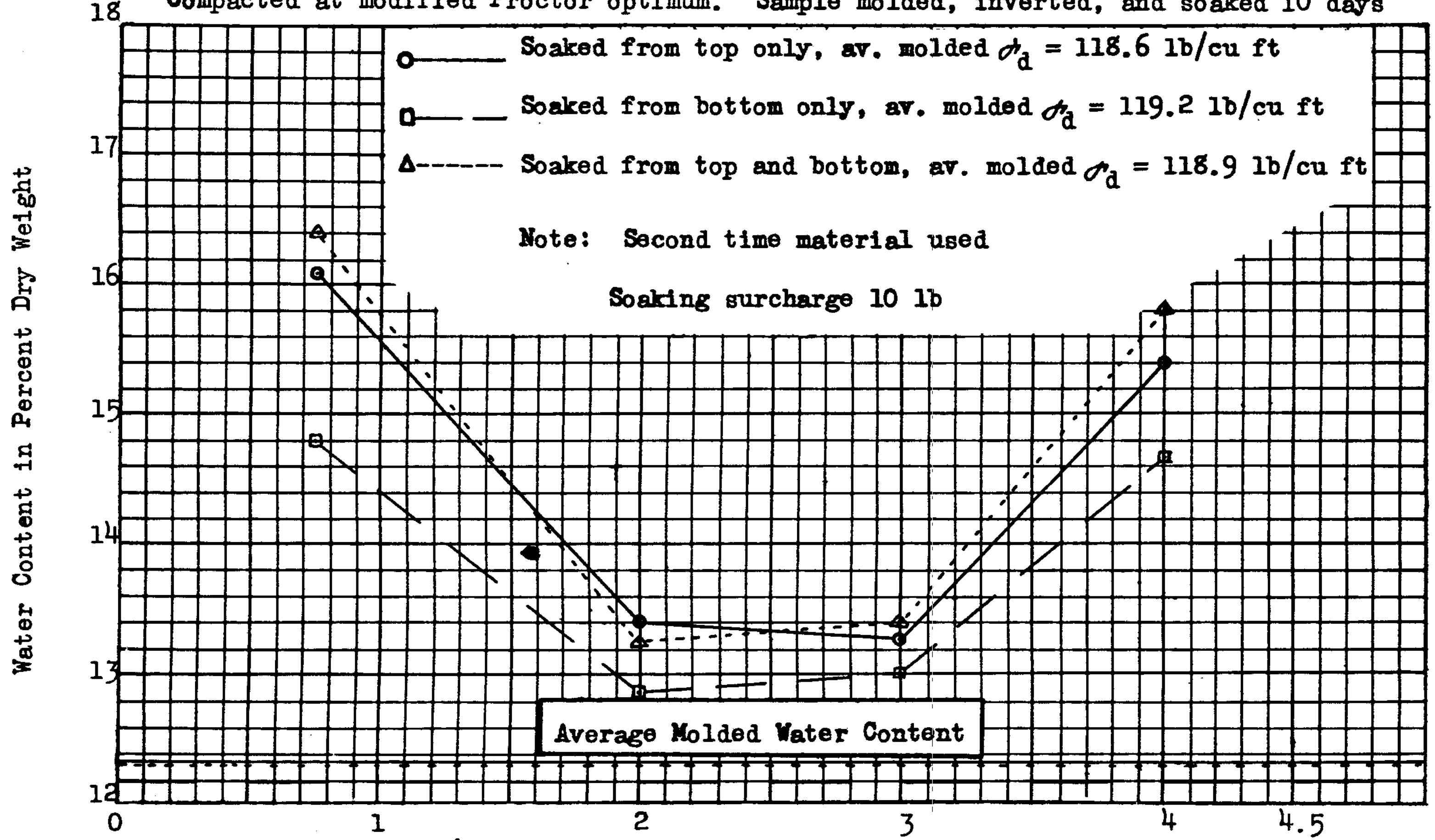
Sample 18 WATER CONTENT VS DEPTH OF SAMPLE IN CBR MOLD Berry Field Lean Clay

Compacted at modified Proctor optimum. Sample molded, inverted, and soaked 10 days



Depth Below Surface of Soaked Sample in Inches

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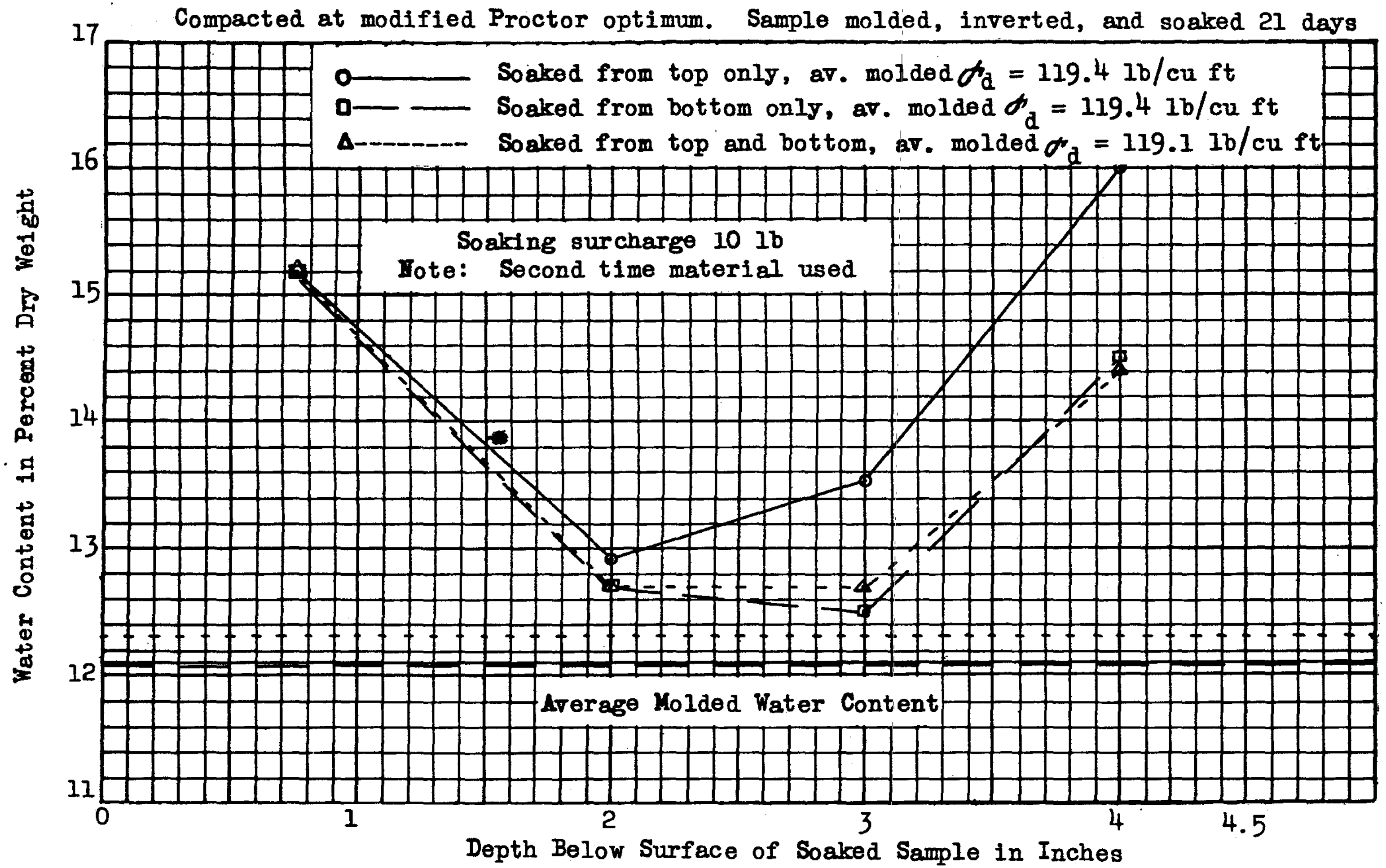
Sample 18 WATER CONTENT VS DEPTH OF SAMPLE IN CBR MOLD Berry Field Lean Clay



