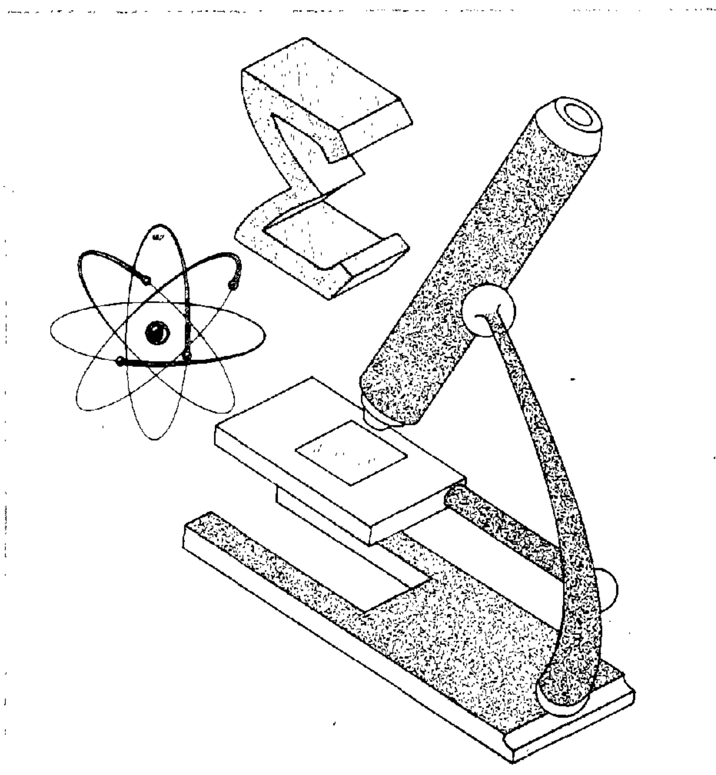
Report No. FHWA-RD-77-101

DETERMINATION OF CONSISTENCY CHARACTERISTICS OF SOILS



May 1977

Final Report

Prepared for

FEDERAL HIGHWAY ADMINISTRATION

Offices of Research & Development

Washington, D. C. 20590

NOTICE

Technical Report Documentation Page

1. Report No. FHWA-RD-77-101	2. Government Accession No.	3. Recipient's Catalog No. PB277465
4. Title and Subtitle Determination of Consistency Characteristics of Soils		5. Report Date May, 1977
		6. Performing Organization Code
7. Author(s) P.J. Huck, M.N. Iyengar		8. Performing Organization Report No. D6109-FR
9. Performing Organization Name and Address IIT Research Institute 10 West 35th Street Chicago, Illinois 60616		10. Work Unit No. (TRAIS) FCP 34F1-263
		11. Contract or Grant No. DOT-FH-11-8754
12. Sponsoring Agency Name and Address Offices of Research and Development Federal Highway Administration U.S. Department of Transportation Washington, D.C. 20590		13. Type of Report and Period Covered Final Report
		14. Sponsoring Agency Code M/0441

15. Supplementary Notes

FHWA Contract Manager: Jerome Blystone (HRS-21)

16. Abstract

The mechanical response of soil may be characterized as stiff, transition, or fluid, depending upon its consistency at various moisture contents. This program was charged with the development of test method and hardware not having the disadvantages of traditional plasticity tests. In particular, the test method should not require removal of aggregate or drying to near zero moisture content. It should test the soil under conditions similar to earthwork in the field, yet have the potential for determining the effect of any stabilization or other treatment that might be used in highway construction.

After reviewing numerous parameters, it was determined that the most attractive approach is to measure strength or stiffness of soil specimens compacted for moisture-density determination (AASTHO T-99 or ASTM D-698). Subsequent testing with 13 soil types indicated that the coefficient of earth pressure at rest (K_o) is appropriate both as a classification test and for use in earthwork design. The K_o test device and procedure are fully described.

(Kaub)

17. Key Words Atterberg limits, K_o test, moisture content, soil consistency, soil classification, soil stiffness, soil strength, soil tests.	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	96	PC A05/MF-A01

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

FOREWORD

This project was funded by the Department of Transportation, Federal Highway Administration, Office of Research under contract DOT-FH-11-8754. The support and guidance of the FHWA technical staff, in particular Messrs. Jerome Blystone and Don Fohs, are gratefully acknowledged.

Mr. Huck acted as program manager for IIT Research Institute, with much of the direct day to day effort in the hands of Mr. Iyengar. Contributors to the program included E. Aleshin, D. Cahill, D. Dickson, D. Fedor, J. Jeka, M. Ranada, and S. Varadhi.

A portion of the project was conducted under subcontract to Illinois Institute of Technology. A major contribution was made by Professor Eben Vey and it is with great sorrow that we note his recent death. This portion of the effort was completed by Dr. S. Saxena with the assistance of M. Secora.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS
(Cont'd)

LIST OF TABLES

DETERMINATION OF CONSISTENCY CHARACTERISTICS OF SOILS

1.0 INTRODUCTION

The design and construction of earth structures such as highways depends upon soil classification systems that organize soils into discrete groups displaying similar behavior. In the past, such systems have typically incorporated grain size data and soil consistency parameters such as the Atterberg limits. Because soil classification data may be used contractually for material acceptance purposes, high standards of test repeatability are necessary to avoid dispute when identifying particular soils. A soil classification test recognizing that soils form a smooth spectrum rather than discrete groups would be preferable. Further, it is desirable that a soil classification system be related as directly as possible to earthwork performance. Ideally, the same test method should be usable for classification, design and acceptance.

The Atterberg limits were intended to separate the moisture regimes in which soils display significantly different behavior (Bauer, 1959). Their widespread use has been paralleled by the propagation of modified apparatus and procedures. In addition to differences in the design of groove cutting tools and base resiliency in the liquid limit apparatus, a number of one-point methods and variations in mixing procedure contribute to minor variations between test laboratories. The test method has the further disadvantage that it requires that the soil be dried and aggregate larger than #40 (0.42 mm. opening) be removed prior to testing (ASTM D-423). Oven drying raises concern that irreversible changes in soil fabric may occur in some soils, while we would prefer to test the entire soil without removing aggregate. In short, while the Atterberg limits have provided great service since their development in the early decades of this century, there is a definite need for improved methods of characterizing soil consistency.

1.1 Program Objectives

The objective of the present program was to develop a "practical, automated methodology for measuring soil and soil aggregate response to moisture changes." The test method to be developed must be able to evaluate soils containing up to two inch (5 cm) gravel. During the early stages of the project, more specific test characteristics were developed for internal use in evaluating candidate test methods.

Test Significance

It is desirable that the test method be related to performance in earth structures. Ideally, the test should provide quantitative data appropriate for use in the rational design methods currently being developed. Thus, it is best if the test can operate on soil that is compacted and treated as it would be in the field. If stabilization treatment is considered, for example, the treated soil should be tested. Similarly, if soil is to be compacted at one moisture content, but it is anticipated that the service conditions may involve changes in moisture, this too should be subject to direct test.

Test Precision

Test precision is defined here in the limited sense of "apparent nearness to truth." This is in contrast to test accuracy, "absolute nearness to truth," which is the subject of the criterion discussed immediately above. We wish the test to be precise in the sense that the same results would be obtained for any soil, regardless of where or when the test is conducted, or who conducts it. This objective can be approached by identifying critical areas of test design and specifying exact procedures, or better, automating them. Further, if the data reduction can be devised so as to make any experimental scatter obvious, the test precision will be a continuous subject of concern to operators.

Whole Soil

The contract specified that the test method avoid the removal of any size fraction. Also, the test method should not produce significant changes in soil fabric such as might result from drying to near zero moisture content or extremely shear intensive mixing. The effort must be to maintain conditions at least approximately as might be expected in construction.

Cost

No specific limitations were set on test cost, yet acceptance of the test method by the geotechnical community requires that the cost per test over the life of the apparatus be within reason. Capital costs for the hardware should be in the range of several hundred to several thousand dollars, recognizing that high capital cost can be offset by low operating cost only so long as the capital cost is not prohibitive.

Operator Skill

It is desirable that the test not require a high level of operator skill or art to provide consistent results. This objective, which itself contributes to good test precision and low cost, can be achieved by careful preparation of procedures, shrewd design of apparatus or by automation.

A final consideration was based on the wise use of the Atterberg limits procedure. This program was not intended to produce a better plasticity index test, yet our large backlog of Atterberg limit data and experience should not be discarded. It was decided that a correlation between soil plasticity and the test developed under this program should be looked for. It was anticipated that a good correlation would exist, for the Atterberg limits have proven value in predicting soil behavior. If a good correlation were found, the options available for implementation of the new consistency test would then include:

* direct re-write of existing classification schemes, design methods and acceptance criteria that currently make use of the Atterberg limits

as well as

* construction of new classification schemes and design methods based solely on the fact that the new test should quantify the behavior of soil as a function of moisture content.

Throughout the program, it was emphasized that the purpose was to develop a soil classification test that would describe the response of soil to moisture. The test must be appropriate to use by the various elements of the geotechnical community if the program is to be worthwhile.

1.2 Summary

Execution of the subject program required the evaluation of a number of candidate test methods which displayed some potential for development as soil consistency tests. To organize the effort, candidate test methods were categorized as being either a basic parameter or mechanical response test. The category "basic parameter test" included those tests directed toward the detailed behavior of the soil-water system. Included were such methods as zeta-potential, the various tests describing the moisture-tension response of the soil, electrical dispersion and others. These tests can all be considered as describing the behavior, or the state, of water on or near the surface of a soil grain. The other category, "mechanical response tests", consisted of those tests that ignore the detailed behavior of pore water, but direct themselves to the gross mechanical behavior of soil at different moisture contents. Tests for such properties as strength, stiffness, and the Atterberg limits fall in this category. Following a preliminary review of a number of tests from both categories, it was determined that the effort should be concentrated on the mechanical response tests. In many cases, basic parameter tests show considerable potential, but must still be considered as research tools. They need somewhat more development than can be accomplished in a single project before being recommended as production tests.

Of the mechanical response tests, a number are viable candidates for soil consistency tests. Some were not well suited for use in a field lab, while others would not work with soil containing gravel. The final candidates were a plate bearing test and a K_o test. The latter was ultimately selected for development.

Test selection and validation were based on tests conducted on a wide range of soil types. These included natural and processed soils. While the program emphasis was on "real world" natural soils, it was recognized that certain processed soils such as Ottawa Sand consititute laboratory standards, and should be included. In all, 13 soils were represented. Future work should be considered to increase this number.

2.0 PROGRAM DESIGN

The effort to develop the required soil consistency test required that a wide range of candidate technologies be considered, that promising methods be evaluated in the laboratory and that a final candidate test method be extensively tested, using a range of natural soils. To organize the test methods identified in a review of literature, they were grouped as being either basic parameter or mechanical response tests. The basic parameter group included tests that were thought to measure the behavior of water in soil in detail. In general, it was expected that these tests might make use of research equipment not found in a soils lab, and would borrow from disciplines other than Civil Engineering. The intention of these tests is to describe some parameter controlling water behavior. The mechanical response tests are those that ignore the inner workings of the soil-water system, concentrating instead on its gross behavior. It was anticipated that these tests would be familar to civil engineers, and would build on established soil mechanics technology.

2.1 Basic Property Tests

2.1.1 Moisture Tension

The moisture tension test is a technique for measuring capillary pressure as a function of soil moisture content. This is a basic parameter test used by civil engineers and there is an established test procedure(ASTM D2325). Investigators who have used this method to study liquid or plastic limits include Russell and Mickle (1970), Uppal (1966), Livneh, et al (1970) Croney, et al (1952, 1954), and Coleman and Marsh (1961). The test is most attractive in concept, in that it should give, in a single parameter, considerable information about the behavior of moisture in soil. Several of the investigators cited above argued that the test should replace one or both of the Atterberg limit tests. It was not accepted for intensive evaluation in this program because of the large sample volume required in a soil that contains two inch rock. The test requires approximately 24 hours to establish equilibrium moisture in a 0.5 inch (1 cm) soil pat. This time would increase with the square of sample thickness, becoming several weeks or more in a 4 inch (10 cm) thick specimen of low permeability soil. Such delays were considered unacceptable. The sensitivity of the test to pore fabric would be a minor drawback, making it rather sensitive to variations in compaction. It should be noted that many investigators in soil mechanics and soil science find the test effective and useful for fine grained soils not containing large aggregate.

2.1.2 Electrical Dispersion

An electrical dispersion test measures the electrical conductivity of soil as a function of impressed frequency from DC up to, in some reports, VHF radio frequencies. An enormous amount of information can be obtained about the soil and its pore fluid in this test. In particular, it appears that the test should distinguish between free and bound water in the soil. Investigators who have used some form of electrical dispersion or conductivity include Mitchell and Arulanandan (1968), Arulanandan and Smith (1973), Selig and Mansukhani (1975), Long and Zimmie (1973) and Ramiah and Purushothamaraj (1971). This test was dropped from consideration because of the many factors that affect (and can be measured by) electrical dispersion and because other methods requiring much less complex electronics packages are available.

2.1.3 Zeta Potential

Zeta potential is a measure of the energy required to move a water molecule from infinity to the proximity of a soil particle. The test is usually conducted in a low concentration slurry, but newer methods are being developed that can be used at moisture contents sufficiently low to be referred to as pastes. Investigators at IITRI who routinely do zeta potential work considered it attractive, yet rather operator sensitive for the purposes of this project.

2.2 Mechanical Response Tests

It can be argued that the soil property of greatest importance to geotechnical engineers is soil shear behavior. Shear strength can be measured directly by simple or direct shear tests and triaxial tests or by cone penetration, vane shear, California Bearing Ratio (CBR), plate bearing, or indeed, by the Atterberg limit tests.

2.2.1 Cone Penetrometer

A large volume of literature has been published on various cone penetration tests, many of which have been set forth as Atterberg limit replacements. This literature is so extensive as to prohibit citation here; instead a special section of the references is set aside. Certain of this is summarized in Table 1. The cone penetrometer is competitive in cost, speed, precision, and accuracy with the Atterberg limit tests. However, when used with soils containing large aggregate, the test must be conducted without hitting a rock. A number of tests were, in fact, conducted using a 0.2 in. sq. (3.3 cm. sq.) base area, 30 deg. included-angle cone. To avoid shear rate effects associated with the various fall cones, a constant penetration rate with a load cell to measure penetration resistance was used. The test proved rapid and accurate. The data were quite similar to the plate bearing data which will be discussed at length in Section 3. The test was eventually discarded because the plate bearing and K_o tests also provided good results while including larger volumes of specimen within the zones affecting the test.

Table 1. CONE-PENETROMETERS IN USE FOR DETERMINING THE LIQUID LIMIT OF SOILS

Country of Origin	References	Angle of Cone, Degree	Weight of Cone, g	Definition of Liquid Limit
Sweden	Karlsson (1961) Hansbo (1957)	60	60	Related to the fineness number which is the moisture content corresponding to cone penetration of 10 mm
U.S.S.R. Bulgaria Yugoslavia East Germany	Vasilev (1949) Stefanov (1958) Bozinovic (1958) Matschak and Rietschel (1965)	30	76	Moisture content corresponding to cone penetration of 10 mm
India	Uppal and Aggarwal	31	148	Moisture content corresponding to cone penetration of 25 mm
U.S.A.	Sowers, et al. (1960)	30	75	Moisture content corresponding to cone penetration of 10 mm
France	Laboratoire Central des Ponts et Chaussees	30	80	Apparatus calibrated against Casagrande apparatus

2.2.2 Vane Shear

Vane shear consistency tests were proposed by Darienzo and Vey (1955) and Youssef, et al. (1965). Preliminary evaluations were conducted using a 1/2 in. dia. by 1/2 in. long (13 x 13 mm) constant strain rate laboratory vane. This test can be conducted near the liquid limit, but insertion of the vane into compacted specimens much dryer can cause fracturing and, in some cases, severe disturbance. If gravel is present in the soil, a relatively large vane, having dimensions comparable with field vane shear devices, would be required, together with a correspondingly large compacted specimen. Because the plate bearing and K_o tests were developing well and did not display these disadvantages, the vane shear device was dropped from consideration.

2.2.3 Other Tests

A number of tests based on extrusion, pressing, or rolling have been advanced as consistency tests. In general, these are inappropriate for soils containing coarse aggregate.

2.3 Selection of Final Candidate Test Methods

The selection of a candidate test method for final extensive evaluation was based upon the factors of test significance, precision, ability to handle soil with large aggregate, test cost (both capital and operation), and required level of operator skill.

Following considerable discussion among the project staff and with FHWA and other outside experts, the basic parameter tests were eliminated because:

a) Tests could not be shown to relate directly to an important soil parameter such as strength or damping factor, save by correlation. In some cases, many factors other than soil consistency were found to affect these tests.

b) Some of the tests could not be run on soils that were within the moisture regime we refer to as "plastic."

c) Some require the establishment of additional art before they are fully ready to leave the research laboratory.

Several of these tests, however, have great potential and are deserving of continuing effort.

The mechanical response tests typically are well developed and relate directly to soil shear response. They can be used on soils that are compacted and subjected to changes in moisture or mixed with stabilization agents. After some consideration, the decision was made to evaluate the plate bearing and K_o tests. These tests worked best with soil containing large aggregate, were easy to automate, and measured soil parameters of great significance - strength and stiffness. Ultimately, the K_o test was selected over the plate bearing test. Section 3 treats these test methods in detail.

2.4 Standard Soils Tests

A number of standard soils tests were run for the purpose of characterizing the soils used in this project. In addition, several of the strength tests were used for correlation with the results of the K_o tests as reported later in Section 4.0. The standard tests were run according to AASHTO specifications for testing soils when appropriate. The CBR, unconfined compression and triaxial tests were automated. The standard tests are listed in Table 2 and the results of these tests are shown in Table 3.

Table 2. STANDARD SOIL TESTS

Atterberg Limits
    plastic limit
    liquid limit

Mechanical Analysis
    sieve analysis
    hydrometer analysis

Moisture-Density relationship

CBR (California Bearing Ratio)

Permeability

Unconfined Compressive Strength

Triaxial Strength

Table 3 Results of Standard Soil Property Tests

SOIL NAME	AASHTO Class. and Group Index		Attenberg Limits		Sieve Analysis Percent Passing			Hydrometer Analysis Percent			Optimum Moisture and Density		California Bearing Ratio	Unconfined Strength	Triaxial Strength		Permeability
			PI	LL	10	40	200	0.02	.002	.001	%	pcf		psi	c psi	o deg	cm/sec
Batavia Sand and Gravel	A-1-b	0	NP		80(1)	43	4	-	-	-	-	128	1.1	-	0	36	1.5×10^{-3}
Sand Silt and Clay (No. 2)	A-4	0	8	24	100	81	37	42	13	9	12.6	117	28	46	13	34	9.4×10^{-9}
Batavia Sand and Silt	A-2-4	0	NP		100	92	20	-	-	-	11.5	115	10.7	-	4	36	2.6×10^{-5}
Batavia Sand	A-3	0	NP		100	99	5	-	-	-	8.6	99	0.9	-	5	35	5.3×10^{-3}
Ottawa Sand	A-1-a	0	NP		100	20	1	-	-	-	-	111	0.6	-	0	36	1.6×10^{-2}
Roselle Gravelly Clay	A-6	12	19	37	97	88	72	68	31	21	13.9	115	8	45	13	29	1.6×10^{-8}
Silty Gravel (Soil No. 6)	A-2-4	0	10	28	68(2)	53	33	26	8	7	11.3	123	12	37	11	32	1.3×10^{-7}
Batavia Silt	A-4	0	NP		100	100	47	12	3	1	13.8	108	22	-	0	39	1.2×10^{-5}
Clayey Silt (Velvacast)	A-5	12	8	42	100	100	100	100	35	21	25.2	93	8	38	17	27	6×10^{-7}
Batavia Clay	A-6	5	16	32	100	96	53	42	16	13	15.8	111	23	47	16	21	1.2×10^{-6}
Edgar Plastic Kaolin (EPK)	A-7-5	37	30	62	100	100	100	96	61	47	28.0	88	15	35	12	21	3.5×10^{-8}
Panther Creek Bentonite	A-7-5	77	62	102	100	100	100	61	15	9	44.7	74	33	44	20	8	1.2×10^{-9}
Buckshot Clay	A-7-6	50	44	64	100	100	100	83	54	47	26.2	94	12	45	19	12	3.7×10^{-7}

(1) 100% passing 1-1/2", 98% passing 1", 91% passing 1/2"
(2) 100% passing 3/8", 77% passing #4
(3) 1 psi = 6.9×10^3 P_a
(4) 1 pcf = 16 K_g/m^3

3.0 FINAL CANDIDATE TEST METHODS

The test methods selected for final evaluation were the plate bearing and the earth-pressure-at-rest (K_o) tests. These tests both are capable of testing mechanical response of soils containing aggregate at high moisture contents. Both are scale independent. Should the need arise, tests of any size can be conducted. Both employ soil specimens compacted under standard conditions to a state similar to field compaction. They may be conducted using soils that have been treated, as with lime or cement, or subjected to changes in moisture after compaction. The K_o test acts on the entire specimen volume, the plate test on a smaller region near the soil surface. The literature review did not disclose any previous attempt to evaluate plate or K_o tests across a range of moisture contents: the application proposed in this report may be unique. Finally, the state of the art is such that either test method can be successfully developed within the constraints of the project. Widespread acceptance as a classification test will require additional testing of many soil types prior to ultimate acceptance.

3.1 Plate Bearing Tests

An automated plate bearing test was set up to determine the variation in the penetration resistance of cohesive soils at moisture contents ranging from the dry side of optimum to the point at which the soil took on the character of a viscous liquid.

The test apparatus (Figure 1) consisted of a load machine with a 2,000 pound load cell attached to the loading head, a 1.95 inches (49.5 mm) diameter bearing plate mounted on the load cell, and a DCDT (direct current displacement transducer) to measure plate deflection.

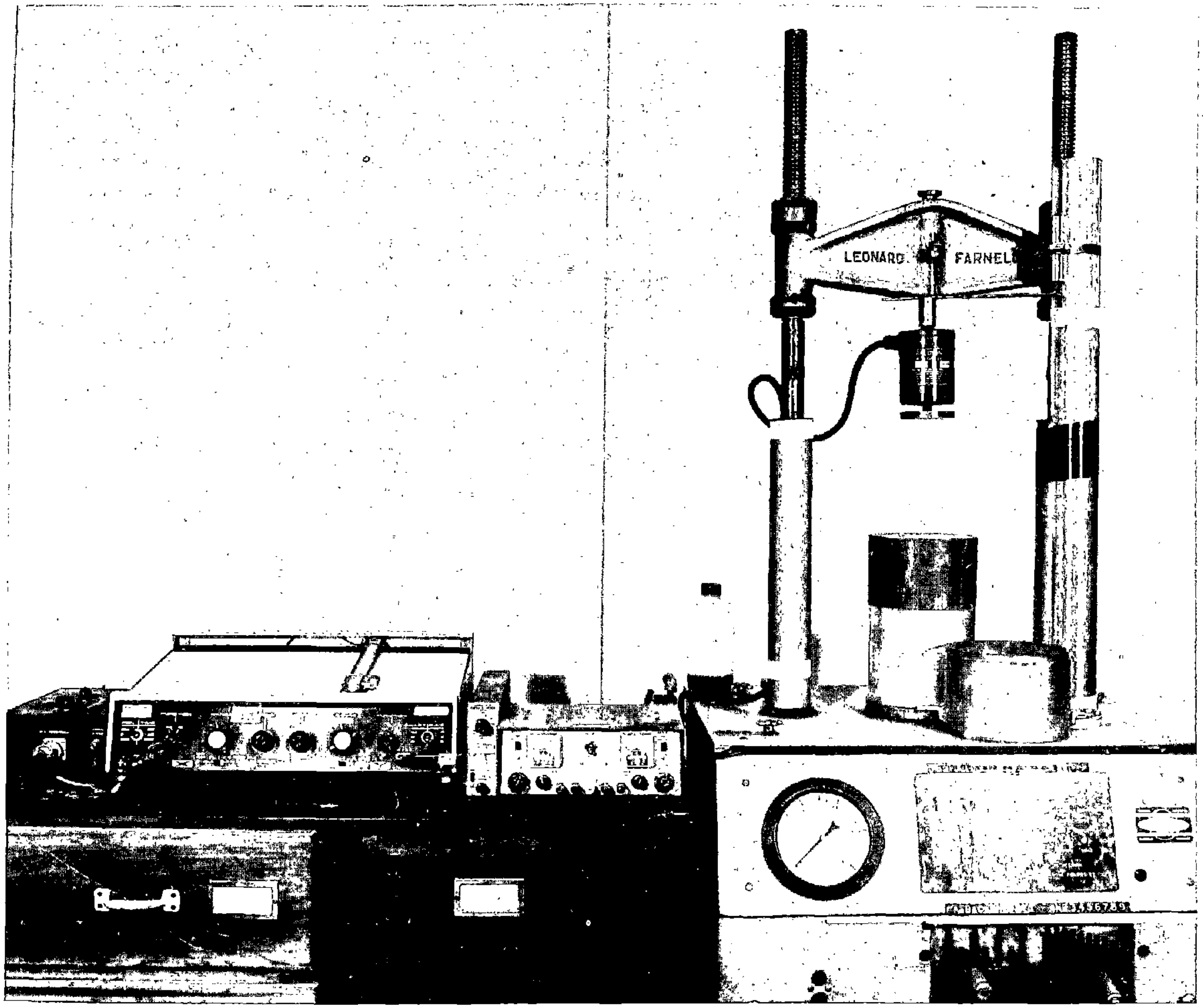


Figure 1. Automated Plate Bearing Test Hardware

The load cell and the DCDT were driven by a 6 volt D.C. power supply. Signals from the load cell and the DCDT were monitored by an X-Y recorder.

Cohesive fine grained samples were compacted in 6 inch (154.4 mm) molds in accordance with AASHTO standard T99-74 for moisture-density relations of soils. Each of the compacted molds of soil were subjected to a plate penetration test at the top and bottom using the apparatus described previously. On the wet side of optimum, compaction was carried out at moisture contents up to the point at which the soil behaved as a viscous liquid and further increases in moisture content were not feasible. Data for the standard compaction curve were also obtained. Densification of cohesionless soils, for which the standard compaction tests did not give a well defined moisture-density relationship curve was achieved by the vibratory method, and load-penetration obtained was as a function of relative density.

A modified hyperbola proposed by Kondner and Krizek (1962) was fitted to the data obtained from the plate load penetration curve to reduce experimental variability. The relationship proposed by the cited authors was:

$$\frac{F}{Aq} = \frac{x/c}{a+b(x/c)}$$

which was modified for this project to:

$$\frac{x}{F} = \frac{ca}{Aq} + \frac{bx}{Aq}$$

where F = plate load

A = plate area

c = circumference of plate

x = deflection

q = unconfined compressive strength

a and b = soil parameters

The soil parameter a is the intercept of a plot of (Aqx/Fc) versus x/c and parameter b is the slope of a plot of (Aqx/Fc) versus x/c: hence 1/a is a measure of the tangent modulus of plate load-sinkage and 1/b is a measure of ultimate plate load (Crenshaw, et al. 1971). The above equations were solved for (ca/Aq) and (b/Aq).

A typical plate-load-deflection curve for Edgar Plastic Kaolin (EPK), as plotted on the X-Y recorder, is shown in Figure 2. Table 4 gives a summary of the load and penetration values obtained from the curves for EPK at various moisture contents. Values of the initial slope and the ultimate plate load are also tabulated for various moisture contents. Figure 3 shows a plot of the initial slope and ultimate plate load versus moisture contents for EPK. In the transition region between stiff and fluid behavior both initial modulus and ultimate load decrease uniformly with increasing moisture content. A plot of the initial slope and ultimate plate load versus moisture contents is shown in Figure 4 for five soil types; the liquid and plastic limits for these soils are noted on the plots.

From Figure 4 it can be seen that as the plasticity index (PI) decreases the slopes of ($1/a_o$) and $1/a_1$) become steeper. For soils at the liquid limit, loads fall below 10 pounds (45 Nt), mostly in the 2 to 4 pound (9 to 18 Nt) range. The plastic limit is not well defined in this plot but can be obtained from the slope versus PI plots.