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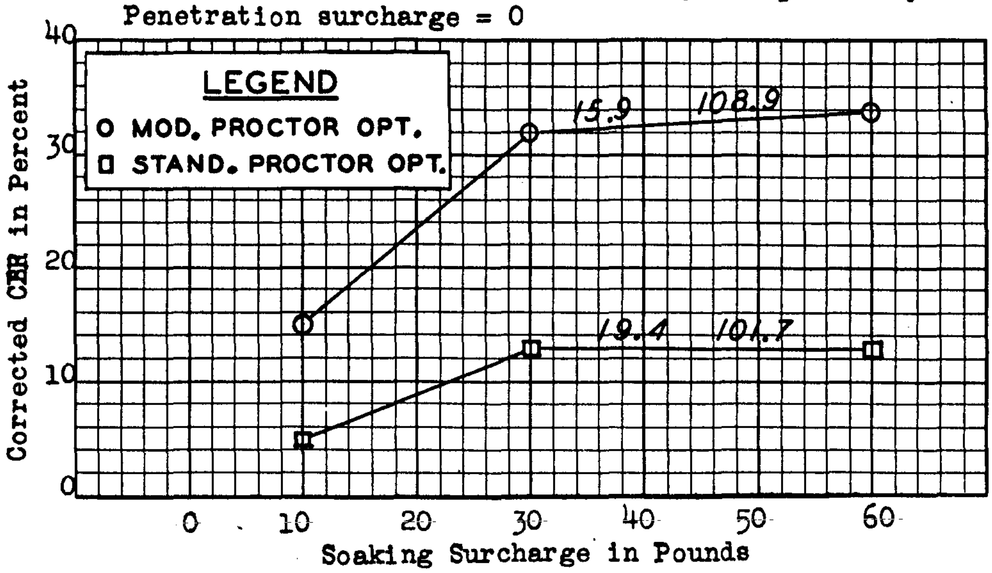
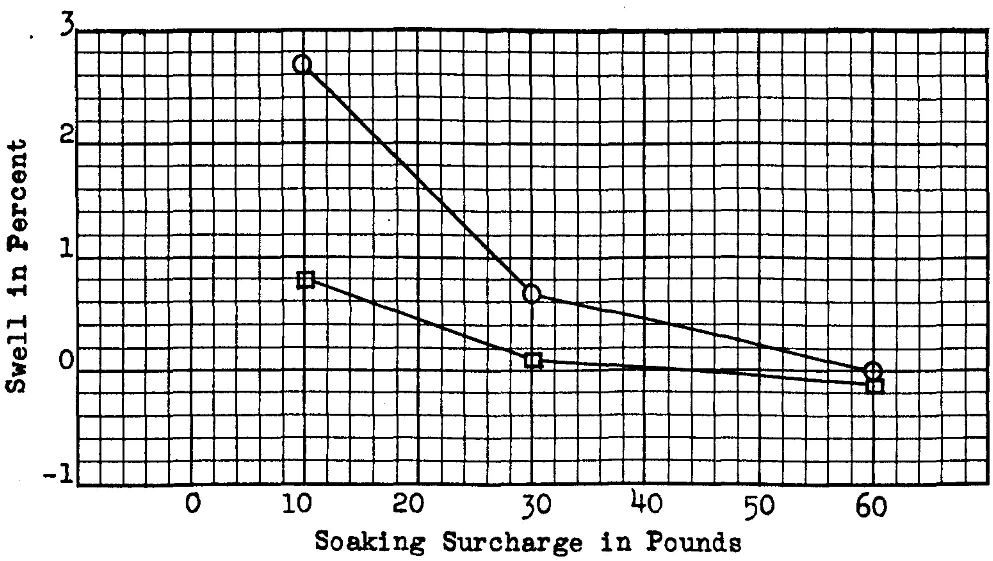
Sample 18

WATER CONTENT VS DEPTH OF SAMPLE IN CBR MOLD

Berry Field Lean Clay

Notes: All specimens soaked from top and bottom for four days. Figures beside curves indicate average molded water content and dry density, respectively. Penetration surcharge = 0





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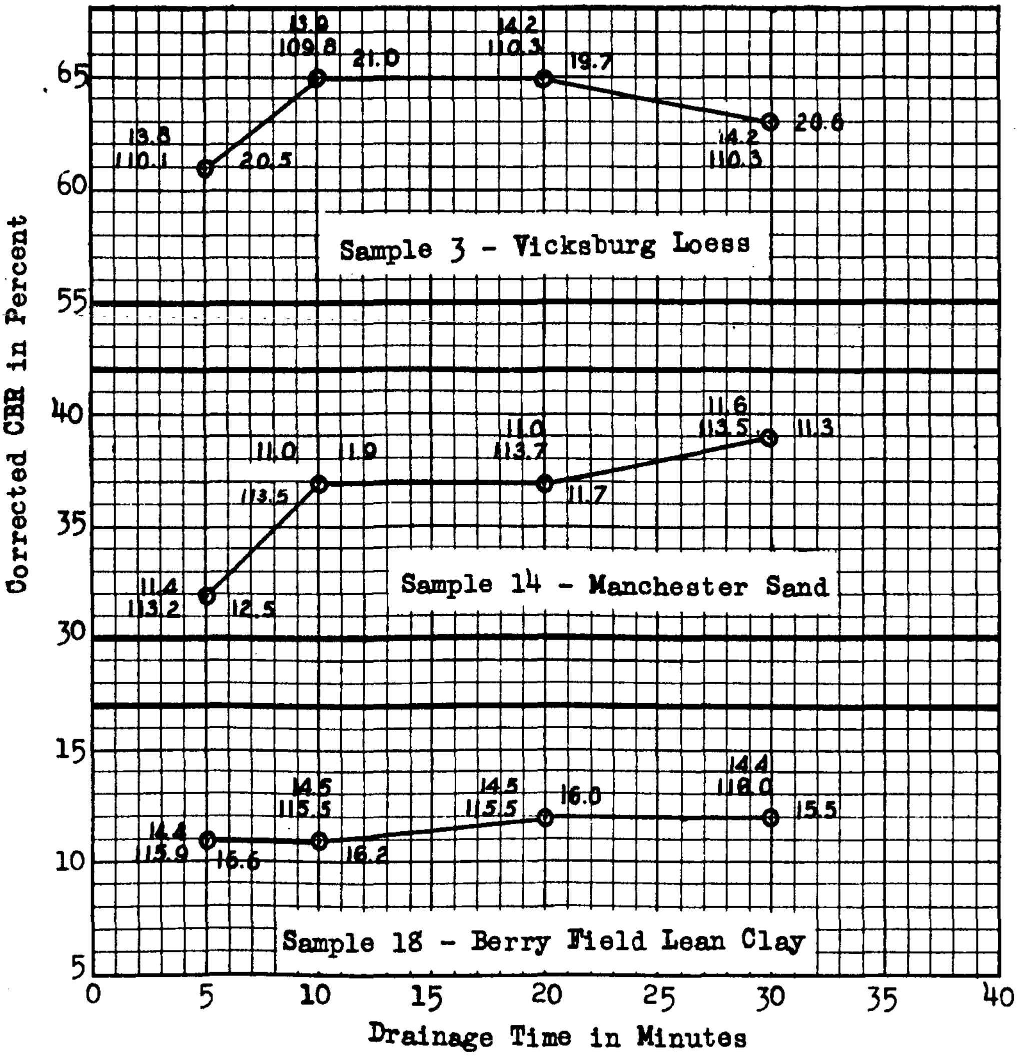
SURCHARGE VS SWELL AND CBR

Sample 3 Vicksburg Loess

Notes:
All specimens compacted at modified Proctor optimum.
Surcharges 10-10.
Points plotted are average of two or more tests.
Numbers at left of point indicate average molding water content and dry density. Number at right indicates average water content of top inch after soaking.



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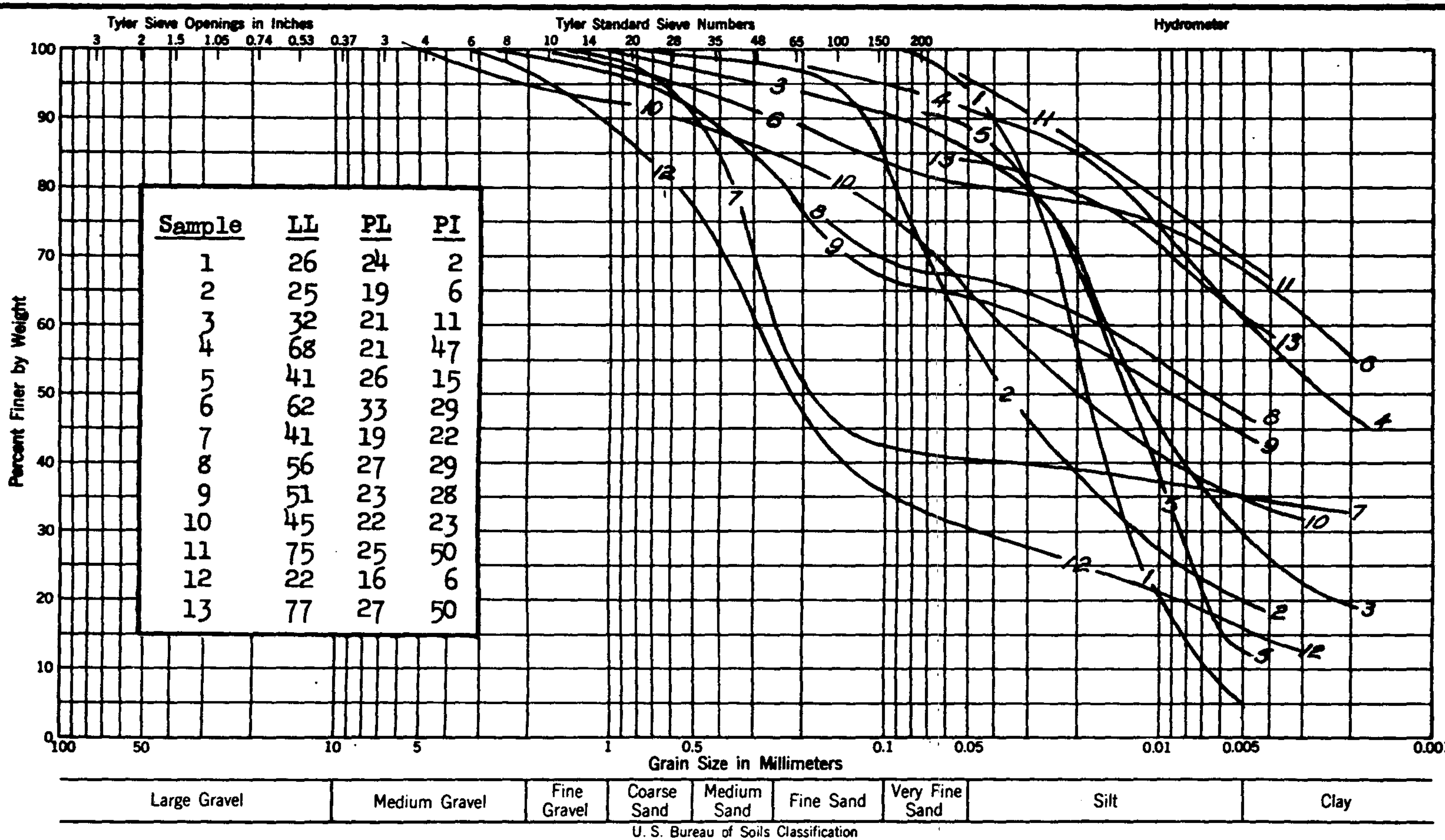
CBR VS DRAINAGE TIME