3.2 Earth Pressure at Rest

The coefficient of earth pressure at rest, K_o, is used to describe soil response under uniaxial strain conditions. K_o is frequently stated in terms of Poisson's

Table 3.1. PLATE LOADS AND HYPERBOLIC PARAMETERS FOR SIX EPK TESTS

Volts Output	Deflection (in)	28 April Moisture 34.4% A	28 April 34.4% B	27 Feb. 30.67% A	26 Feb. 29.56% A	26 Feb. 28.83% A	26 April 27.5% A	2/25/76 26.07% A	2/25/76 26.07% B
.1	.014	25	23	79	73	114	225	205	207
.2	.028	43	41	123	127	209	299	360	328
.3	.042	55	56	147	153	262	328	434	407
.4	.056	68	68	163	170	296	350	476	464
.5	.070	77	79	175	186	323	365	504	506
.6	.084	86	88	187	200	344	381	527	538
.7	.098	93	96	194	213	363	396	547	564
.714	.100	94	97	196	214	365	398	552	568
.8	.112	100	103	204	224	378	411	567	585
.9	.126	108	109	212	235	392	426	587	603
1.0	.140	114	115	218	245	406	442	606	618
1.1	.154	121	120	226	254	420	456	626	631
1.2	.168	127	124	233	265	431	471	644	641
1.3	.182	132	128	239	273	443	487	663	651
1.4	.196		132	245		455	501	682	660
1.429	.200		133						662
coefficient of determination r^2		.98490		.99462	.99156	.99744	.98594	.99452	
$1/a_o$ stiffness (lb/in)		1813		6685	6143	10730	17510	19942	
$1/a_1$ load (lb)		209		290	346	566	522	793	

notes: A measured load in pounds (1 lb = .454 Kg)

B load computed from hyperbolic fit in lbs.

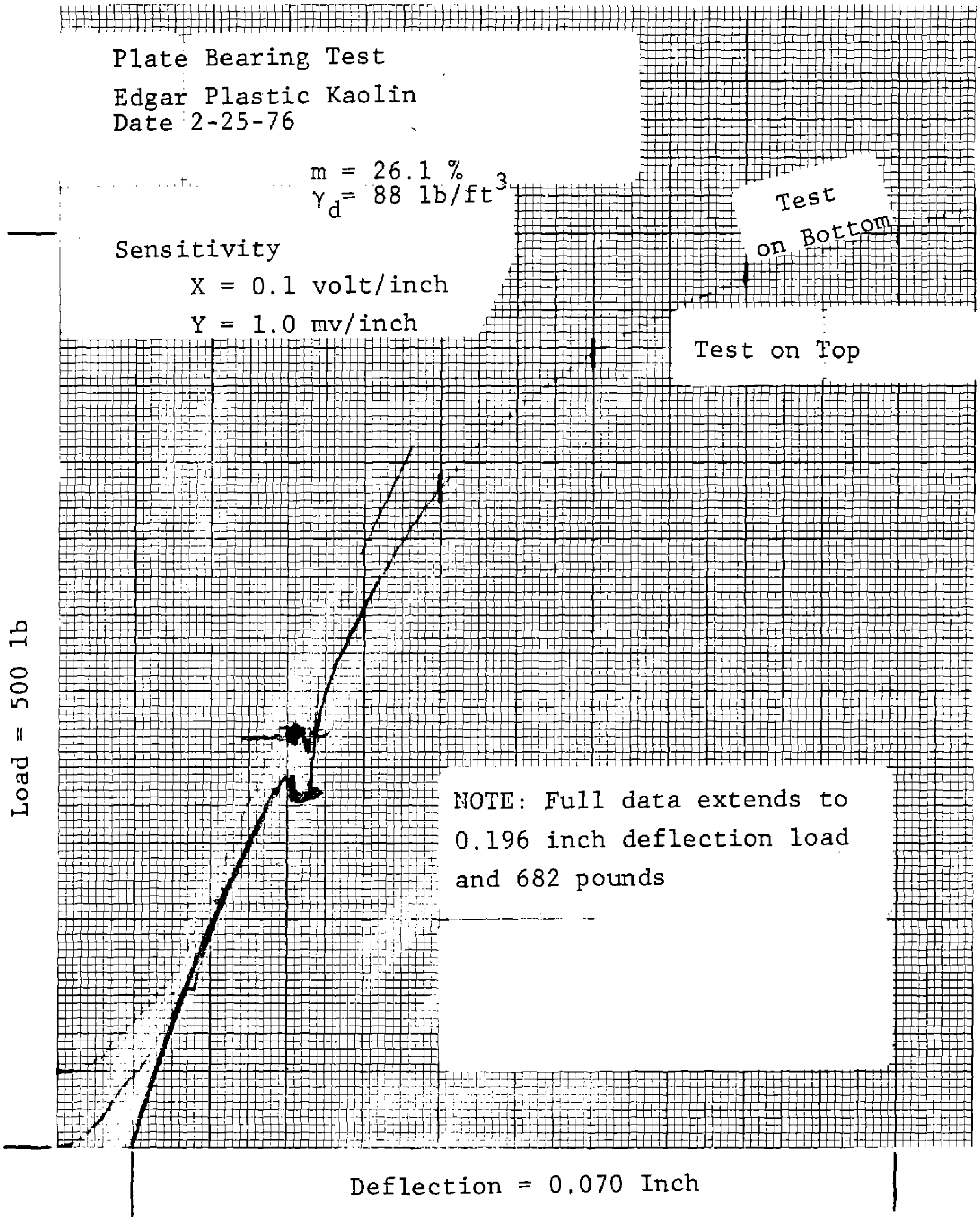


Figure 2. Portion of Recorder Trace for Plate Bearing Test on Edgar Plastic Kaolin

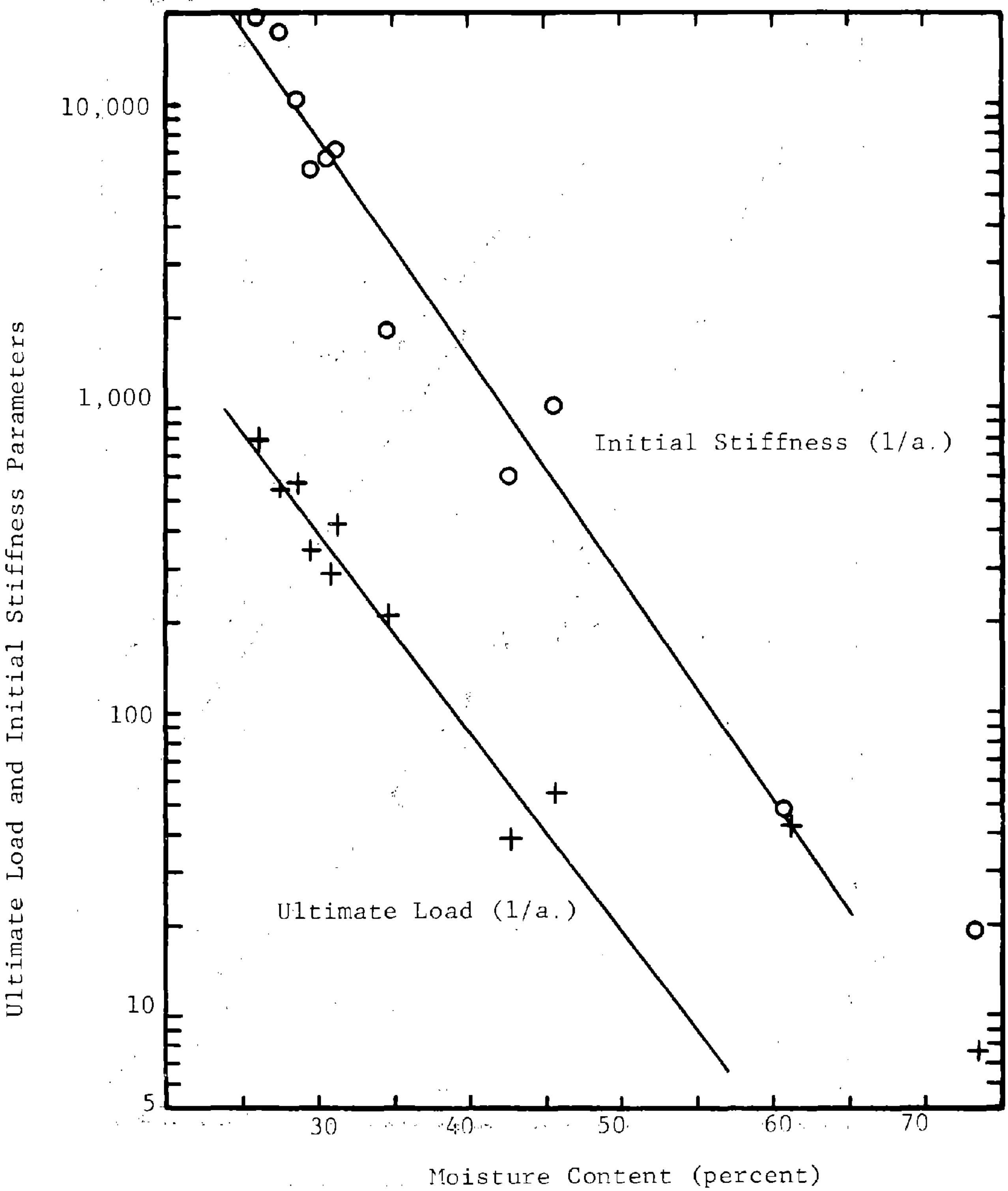


Figure 3. Hyperbolic Plate Bearing Parameters vs Moisture Content for Edgar Plastic Kaolin

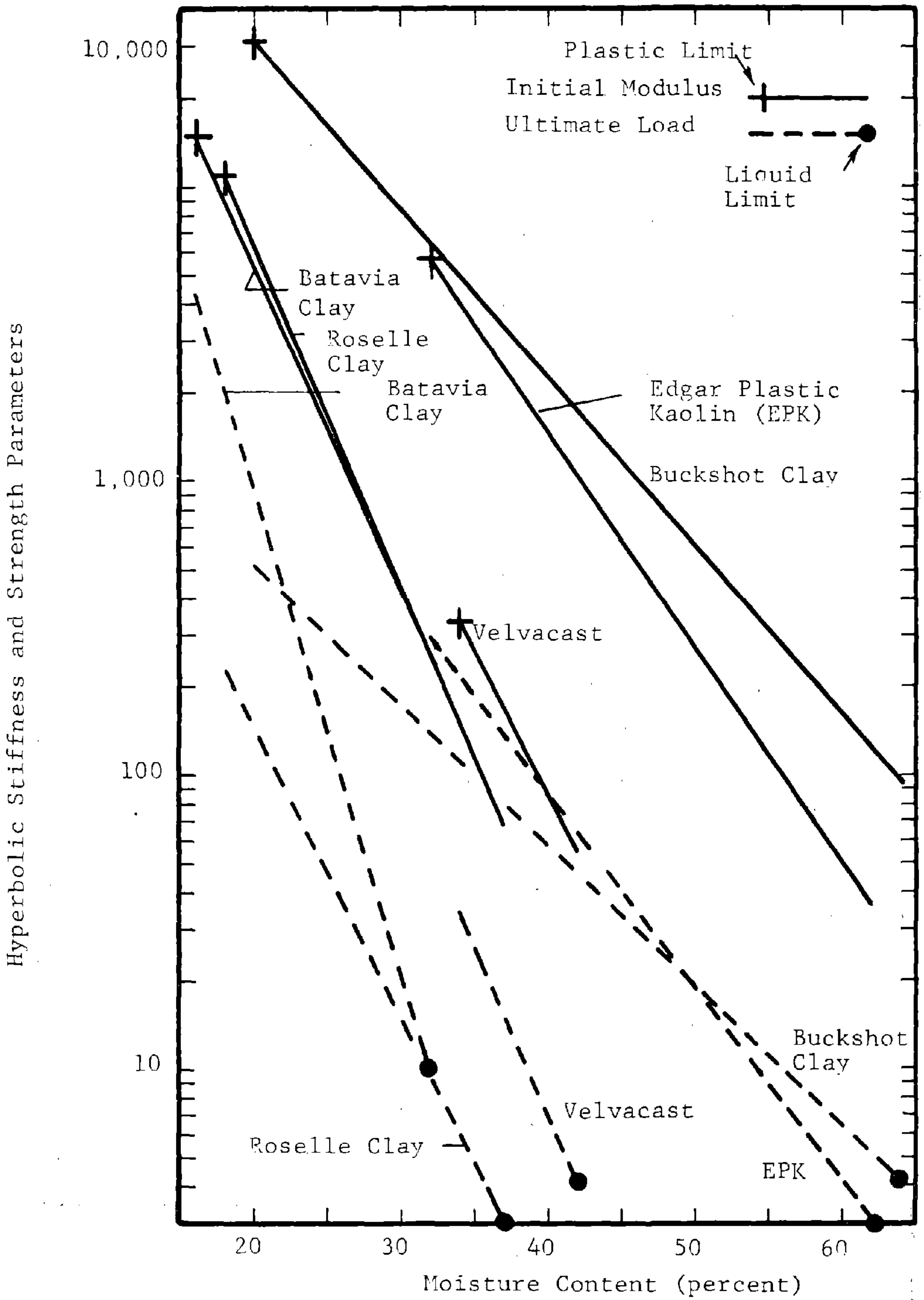


Figure 4. Plate Bearing Hyperbolic Parameter vs Moisture Content

ratio, ν, as

$$K_o = \frac{\nu}{1-\nu}$$

Ignoring the fact that ν itself is formally defined only for uniaxial stress, a parameter that appears to be related solely to soil stiffness is a plausible candidate for measuring consistency.

A more useful definition is

$$K_o = \frac{\sigma_3}{\sigma_1}$$

for conditions of: $\varepsilon_2 = \varepsilon_3 = 0$

$\sigma_2 = \sigma_3$

σ_1, σ_2 and σ_3 are effective principal stresses

$(\tau_{12}, = \tau_{13} = \tau_{23} = 0)$

Some investigators prefer to use a tangent definition $K_o = d\sigma_3/d\sigma_1$ as being more general.

A number of formulations are available for K_o in terms of various soil parameters. The most familiar rests on Jaky's expression, $K_o = 1 - \sin\phi$, for discussion of which see Huck, et al. (1974). Formulations for overconsolidated natural soils include:

$K_o = \lambda + a\,(P_r - 1)$ Sherif and Strazer (1973)
in which P_r = overconsolidation ratio
a, λ = empirical constants weakly dependent upon liquid limit

and

$K_o = K_o^* \, P_r^h$ Schmidt (1973)
in which K_o^* = normally consolidated K_o
h = empirical constant

In addition, Brooker and Ireland (1965) plot K_o as a function of ϕ, plasticity index, and overconsolidation ratio. The data cited above do not indicate a strong relation between consistency and K_o. However, the investigators were not considering K_o as a function of moisture content, but rather as a soil property. As will be discussed later, K_o would be expected to change with moisture content in a fashion that correlates with soil consistency.

3.2.1 Laboratory Tests

Laboratory K_o tests can be categorized by specimen geometry. The requirements that lateral strain be zero and that shear stress be zero are expressed in two distinct classes of experimental hardware.

Special Triaxial Tests

A triaxial cell clearly provides zero shear stress on a specimen surface. If the proper confining pressure is used in a standard triaxial cell, the lateral strain can be controlled at a value near zero to approximate K_o conditions. A number of methods are cited in Table 5. Two depend upon the fact that lateral strain is zero if the axial and volumetric strains are equal. The others employ transducers that measure any non-zero lateral strain directly, so that any deviation from the desired stress ratio can be rectified. A disadvantage of this test geometry is the possibility that the lateral strains may not be everywhere zero. If the specimen itself is not uniform over its entire length, or if pore pressures vary in the specimen, differential lateral stresses may result.

Special K_o Devices

The investigators cited in Table 6 present special K_o devices which enforce the condition of zero lateral strain by the use of rigid confining chambers. The problem of maintaining zero shear stress is approached

Table 5

METHODS TO MONITOR OR CONTROL LATERAL STRAIN IN SPECIAL TRIAXIAL TESTS FOR K_o

	Remarks
Bishop (1958)	apparatus geometry forces equality between volumetric and axial strains, hence lateral strains average to zero over the specimen length
Altschaeffl and Mishu (1970)	non-contacting capacitive transducer for lateral strain
Moore (1971)	circumferential strain gage transducer
Andrawes and El-Sohby (1972, 1973)	use several constant-stress ratio tests, interpolate to stress ratio giving zero lateral strain. Volumetric strain from burette, axial strain from dial gage.
Dudley (1971)	photogrammetry and optical tooling methods to monitor lateral strain.

Table 6

SPECIAL K_o DEVICES HAVING RIGID CONFINING RINGS

	Remarks
Brooker and Ireland (1965)	thin-wall fluid backed confining ring. Foil strain gages and pressure compensation to maintain zero lateral strain. Fluid pressure gives lateral stress.
Obrcian (1969)	heavy wall split confining ring with load cells to measure lateral stress
Sherif and Strazer (1973)	University of Washington Stress-Meter. Heavy wall split confining chamber with load cells to measure lateral stress.

through the use of polished chamber walls and geometry (low specimen aspect ratio) that minimizes the length over which this shear stress can act. As a consequence, this class of device tends to resemble odometer test hardware.

It can be seen that a variety of hardware and methods are available to conduct laboratory K_o tests. All depart somewhat from the ideal conditions of uniformly zero lateral strain and zero shear stress, but the approximation of K_o conditions can be made very good in most cases.

3.2.2 Anticipated K_o Response

For a given soil density and moisture content, K_o will be essentially constant upon initial loading. A perfectly rigid material would have a K_o value of zero, while a fluid will display a K_o value of one, corresponding to a hydrostatic state of stress. Soils at normal field moisture and density conditions are expected to have K_o values of perhaps one-quarter to one-third for sand and slightly higher for clay. With the addition of water and corresponding reduction of dry density, the soil behavior changes from stiff to fluid. The abruptness of this change can be used as an indicator of the soil plasticity. Also, fine grained soils can be dispersed to an indefinite extent with increasing moisture content, whereas coarse grained soils have more definite limits to the maximum moisture content. Thus, a clay will accept sufficient water to display fluid behavior ($K_o = 1$) whereas a sand will always display shear resistance (non-fluid behavior) if effective stresses are non-zero. The general behavior that was anticipated is shown in Figure 5. At low moisture contents, the soil will display the low K_o values that correspond to stiff behavior. At high moisture contents, the soil

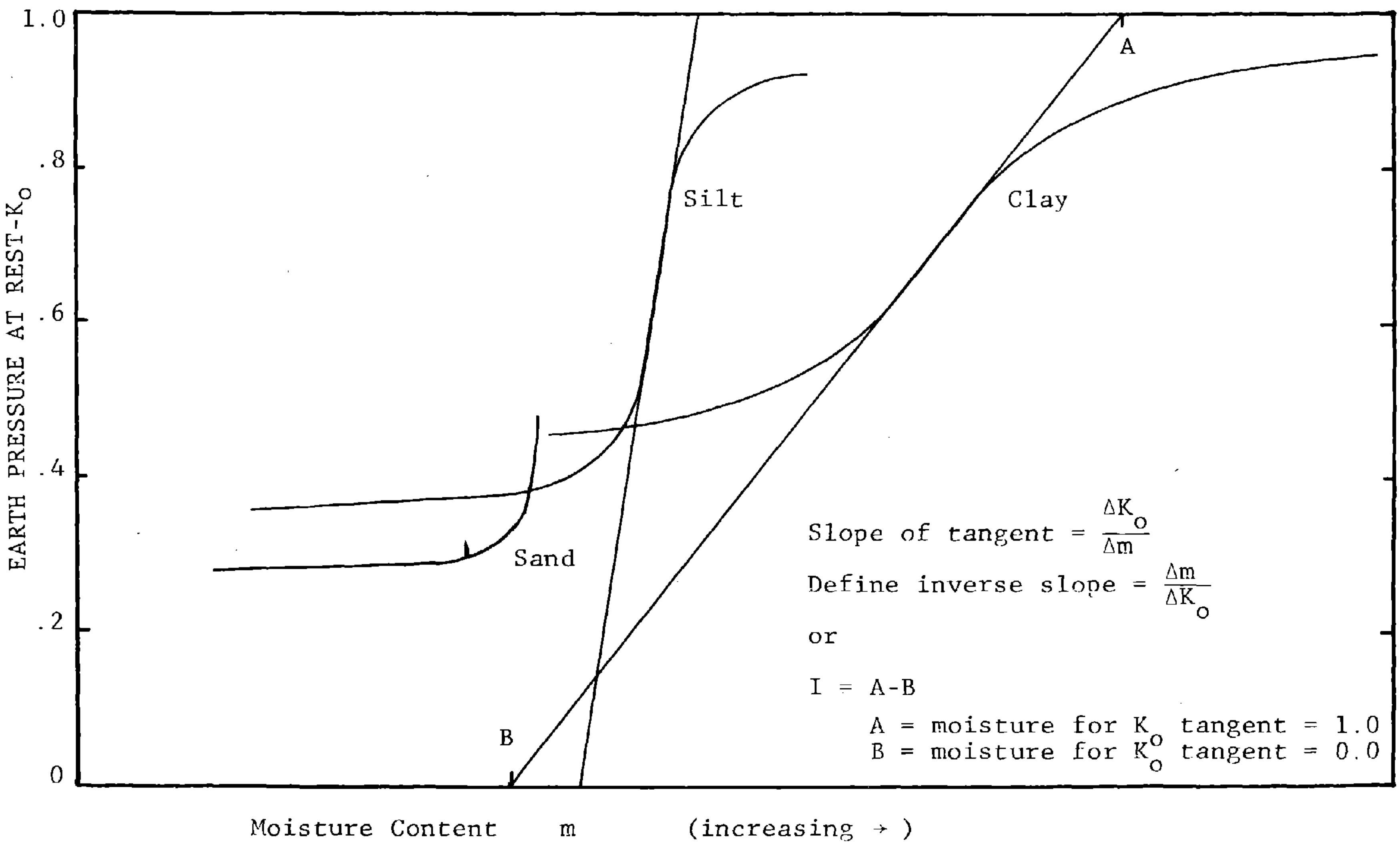


Figure 5. Hypothetical K_o Response of Various Soils to Changing Moisture Content

will display high K_o values, perhaps approaching the perfect fluid $K_o = 1.0$ condition. The transition region between stiff and fluid should display a smooth "S" shaped curve. To obtain a measure of soil consistency, we may strike a tangent to the mid-portion of the K_o -m curve. The steepness of this tangent is a measure of the change in moisture required to take the soil from stiff to fluid behavior. The slope of the tangent is dK_o/dm. To obtain a parameter that increases as the slope flattens, the inverse slope I may be defined as

$$I = \frac{dm}{dk_o}$$

or as

$$I = A-B$$

where A is the moisture content at which the tangent intersects the line $K_o = 1.0$, B is the moisture content at which the tangent intersects the line $K_o = 0.0$.

A large value of I will then indicate that the soil requires a large change in moisture content for a given change in behavior.

We may now consider how different soil types will behave.

A clay soil will change gradually from rigid to fluid behavior, this change occurring in the moisture regime in which we think of the soil as displaying plastic behavior. Above this range, the clay is completely fluid and should approach the value $K_o = 1.0$ asymptotically. For moisture contents much below optimum, K_o would be governed largely by density (strength). Clay soils should display large I values, conversely, a sand has a definite maximum moisture content. Sands will display little if any change in K_o with increasing moisture content until saturation occurs. As the sand nears saturation, K_o would depend upon the relative compressibility of the system components - sand, water and air. The maximum K_o values would be less than 1.0 since the frictional sand always retains some shear strength unless it is placed at a density below the critical void ratio and becomes quick. Using compacted specimens, it is more likely that the soil will become too weak to handle while still at low K_o values. A non-plastic silt would display behavior midway between sand and clay, as a first approximation. It can be taken to a higher moisture content than sand, but being at least somewhat frictional in nature, would not be expected to approach a fluid K_o = 1 condition.

3.2.3 Hardware Considerations

The design of a prototype K_o apparatus to evaluate soil consistency involves departures from the ideal. For this program a rugged apparatus with low life-cycle costs to monitor soil consistency is desired.

To evaluate the effects of sidewall friction prior to the design and fabrication of the K_o cell, a small K_o cell capable of accepting various length specimens was assembled. This unit consisted of a segment of 2.88 in. (73 mm) diameter Shelby tube with strain gages mounted near one end to sense circumferential strain. A solid plug was inserted 0.5 in. (13 mm) below the strain gage level, and a pneumatic loading plug could be inserted to various depths to load different length soil samples. To obtain worst case conditions, the interior of the Shelby tube was not polished, nor was a lubricated membrane used. Tests were conducted on Ottawa sand and Velvacast (clayey silt) as shown in Table 7 below. The sand was placed directly in the K_o cell by pouring. The clayey silt was cored from a 6 in. (150 mm) compaction mold and cutting the core to 1.0, 1.2 and 2.0 inch (25, 31 and 51 mm) length specimens. Some non-uniformity due to layering in the compaction mold may have been present.

Table 4. Effect of Specimen Length on K_o

Soil	Specimen Length inch (mm)	Aspect Ratio L/D	K_o
Ottawa Sand	1.6 (41)	0.56	0.38 ± 0.02
	2.1 (53)	0.73	0.42 ± 0.05
Velvacast	1.0 (25)	0.35	0.99 ± 0.06
	1.2 (30)	0.42	0.76 ± 0.08
	2.0 (51)	0.69	0.84 ± 0.04

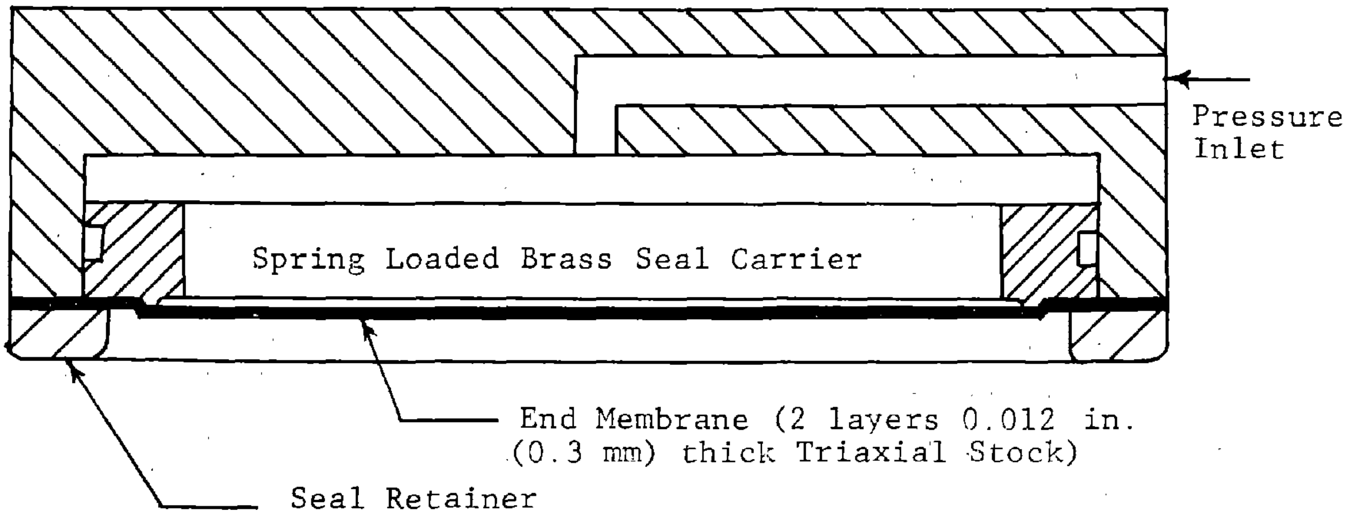
If wall friction is serious, a uniform decrease in K_o with increasing specimen length should appear. If the specimens were many feet long, the soil and the tube walls would act together, with most of the axial load being carried by the relatively stiff tube. Any such trend is obscured by experimental error over the range of aspect ratios involved in these tests. These data indicated that wall friction would not be an intractable problem, and the K_o design could proceed with due care.

Sample size is the first design consideration. The presence of up to two inch rock dictates a sample size larger than the usual K_o hardware will accept. A standard compaction mold is convenient. Also, if standard compaction procedures are followed, the test method will give moisture-density data at no additional cost. The decision was to use a standard 4 inch (10.16 cm) diameter specimen compacted according to T-99. The test could then easily be modified to accept a 6 inch (25.81 cm) specimen if desired.

Based on the selected specimen dimensions, a rigid sidewall or triaxial chamber device may be used. The decision was made to use a rigid chamber to limit the possible effects of layering in the compacted specimen. The apparatus was designed as shown in Figure 6. The specimen is loaded by pneumatic caps at either end, pressure being transmitted by a thin membrane which seals the specimen ends. Loading both ends reduces the effective specimen aspect ratio to 0.56:1. An aspect ratio of 0.2 to 0.3 would be more desirable. Friction is reduced by the use of polished steel sidewalls and a thin lubricated membrane.