Samples 3, 14, 18



Sample	LL	PL	PI
1	26	24	2
2	25	19	6
3	32	21	11
4	68	21	47
5	41	26	15
6	62	33	29
7	41	19	22
8	56	27	29
9	51	23	28
10	45	22	23
11	75	25	50
12	22	16	6
13	77	27	50

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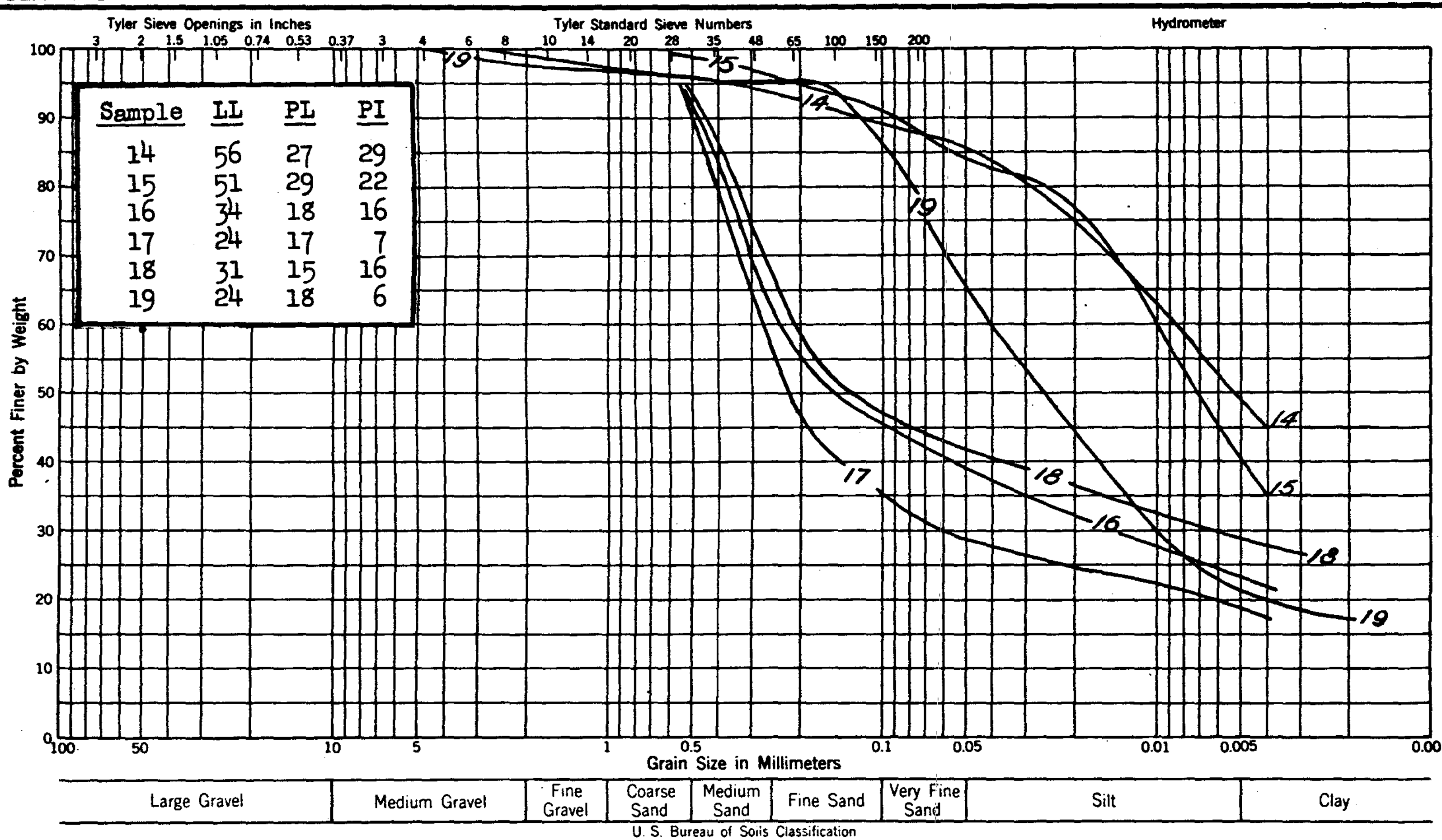
California Bearing Ratio Investigation

CLASSIFICATION DATA FOR PLASTIC SOILS

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



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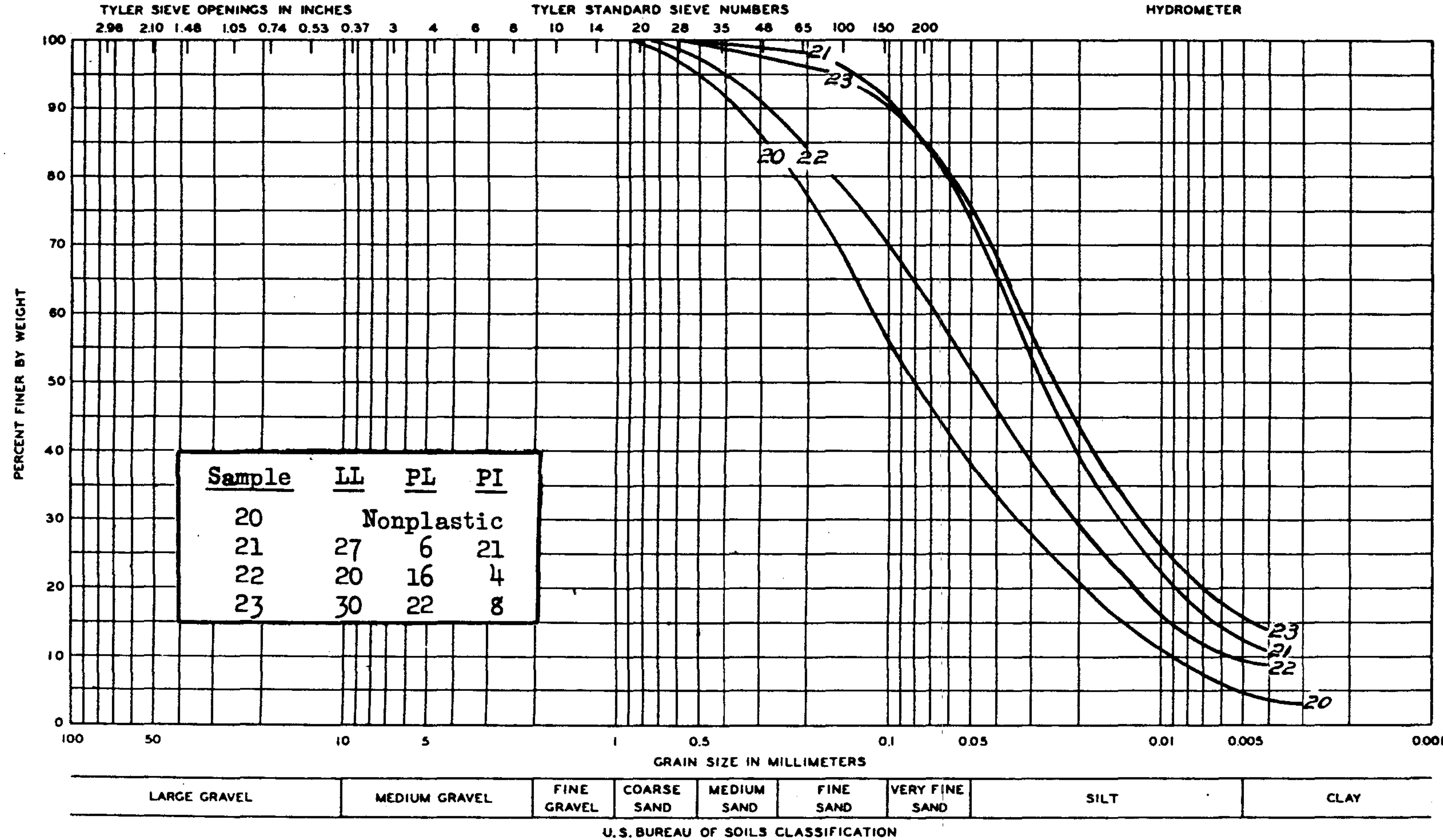
California Bearing Ratio Investigation