Lateral stress is measured by foil strain gages mounted on the outside of the cell at its mid-height. The expansion of the cell is determined by the wall thickness and lateral stress. Fully meeting the desired zero-strain situation would result in a zero amplitude signal. A signal level of 200 microstrains (0.02 percent strain) was selected as being large enough to read with standard circuitry, yet small enough to be considered as approximately zero. For example, a 0.5 percent axial and 0.02 percent radial strain give a strain ratio of 0.04. From Andrawes and El-Sohby data on sand, such an error would change K_o from 0.357 to 0.370, about four percent change. This seems an acceptable price to pay to avoid more elaborate instrumentation or servocontrol feedback systems.

The problem of wall friction is less easily evaluated. The data of Potyondy (1961) indicate that soils on smooth steel fail in friction at about one-half to two thirds the normal soil strength. Work conducted at Purdue (e.g., Brummund and Leonards, 1973) confirms this general magnitude by noting coefficients of friction in the range of one-half. If wall friction is fully mobilized in the K_o cell, these friction values would be unacceptably high.



Upper & Lower Loading Heads Identical

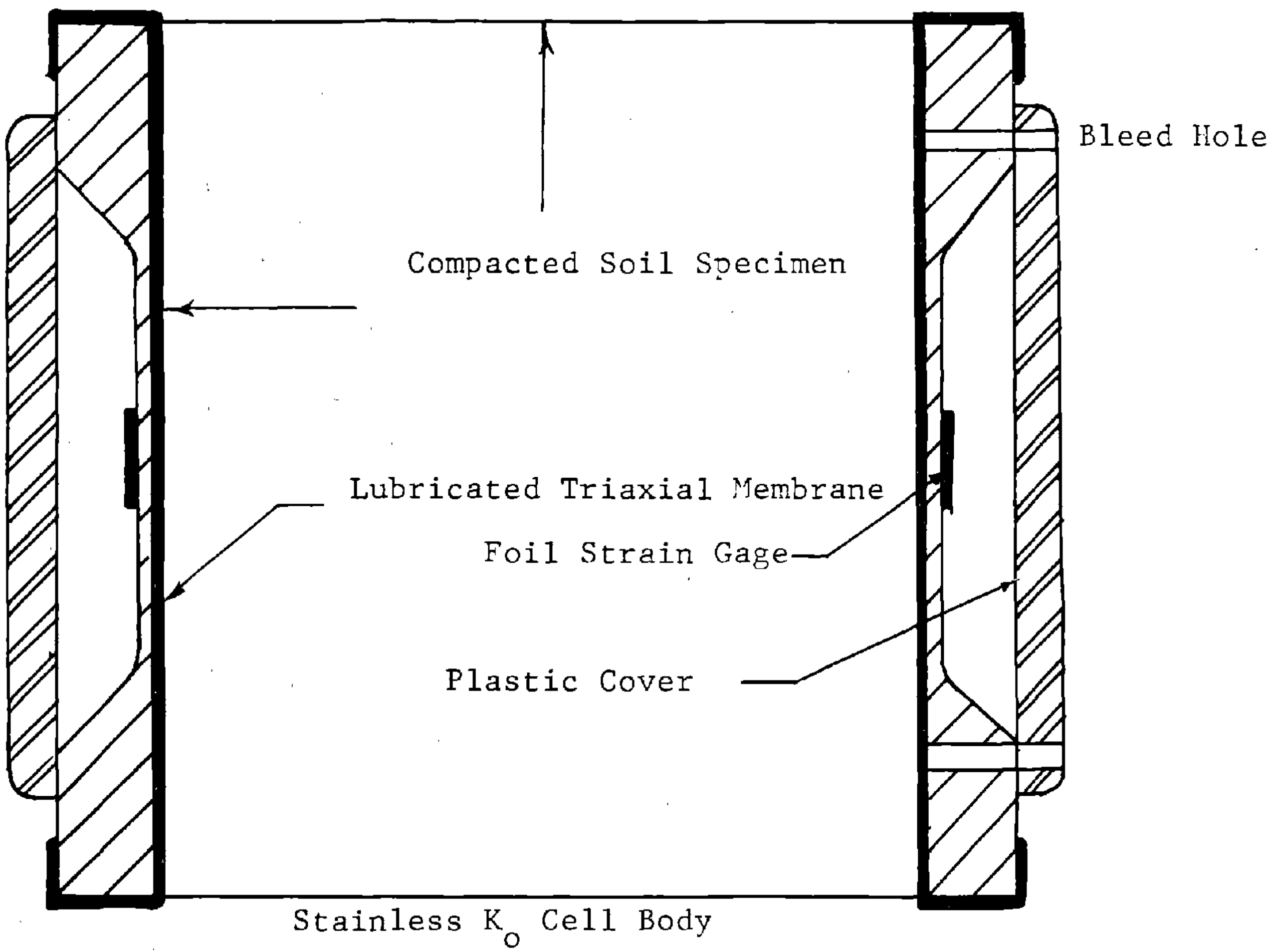


Figure 6. K_o Device - Cell Body and One of Two Loading Heads

This possible problem was avoided by the use of a lubricated membrane. The ultimate test is that the device as designed, did in fact prove capable of measuring soil consistency.

Support equipment includes standard compaction equipment to produce specimens, pneumatic circuitry to load the specimen, and strain gage balancing and recording circuitry. A system block diagram is shown in Figure 7. The unit can be taken into the field, requiring only a compressed gas cylinder and line voltage for the recorder.

3.2.4 Test Procedure and Data Reduction

Detailed test procedures are appended to this report. In brief, a K_o test is conducted in the following steps.

a) Compact a 4" (10.1 cm) specimen at the desired moisture content using T-99 procedures.

b) Remove the specimen from the compaction mold intact. Either a split mold or core extruder is required.

c) Place a triaxial membrane around the specimen and load into the K_o device. Mount end caps, zero instrumentation and mark calibration tics on X-Y recorder.

d) Record five load-unload cycles. For these tests, the specimens were loaded to 60 psi ($4x10^5$ Pa) in 25 to 30 seconds. On each cycle, the vertical and lateral stresses are cross-plotted by the X-Y recorder.

e) Remove specimen and determine moisture content.

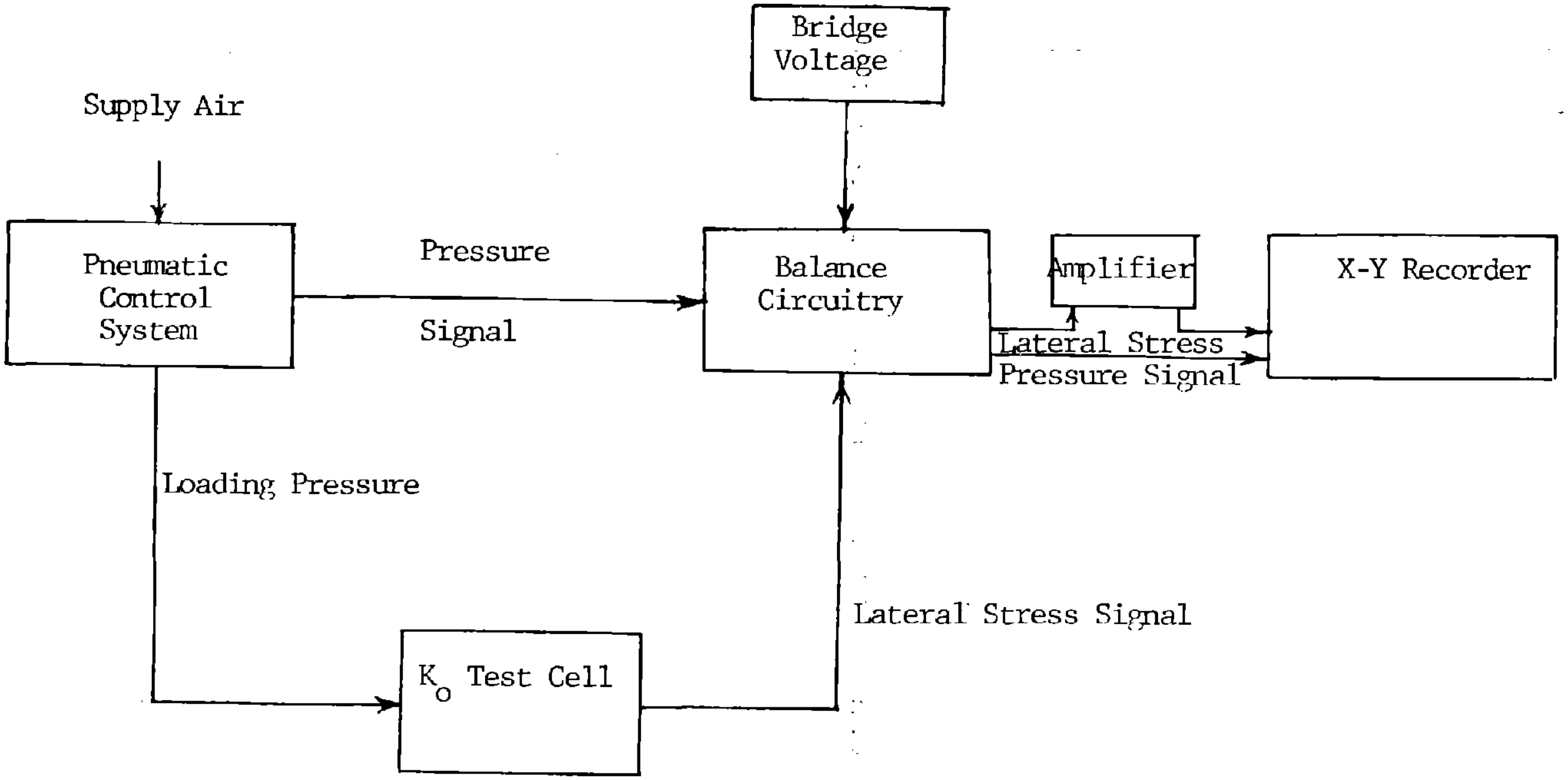
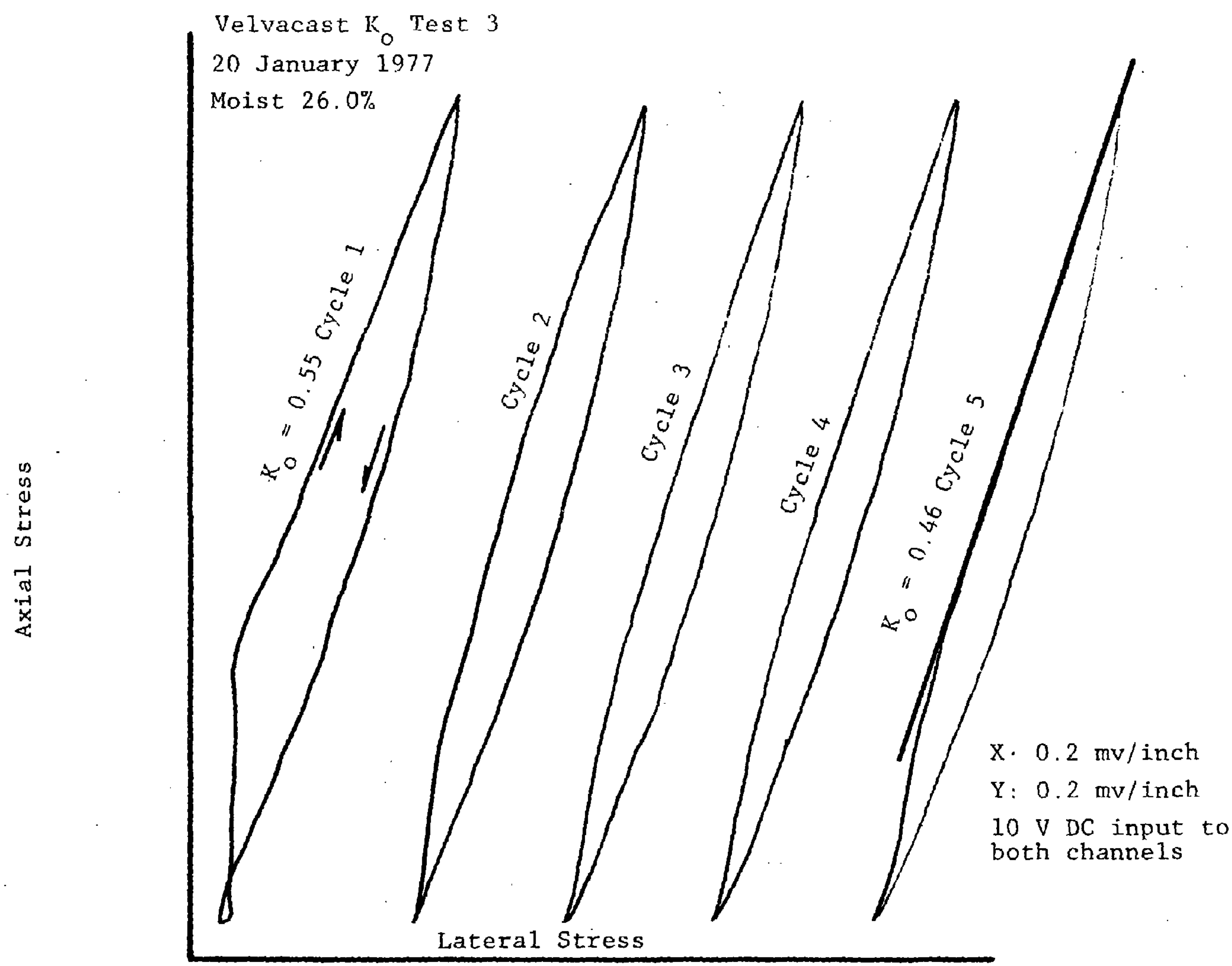


Figure 7. Block Diagram of K_o Test System

Each such test provides the value for K_o at one moisture content, as well as moisture-density data. A series of tests is required to give the curves showing K_o and density as functions of moisture. The raw data from a K_o test is shown in Figure 8. When the specimen is initially placed in the K_o cell, it has a clearance of about 0.020 in. (0.5 mm) on the radius. The specimen is expanded to close this gap on the first cycle. The first several cycles may be irregular for this reason. The specimen behaves consistently by the third or fourth cycle, and changes little on subsequent cycles. The fifth cycle was chosen as a desirable point to determine K_o. The soil in Figure 8 is near optimum moisture content. As the moisture content increases, the hysteresis decreases. At K_o values of 0.9 or higher, the soil has little shear strength, and the hysteresis loop has closed very nearly to a straight line.