CLASSIFICATION DATA FOR PLASTIC SOILS



Sample	LL	PL	PI
20	Nonplastic		
21	27	6	21
22	20	16	4
23	30	22	8

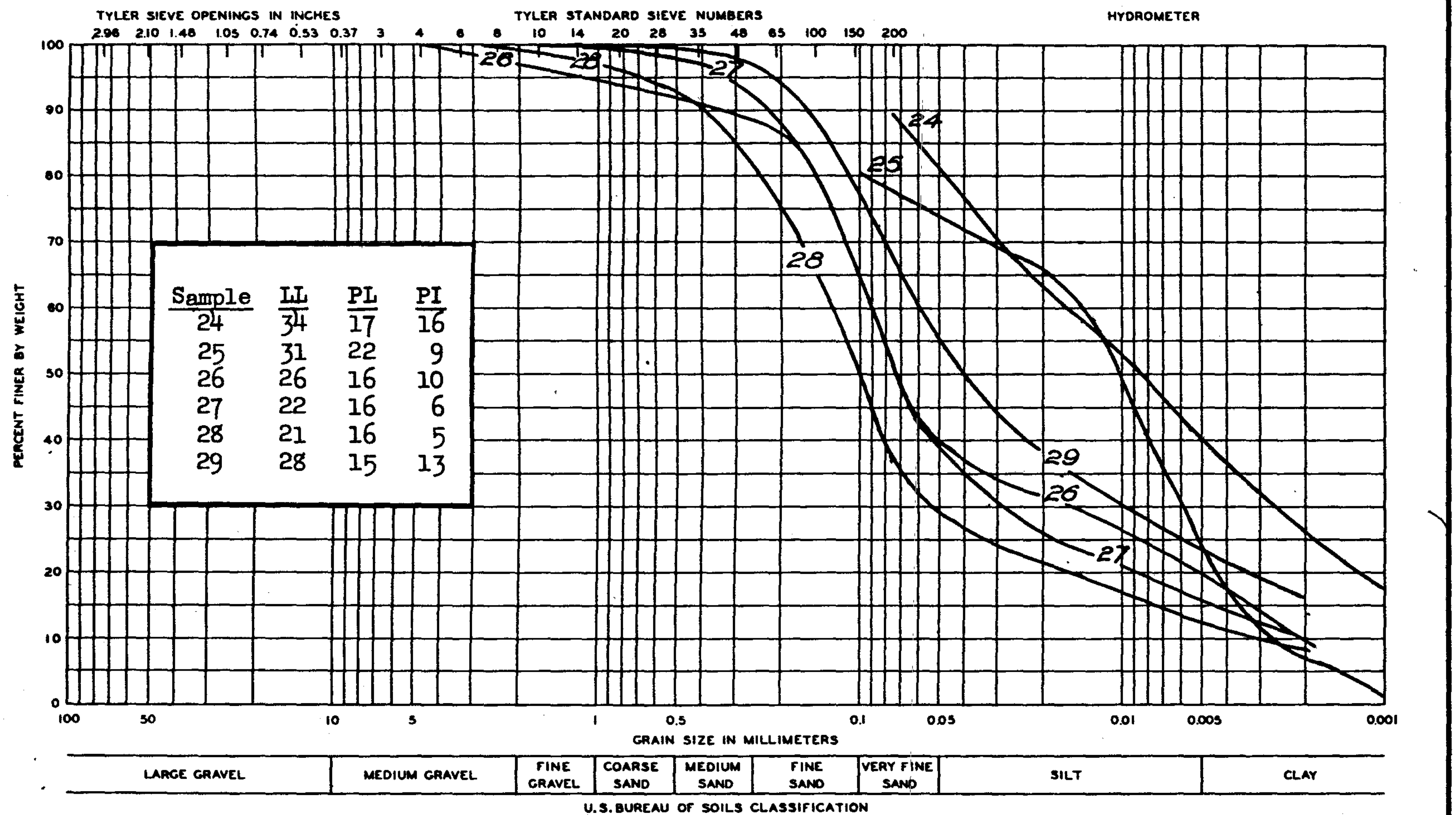
Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

CLASSIFICATION DATA FOR PLASTIC SOILS

PLATE 152

WAR DEPARTMENT

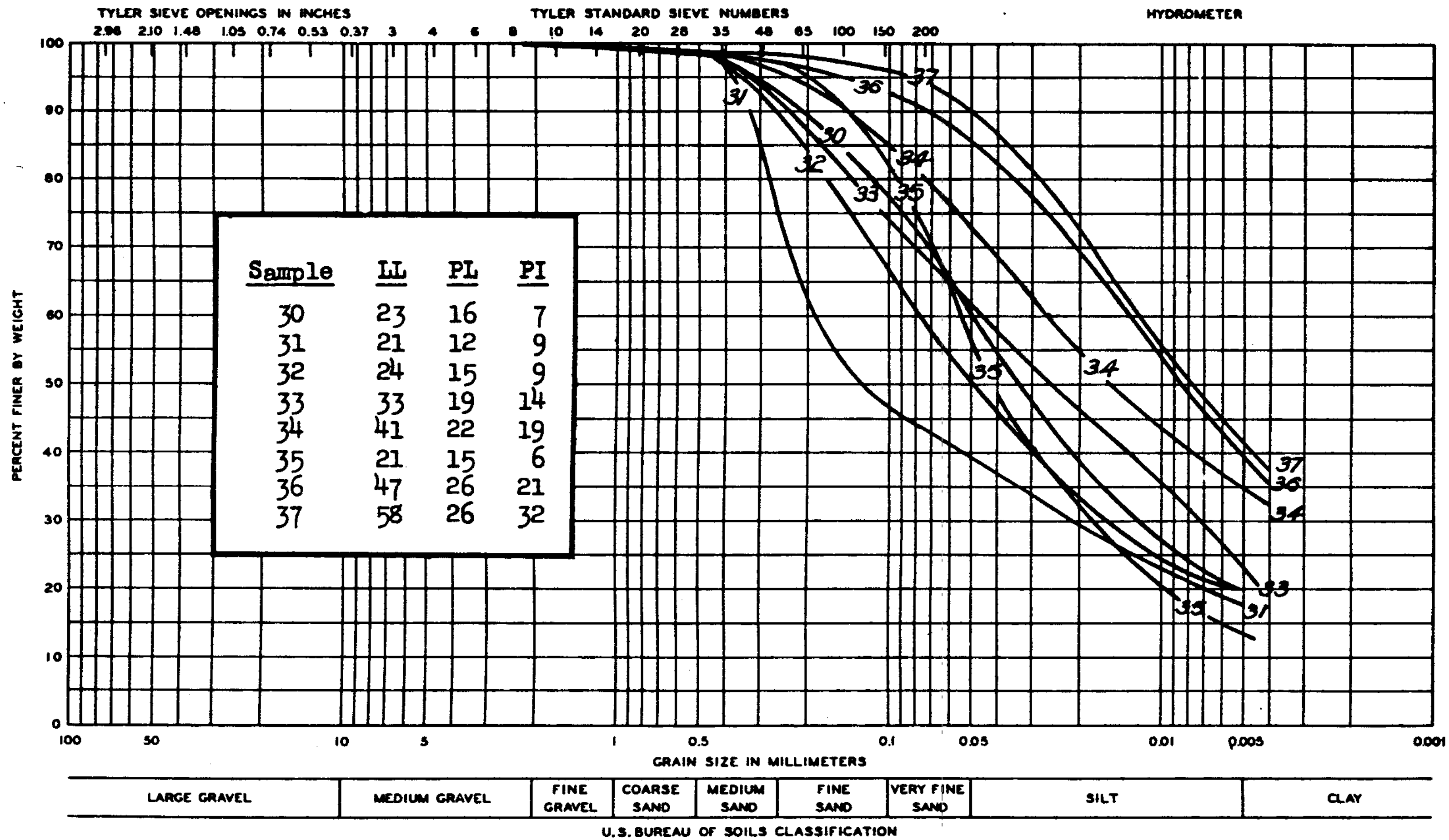
CORPS OF ENGINEERS, U. S. ARMY



Sample	LL	PL	PI
24	34	17	16
25	31	22	9
26	26	16	10
27	22	16	6
28	21	16	5
29	28	15	13

Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation

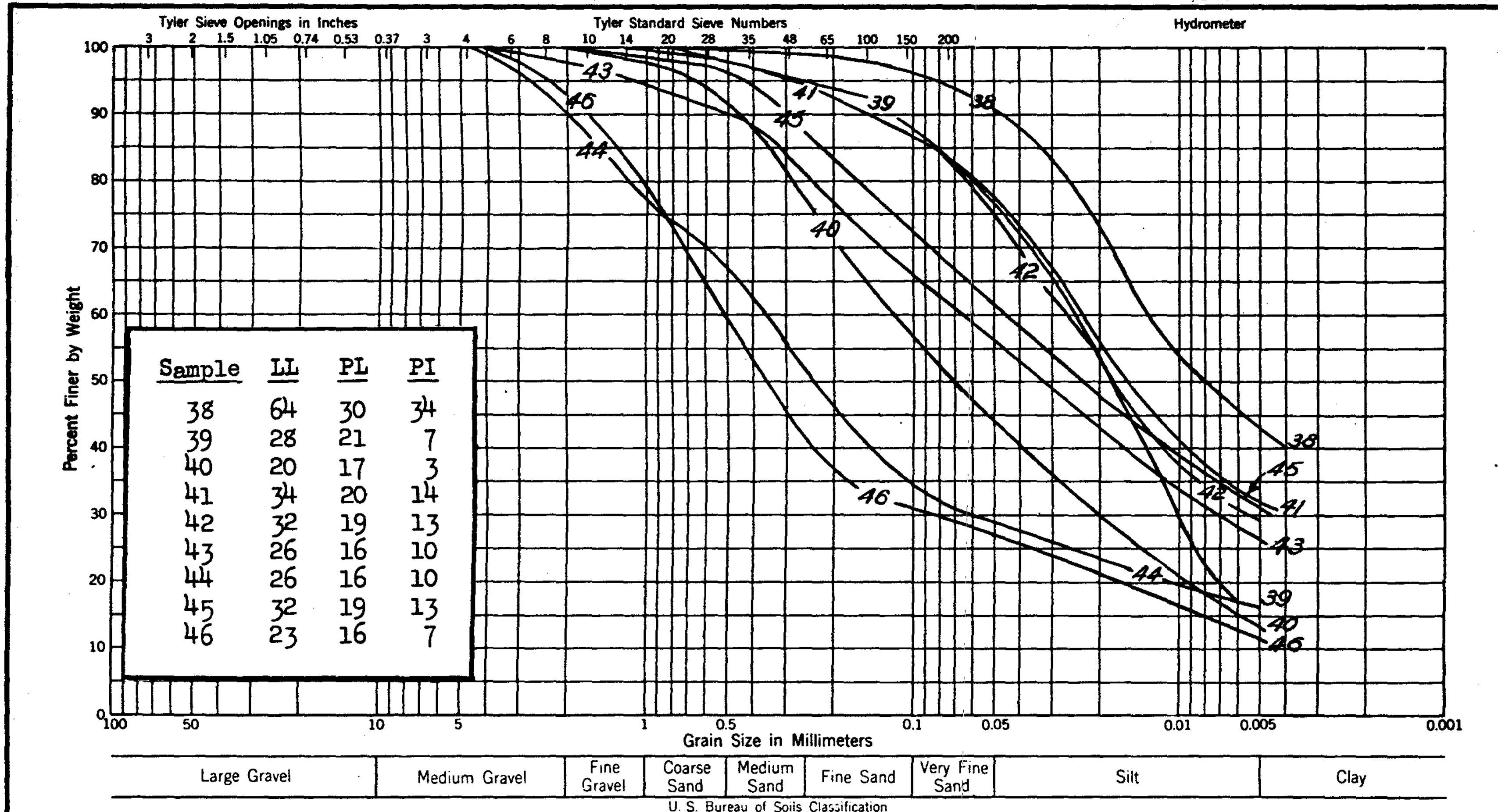
CLASSIFICATION DATA FOR PLASTIC SOILS



Sample	LL	PL	PI
30	23	16	7
31	21	12	9
32	24	15	9
33	33	19	14
34	41	22	19
35	21	15	6
36	47	26	21
37	58	26	32

Mississippi River Commission
U. S. Waterways Experiment Station
California Bearing Ratio Investigation
CLASSIFICATION DATA FOR PLASTIC SOILS

WAR DEPARTMENT CORPS OF ENGINEERS U. S. ARMY



Sample	LL	PL	PI
38	64	30	34
39	28	21	7
40	20	17	3
41	34	20	14
42	32	19	13
43	26	16	10
44	26	16	10
45	32	19	13
46	23	16	7

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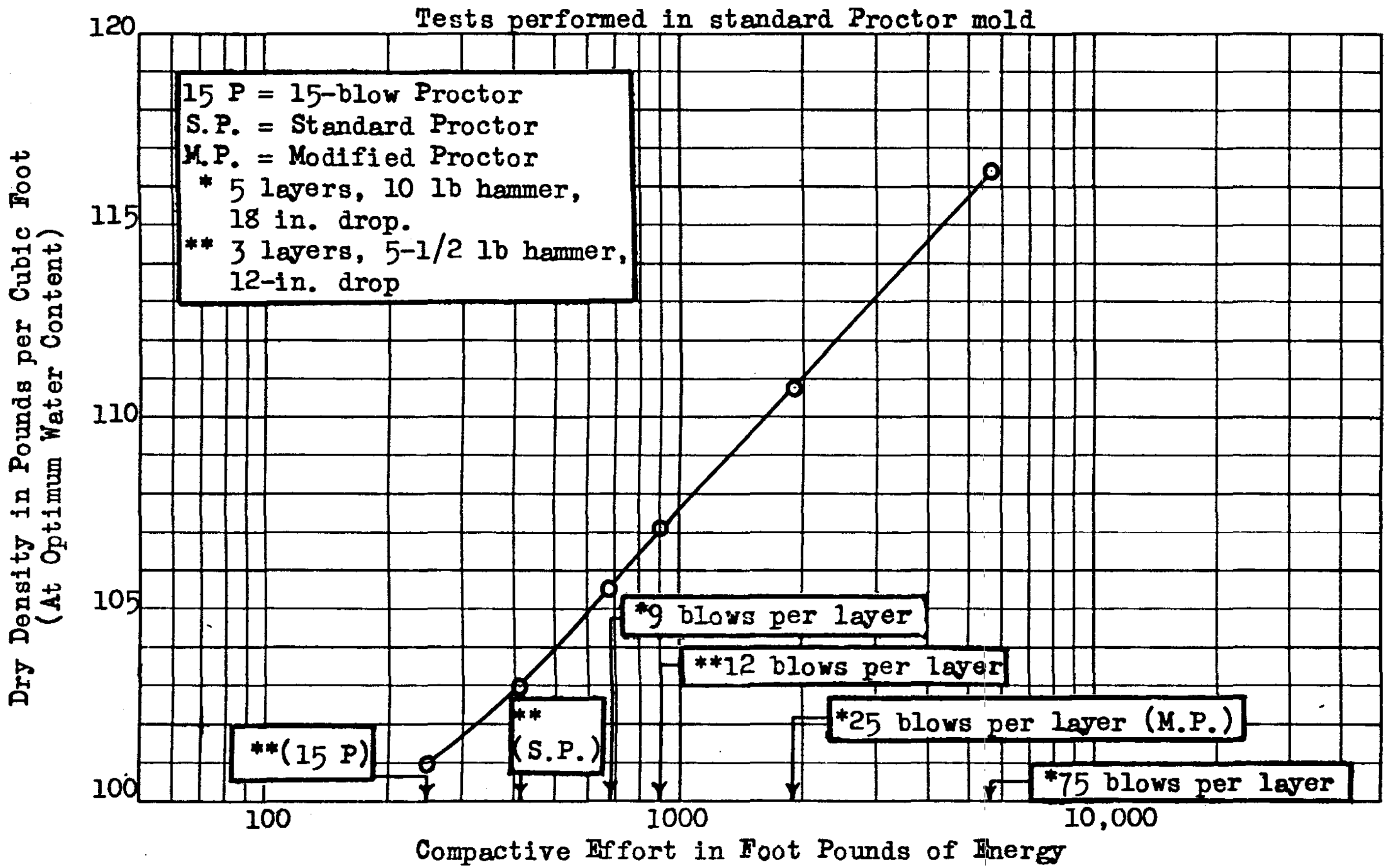
California Bearing Ratio Investigation