Velvacast K_o Test 3
20 January 1977
Moist 26.0%
K_o = 0.55 Cycle 1
Cycle 2
Cycle 3
Cycle 4
K_o = 0.46 Cycle 5
X· 0.2 mv/inch
Y: 0.2 mv/inch
10 V DC input to both channels
Axial Stress
Lateral Stress

4.0 SOIL CONSISTENCY TEST RESULTS

The apparatus described above was used to generate curves of K_o vs moisture content for eight soils with plastic index values between 8.0 and 62.5. An additional five soils were tested and shown to be non-plastic. A soil is defined as non-plastic in terms of its K_o-moisture content behavior.

4.1 Plastic Soils

Results of K_o tests on plastic soils are shown in Figures 9 to 16. Plastic soils are defined as those that display at least stiff and transition states. Clays display a fluid state in addition, whereas silts will not disperse to a fluid state. As previously discussed, a given soil will display an "s" shaped curve. A tangent is drawn to the experimental curve in the transition range, the inverse slope of which is taken as a measure of soil consistency. The inverse slope is defined in Section 3.2.2 and is shown on Figure 9. A low numerical value indicates an abrupt change from stiff to fluid as moisture content increases, while a high value, as in Figures 14 to 16, indicates a more gradual change.

Figure 9 displays the curve quite fully defined. Usually three or four data points are sufficient to define the transition region of the curve. This low plasticity clayey silt changes from stiff to fluid within a 3 percent change in moisture immediately above optimum moisture. If such material is used in earthwork, moisture control will be critical. This may be compared to the silty gravel (Soil No. 6 in Figure 11. This material may confidently be compacted at T-99 moisture or several percent above. Further, the results of compaction and subsequent saturation would be less severe than with the clayey silt material.

If the clayey silt in Figure 9 is compacted at optimum moisture content, it might at some later time become