CLASSIFICATION DATA FOR PLASTIC SOILS



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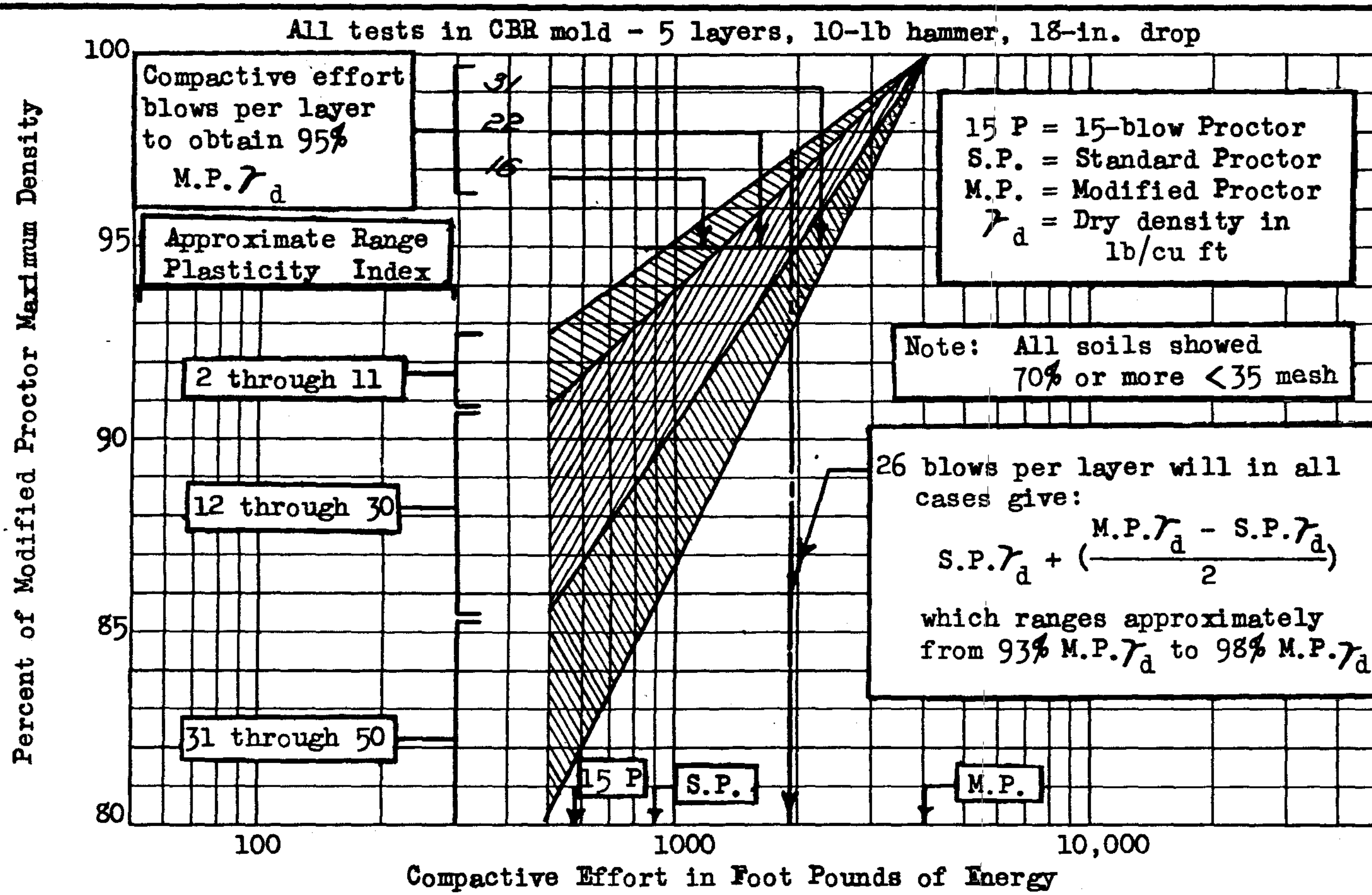
California Bearing Ratio Investigation

COMPACTIVE EFFORT VS DRY DENSITY AT OPTIMUM WATER CONTENT

Vicksburg Loess



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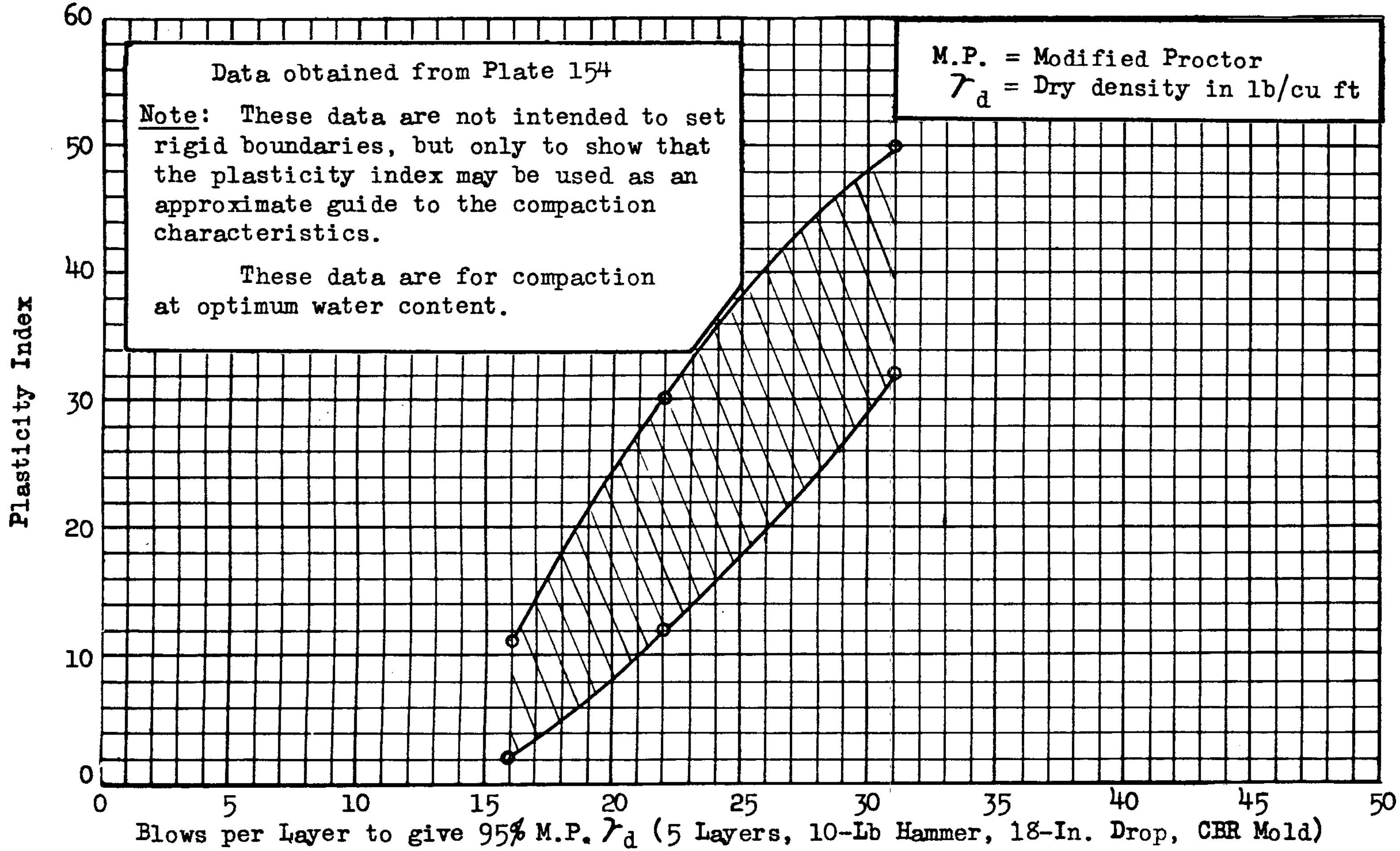
COMPACTIVE EFFORT VS PERCENT OF MODIFIED PROCTOR MAXIMUM DENSITY
(At Optimum Water Content for the Effort Used)