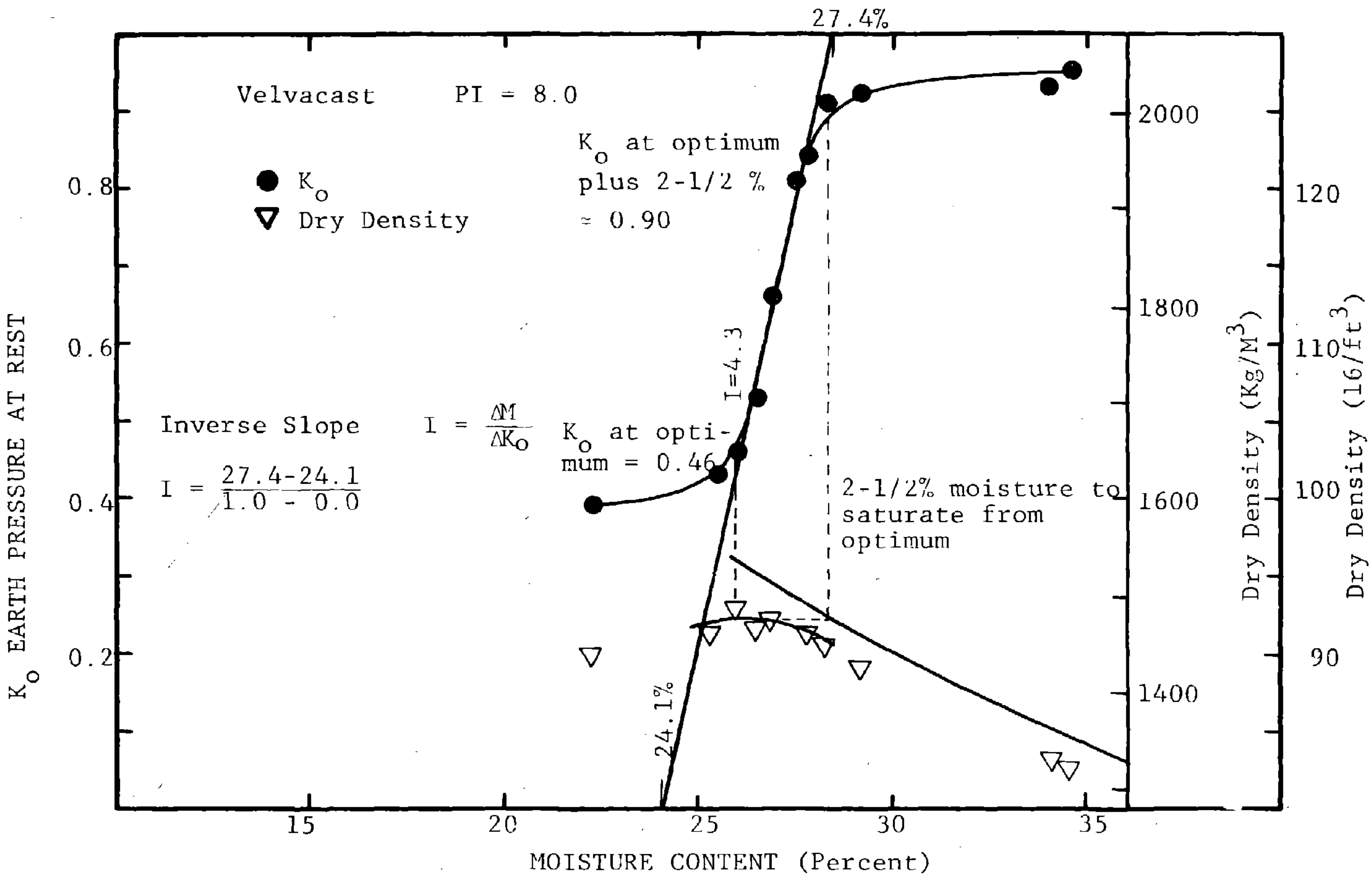


Figure 9. K_o Vs. Moisture Content - Velvacast

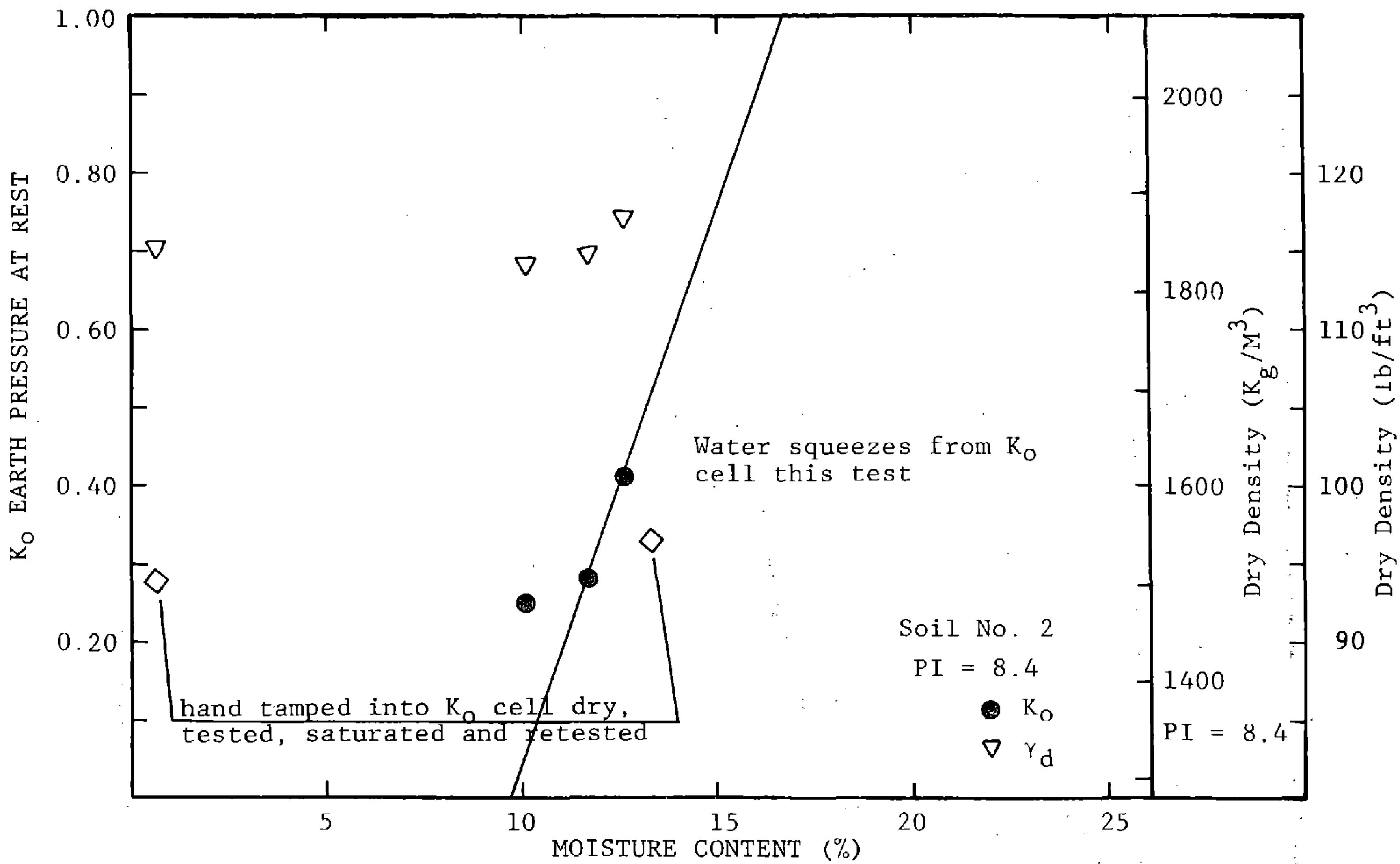


Figure 10. K_o Vs. Moisture Content - Soil No. 2

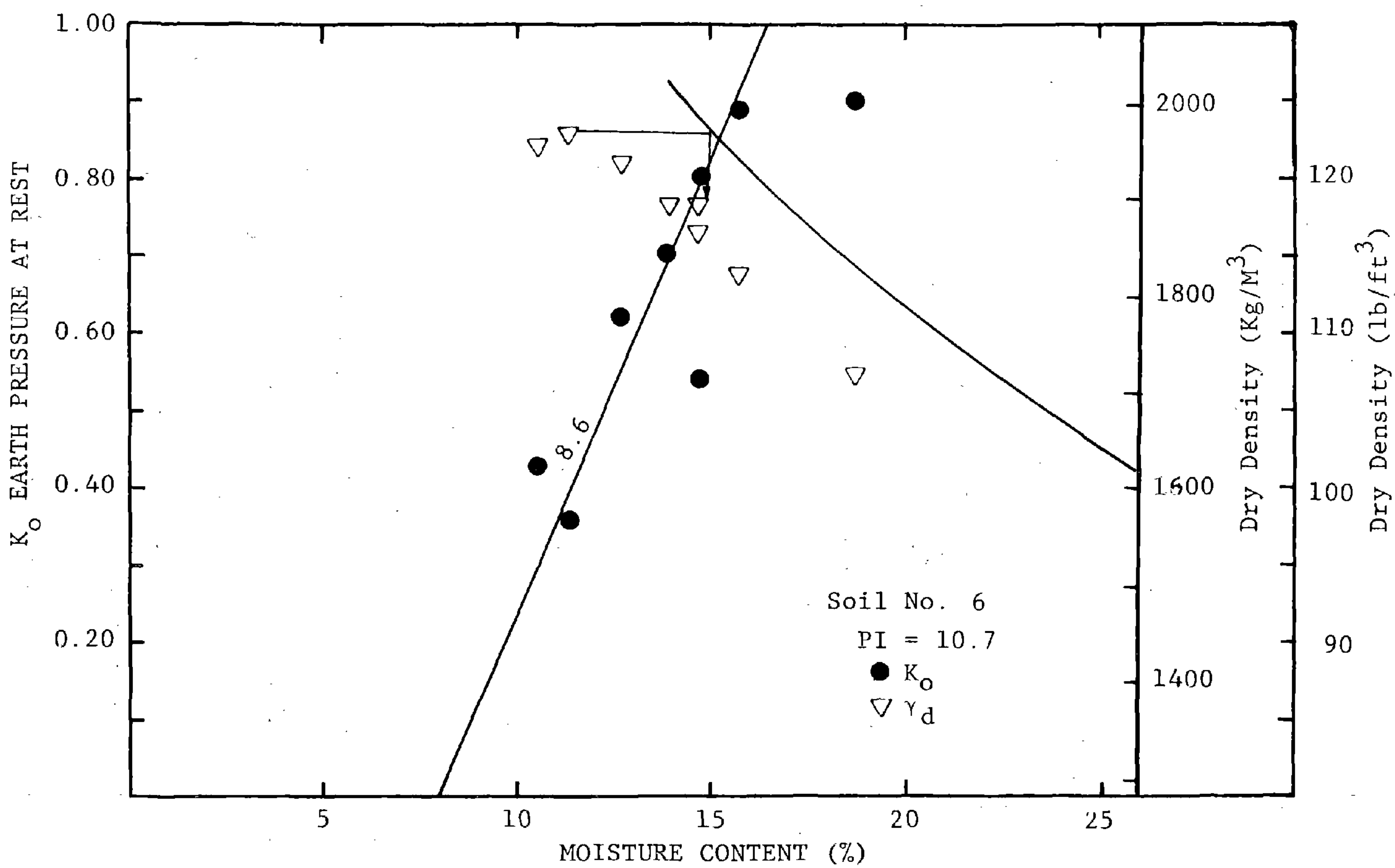


Figure 11. K_o Vs. Moisture Content - Soil No. 6

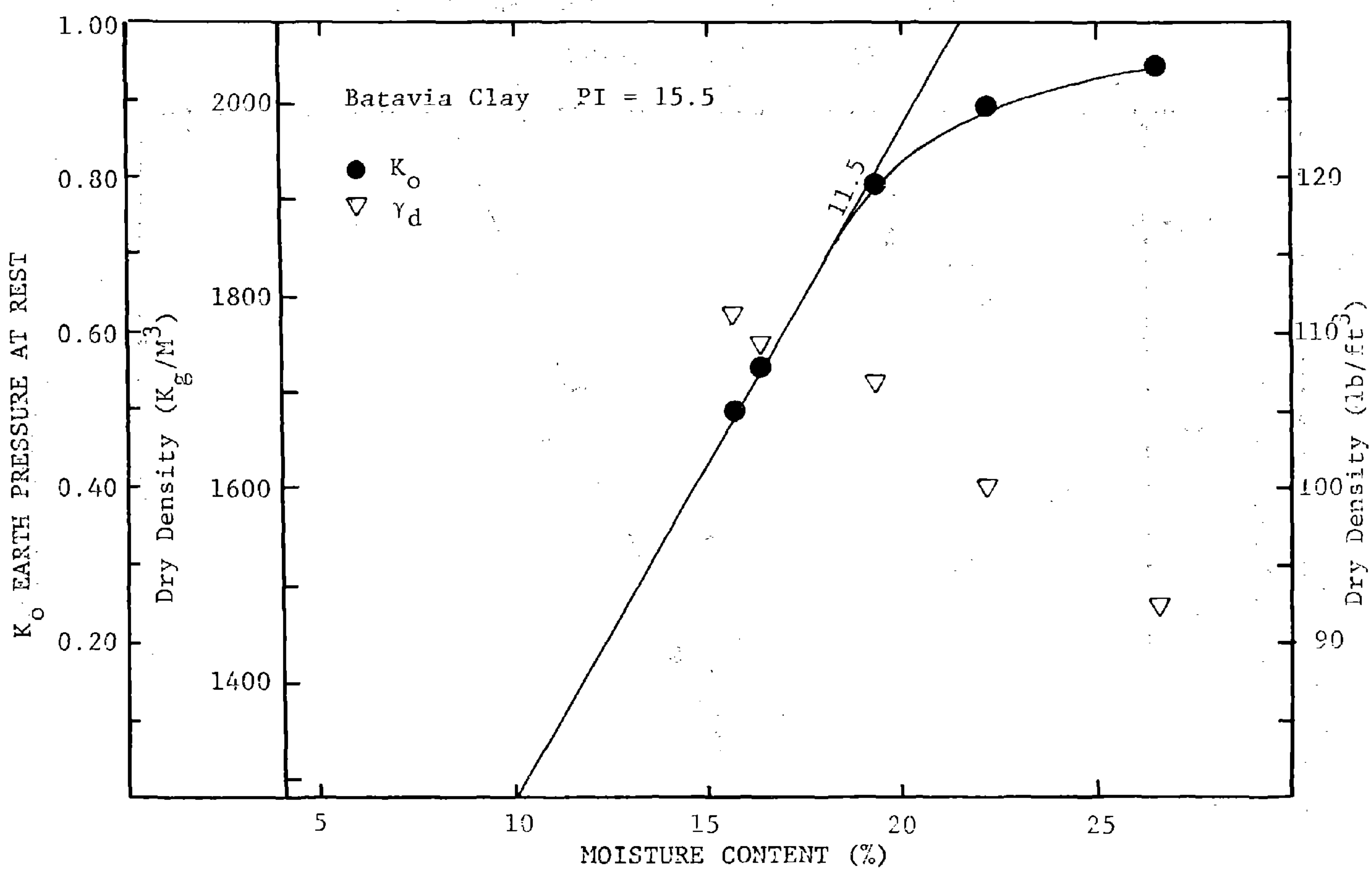


Figure 12. K_o Vs. Moisture Content - Batavia Clay

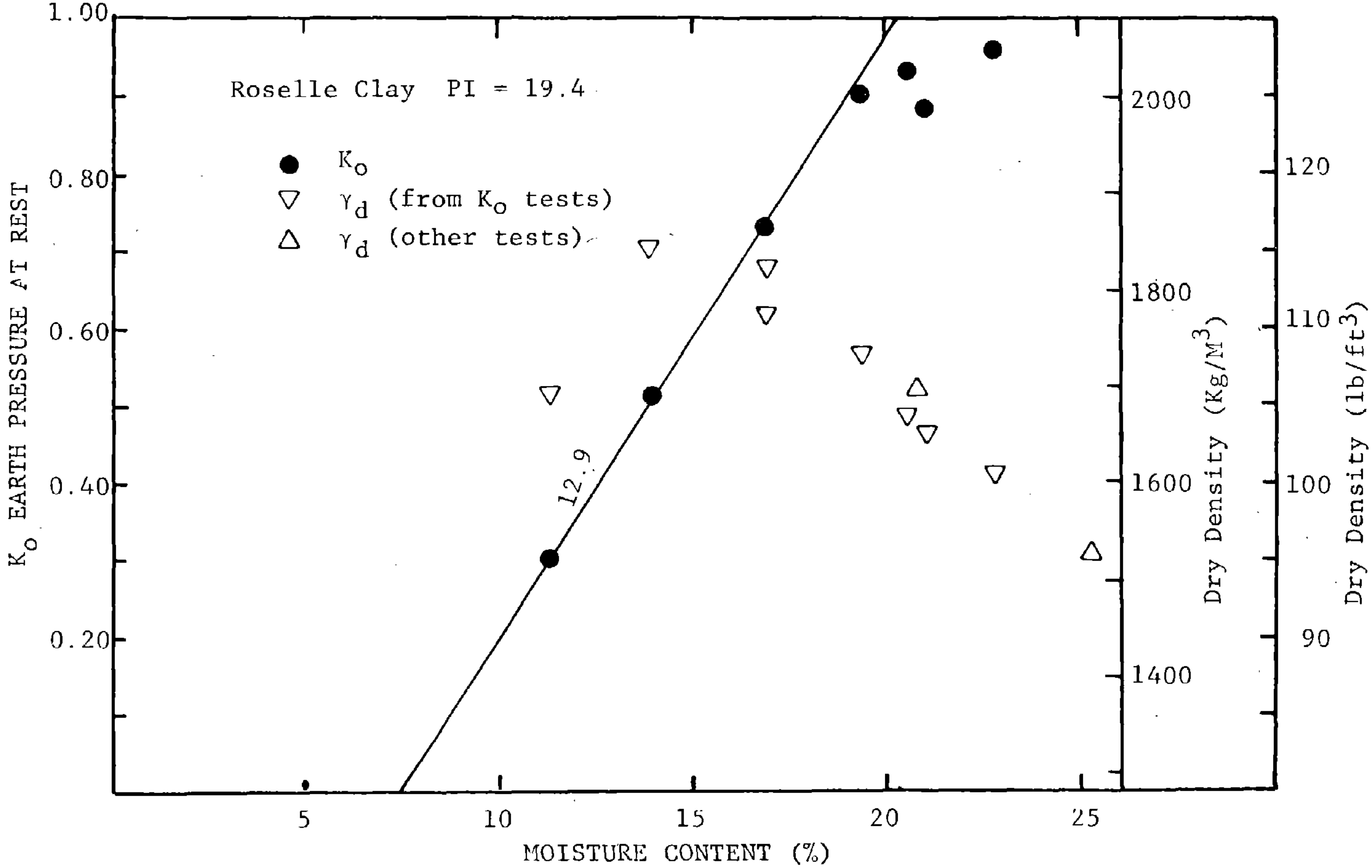


Figure 13. K_o Vs. Moisture Content - Roselle Clay

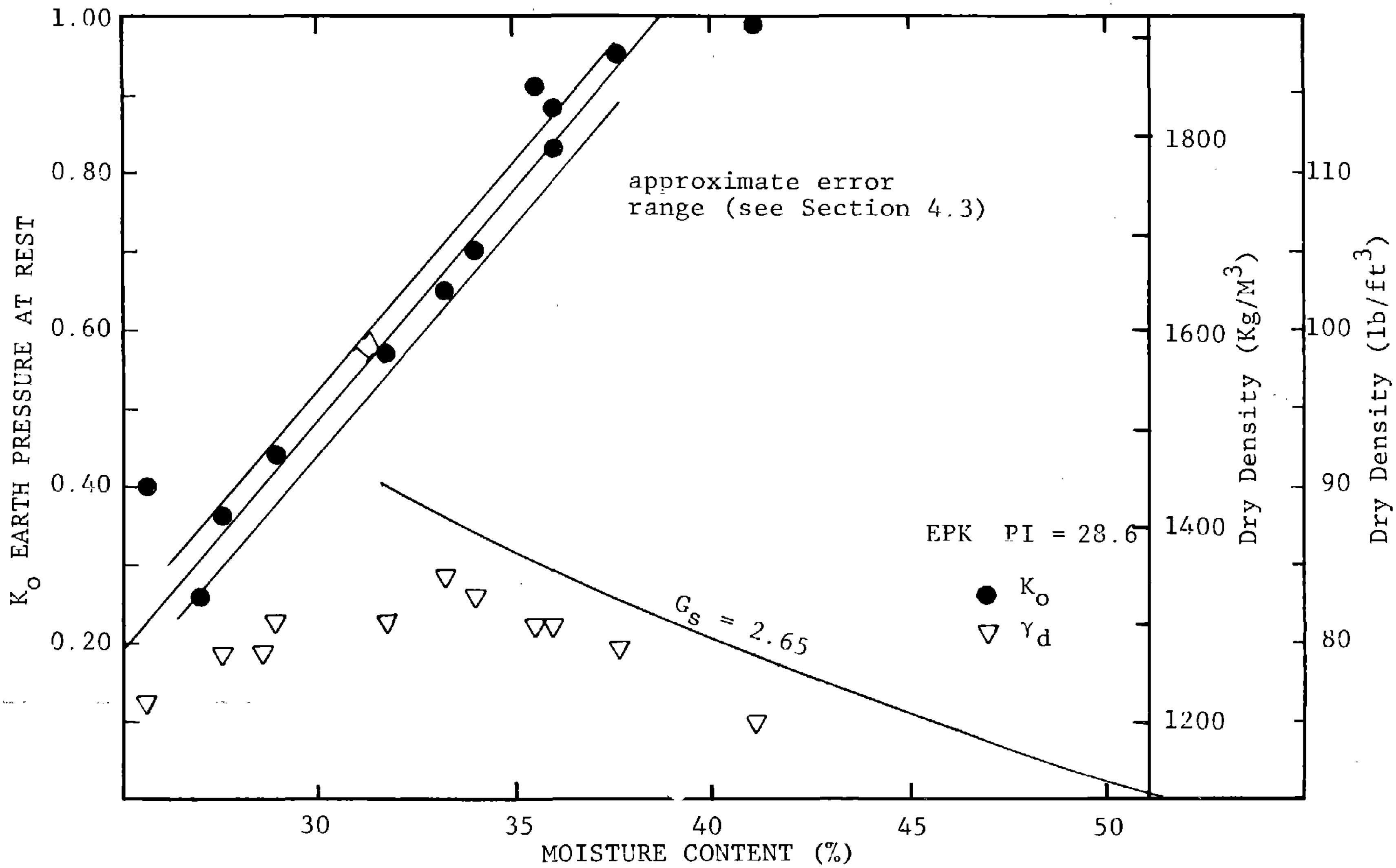


Figure 14. K_o Vs. Moisture Content - Edgar Plastic Kaolin

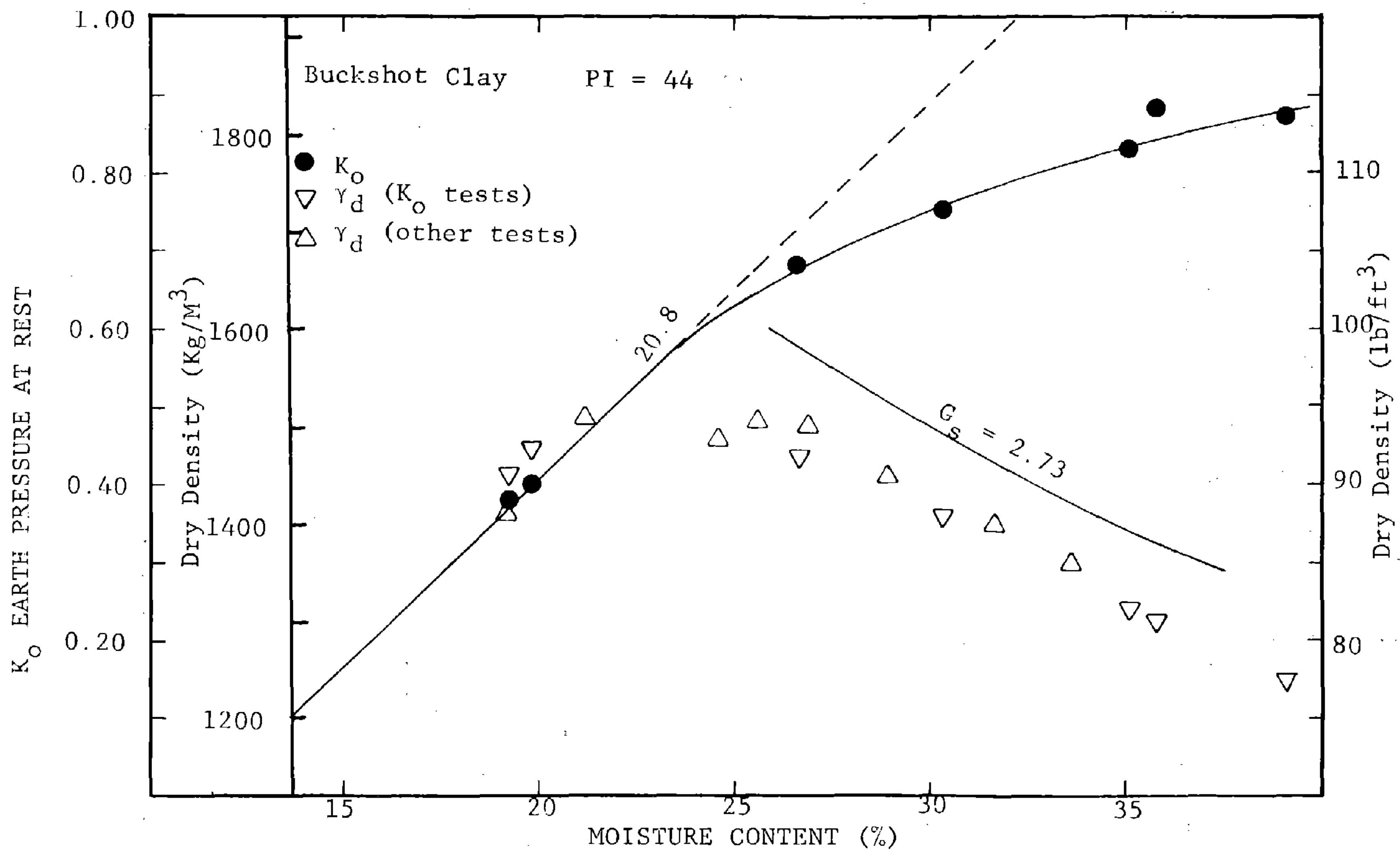


Figure 15. K_o Vs. Moisture Content - Buckshot Clay

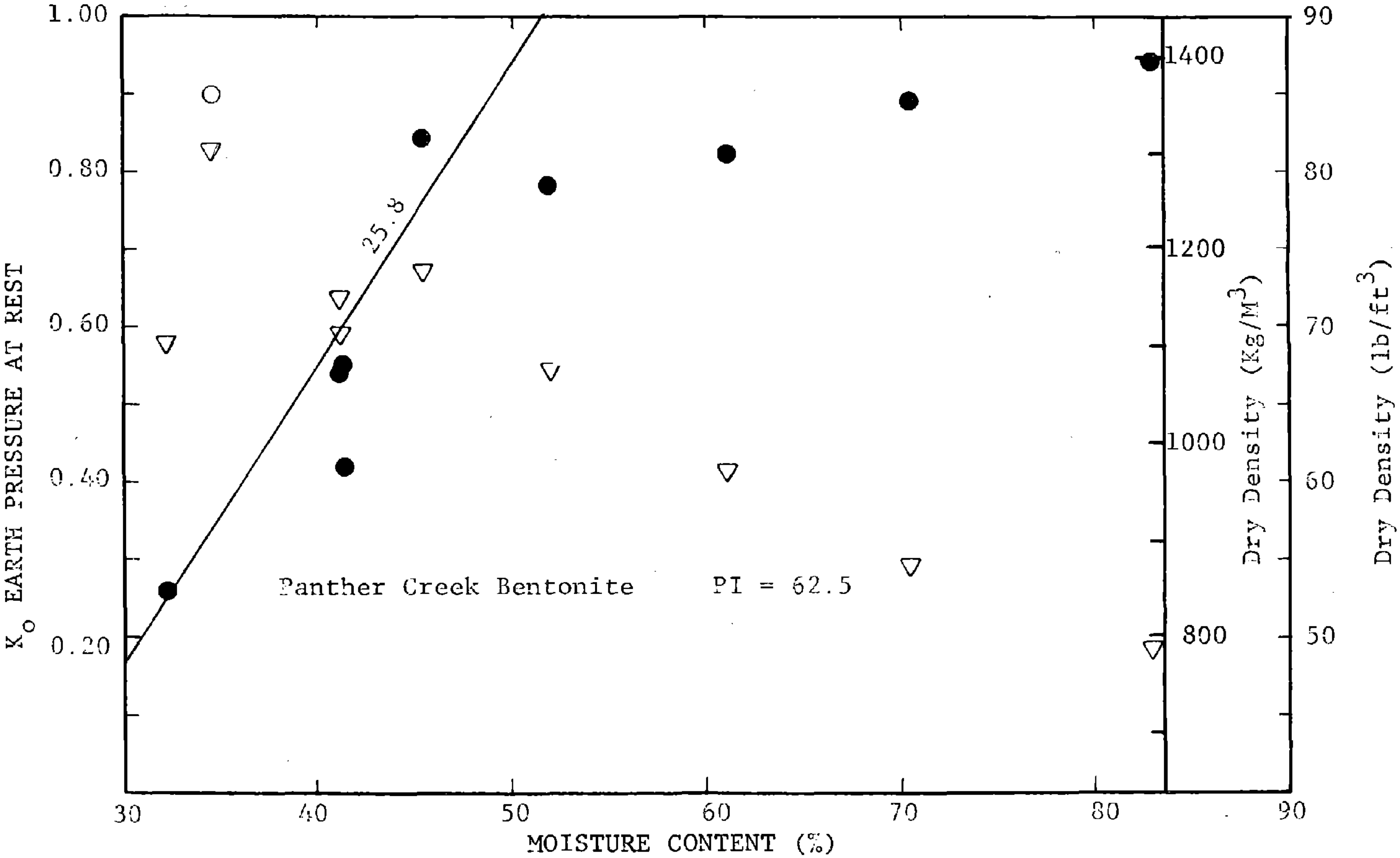


Figure 16. K_o Vs. Moisture Content - Panther Creek Bentonite

saturated by the addition of only 2-1/2 percent additional moisture. At this point, moisture and density conditions are close to those of a K_o test which produced a K_o value 0.91. The soil would have little strength following saturation. In comparison, the silty gravel shown in Figure 11 would require an increase in moisture content of nearly 4 percent to become saturated following compaction at optimum moisture content. At that point, the unchanged dry density would be 5 lb/ft^3 (0.08 gm/cc) greater than the K_o test that gave a K_o value of 0.80. In Sections 4.3.2 and 4.3.3 it will be seen that soil strength begins to drop rapidly as K_o exceeds 0.8. The clayey silt (Figure 9) is clearly more sensitive to moisture changes than is the silty gravel (Figure 11). By the AASHTO group index, both soils would be rated as good subgrade materials, with the silty gravel ranked slightly better. This is confirmed by Figures 9 and 11.

4.2 Non-Plastic Soils

Five non-plastic soils were tested. Of these, three were sufficently cohensionless that they could not be transferred from the compaction mold to the K_o cell. These specimens were compacted directly in the K_o mold, tested dry, flooded while still in the mold, and then tested wet. Test data are shown in Table 8.

Table 8. K_o Values For Cohensionless Soils

	K_o Dry	K_o Wet	% Moist.
Batavia Sand and Gravel	0.31	0.37	15.7
Ottawa Sand	0.38	0.44	20.8