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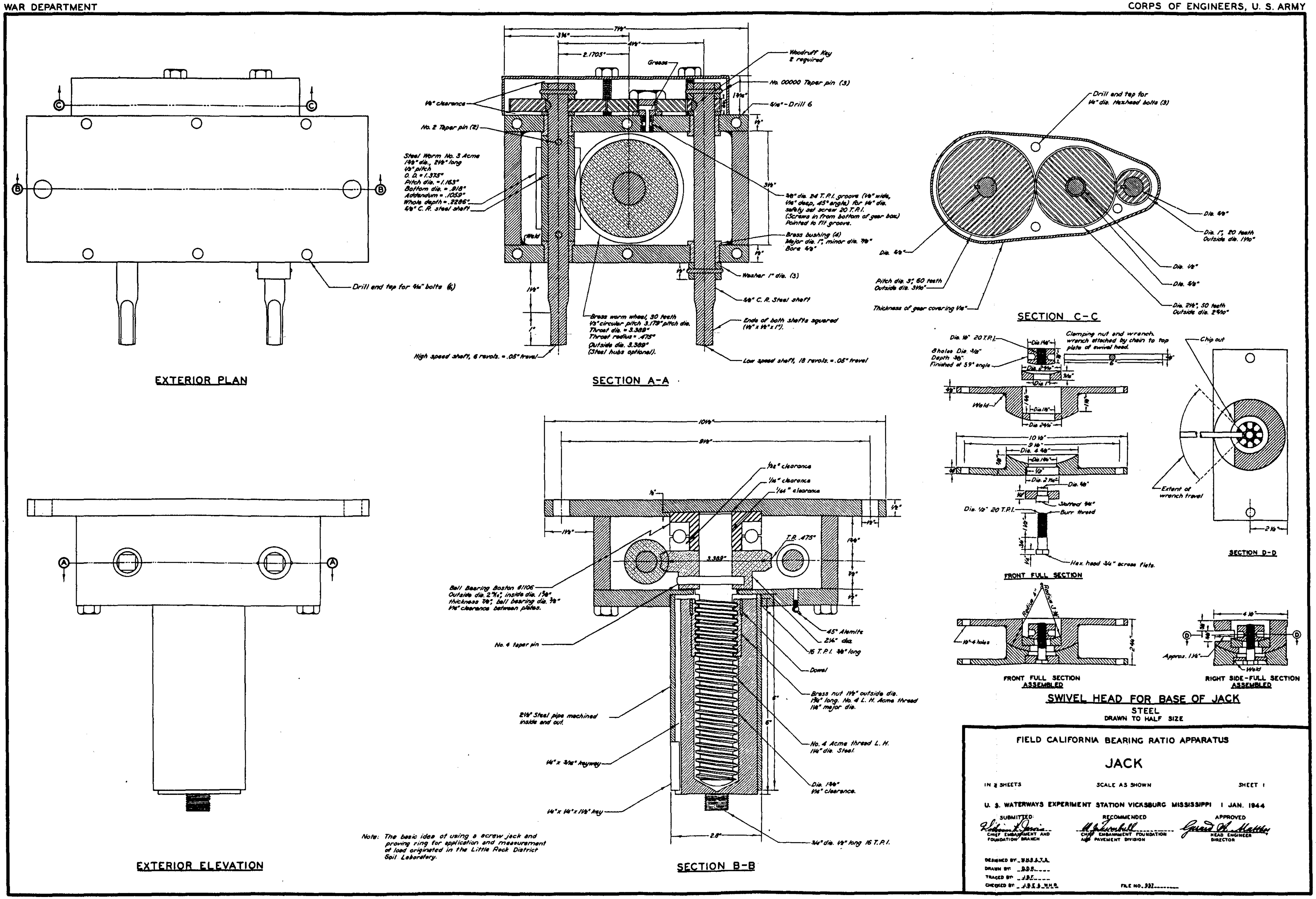
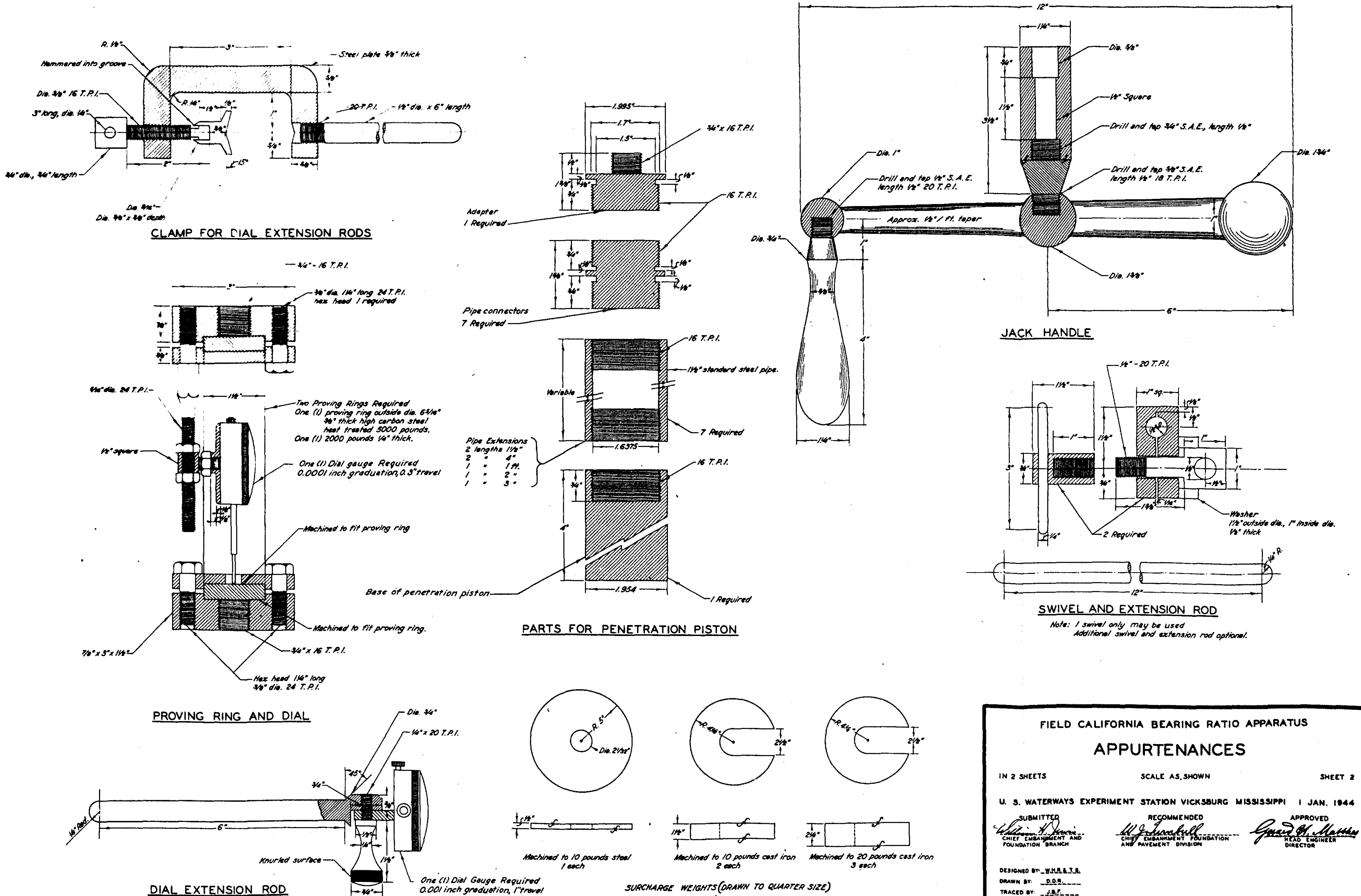
California Bearing Ratio Investigation

PLASTICITY INDEX VS COMPACTIVE EFFORT TO GIVE 95% MODIFIED PROCTOR DENSITY


EXTERIOR PLAN
Drill and tap for 9/16" bolts (6)
SECTION A-A
Woodruff Key 2 required
No. 00000 Taper pin (3)
Grease
1/8" clearance
No. 2 Taper pin (2)
Steel Worm No. 3 Acme 1 1/2" dia., 2 1/2" long 1/3" pitch O.D. = 1.375" Pitch dia. = 1.163" Bottom dia. = .918" Addendum = .1059" Whole depth = .2286" 5/8" C. R. steel shaft
Weld
Brass worm wheel, 30 teeth 1/3" circular pitch 3.179" pitch dia. Throat dia. = 3.388" Throat radius = .475" Outside dia. 3.389" (Steel hubs optional).
High speed shaft, 6 revols. = .05" travel
Brass bushing (4)
Washer 1" dia. (3)
5/8" C. R. Steel shaft
Ends of both shafts squared
Low speed shaft, 18 revols. = .05" travel
SECTION C-C
Drill and tap for 1/4" dia. Hexhead bolts (3)
Pitch dia. 3", 60 teeth Outside dia. 3 1/10"
Thickness of gear covering 1/8"
Dia. 2 1/2", 50 teeth Outside dia. 2 6/10"
Dia. 1", 20 teeth Outside dia. 1 1/10"
EXTERIOR ELEVATION
SECTION B-B
Ball Bearing Boston 61106
No. 4 taper pin
2 1/2" Steel pipe machined inside and out.
45° Alemite
16 T.P.I.
Dowel
Brass nut
No. 4 Acme thread L. H. 1 1/4" dia. Steel.
3/4" dia. 1/2" long 16 T.P.I.
Note: The basic idea of using a screw jack and proving ring for application and measurement of load originated in the Little Rock District Soil Laboratory.
FRONT FULL SECTION
Hex. head 3/4" across flats.
SECTION D-D
Chip out
Extent of wrench travel
FRONT FULL SECTION ASSEMBLED
RIGHT SIDE-FULL SECTION ASSEMBLED
SWIVEL HEAD FOR BASE OF JACK
STEEL
DRAWN TO HALF SIZE
FIELD CALIFORNIA BEARING RATIO APPARATUS
JACK
IN 2 SHEETS
SCALE AS SHOWN
SHEET 1
U. S. WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI 1 JAN. 1944
SUBMITTED: CHIEF EMBANKMENT AND FOUNDATION BRANCH
RECOMMENDED: CHIEF EMBANKMENT FOUNDATION AND PAVEMENT DIVISION
APPROVED: Gerard H. Matthes HEAD ENGINEER DIRECTOR



FIELD CALIFORNIA BEARING RATIO APPARATUS

APPURTENANCES

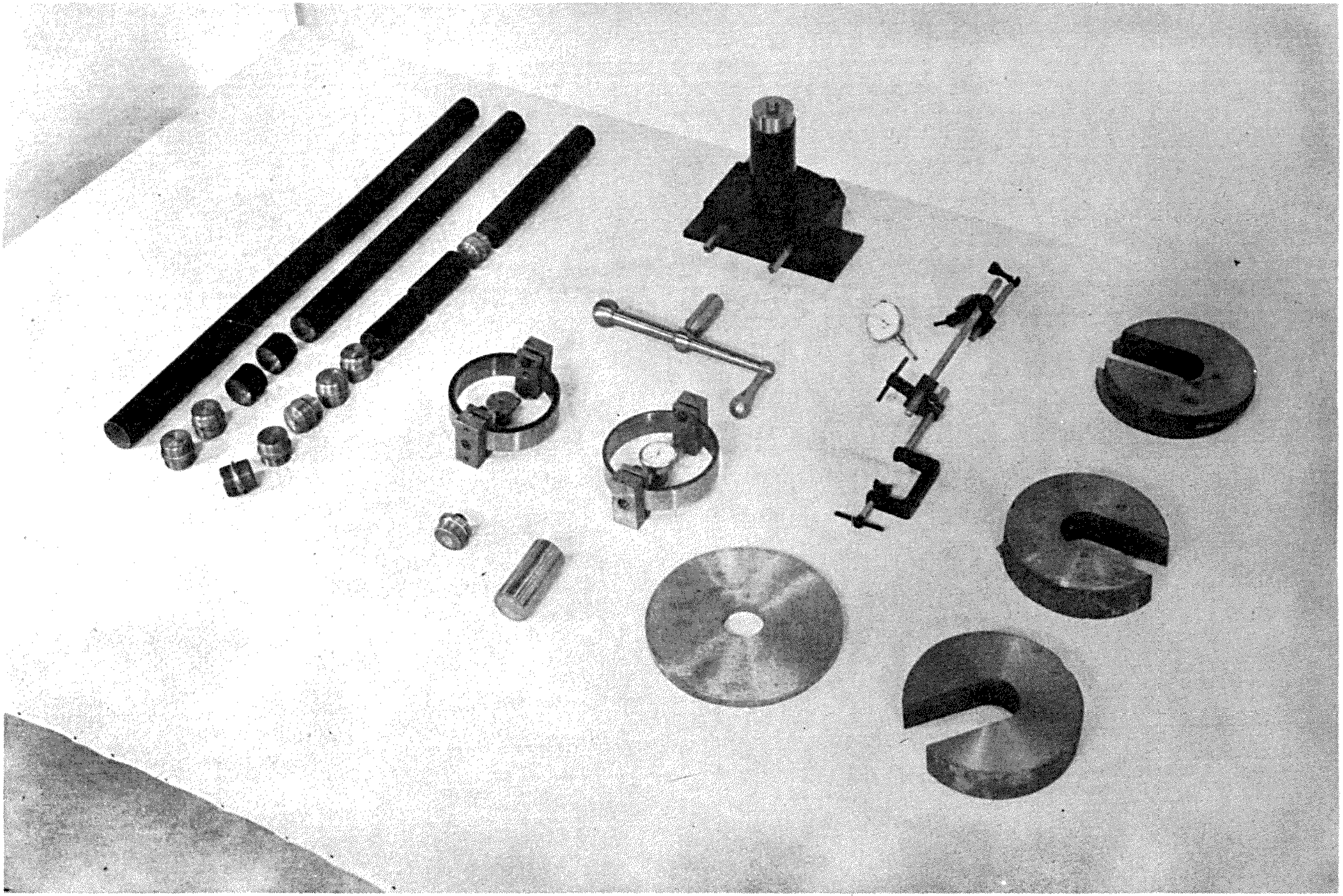
IN 2 SHEETS — SCALE AS SHOWN — SHEET 2

U. S. WATERWAYS EXPERIMENT STATION VICKSBURG MISSISSIPPI 1 JAN. 1944

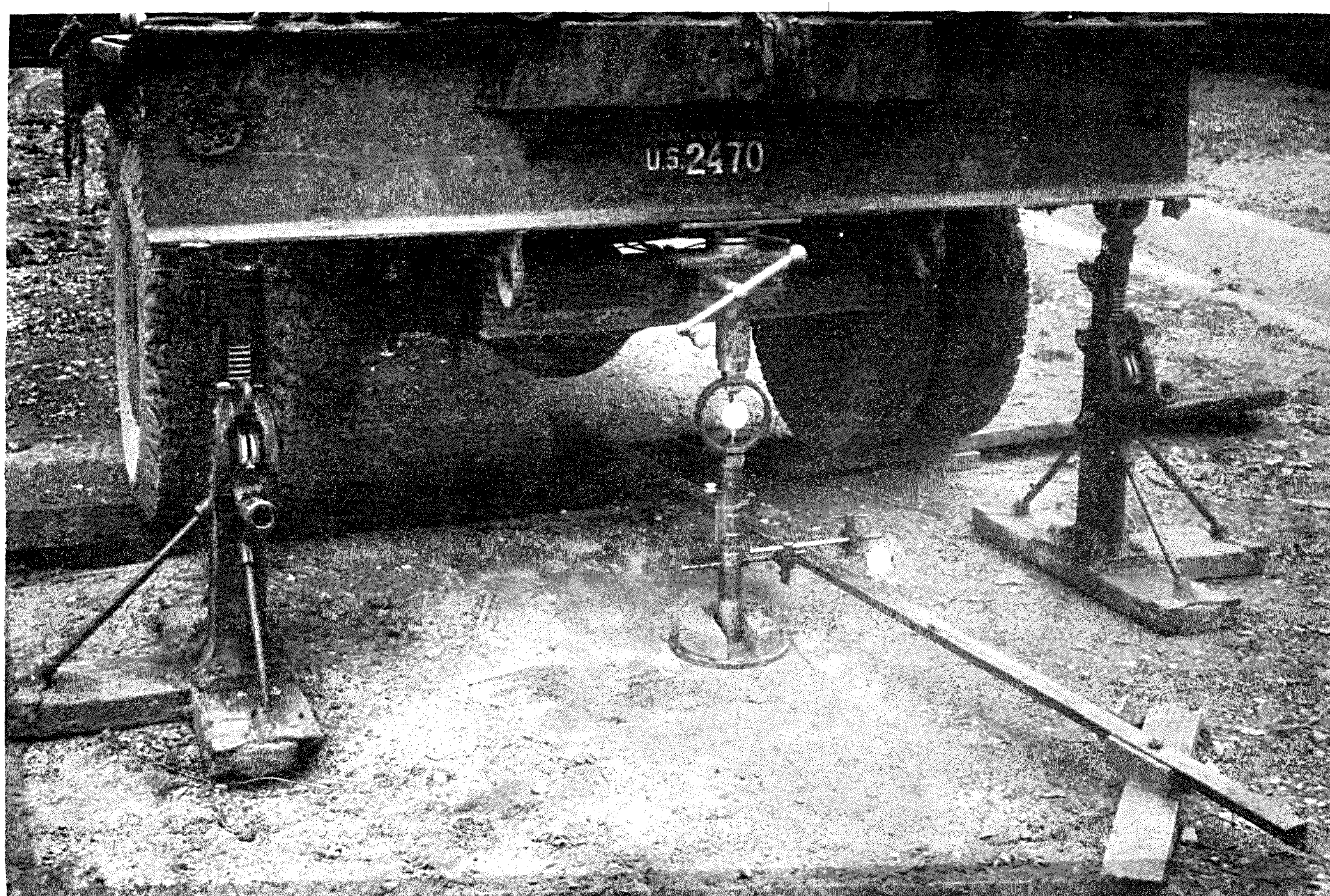
SUBMITTED	RECOMMENDED	APPROVED
CHIEF EMBANKMENT AND FOUNDATION BRANCH	CHIEF EMBANKMENT FOUNDATION AND PAVEMENT DIVISION	HEAD ENGINEER DIRECTOR

DESIGNED BY: W.H.R. & T.S.
DRAWN BY: D.D.R.
TRACED BY: J.B.F.
CHECKED BY: J.B.E. & W.H.R.
FILE NO. 357

PLATE 160



FIELD-IN PLACE CALIFORNIA BEARING RATIO APPARATUS (UNASSEMBLED)



FIELD IN-PLACE CALIFORNIA BEARING RATIO APPARATUS (ASSEMBLED)