Batavia Sand	0.38	0.49	23.2

Two other soils displayed sufficient apparent cohesion to permit compaction and K_o testing. K_o data for these soils are shown in Figures 17 and 18. These soils were compacted at increasing moisture contents until they became too wet to handle, but did not make the change from "stiff" to the transition range that would be indicated by increasing K_o values such as shown previously in Figures 10 for Soil No. 2. The inability to achieve K_o values as high as 0.5, plus the lack of a definite upward portion of the K_o curve at lower K_o values are taken as indicators of non-plastic soils. Experience with many slightly plastic or non-plastic soils would be required to permit the formulation of an exact boundary between the classes. The Batavia silt (Figure 18) is the nearest to being plastic of this group, but became too wet to handle before the upward transition occurred. Even near saturation, the low K_o values indicate that these soils still have stiffness.

4.3 Precision of K_o Data

Measurement of precision requires first an estimate of the error associated with a single K_o test, and then the error associated with the K_o inverse slope, I.

The EPK K_o data shown in Figure 14 were taken by three operators, including the training period of one. The estimated standard deviation for all data taken by all three operators is 0.04 K_o units. The scatter in the mid-range (K_o between 0.4 to 0.8) is on the order of 0.01. Review of all data for all tests indicate a probable error of 0.027, which is consistent with Figures 9 through 16.

Because the inverse slope, I, is taken as a measure of moisture susceptibility, the probable error in a group of K_o tests is of interest. Considering two data points having moisture contents and K_o values of (M_1, K_1) and (M_2, K_2) respectively, I is defined as:

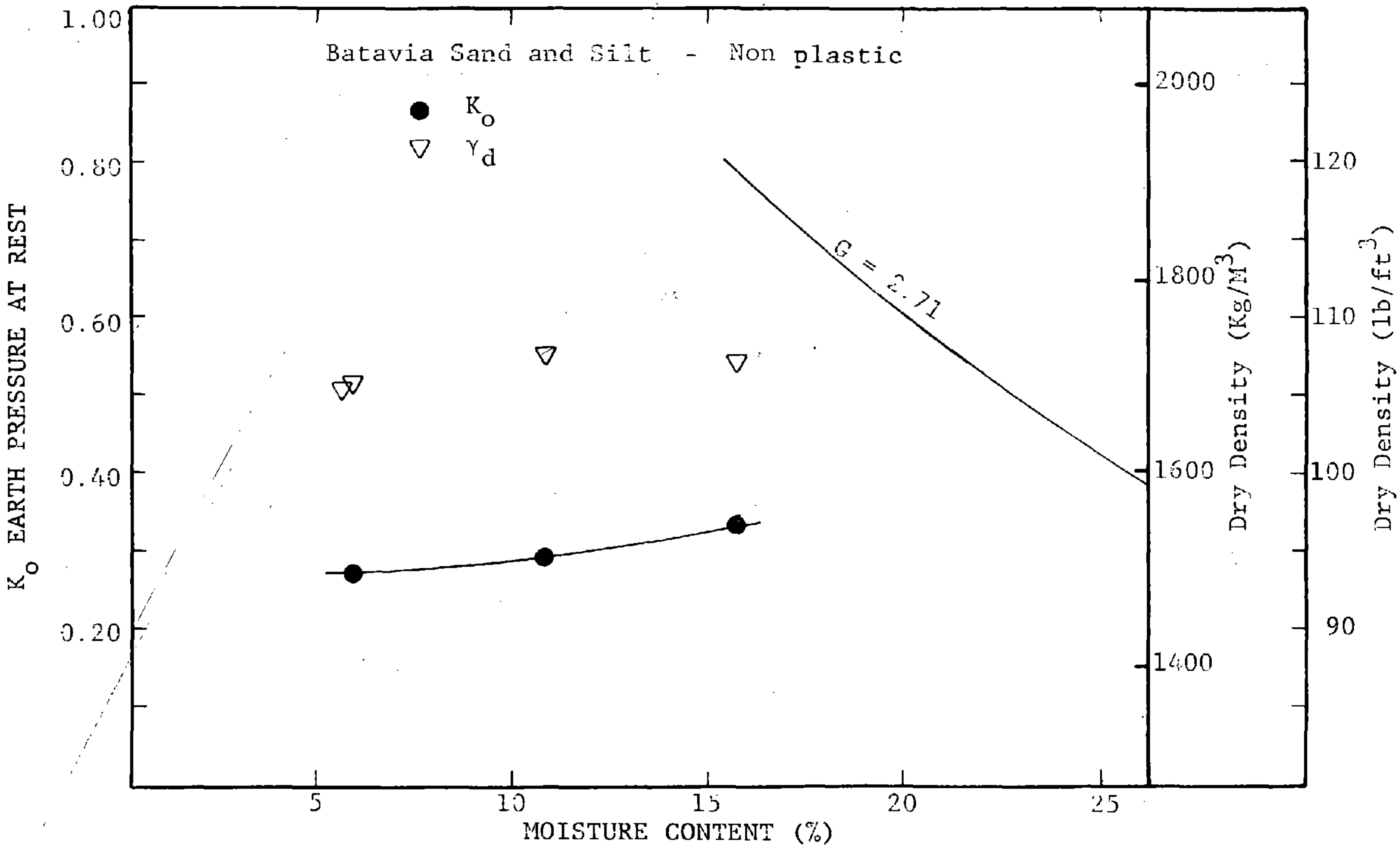


Figure 17. K_o Vs. Moisture Content - Batavia Sand and Silt

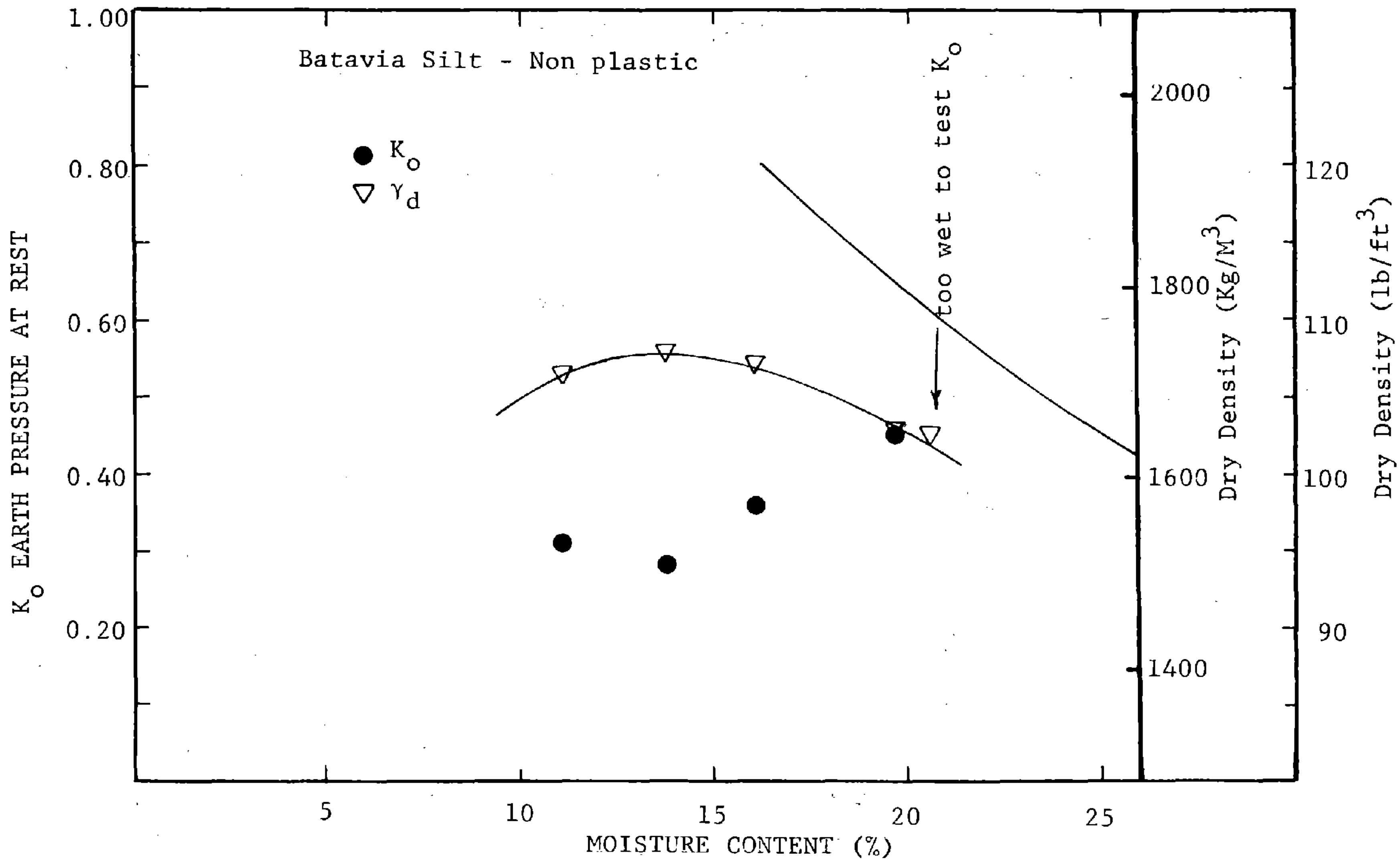


Figure 18. K_o Vs. Moisture Content - Batavia Silt

$$I = \frac{M_2 - M_1}{K_2 - K_1}$$

and the probable error E associated with I is given by

$$E = \left[\frac{2}{(K_2 - K_1)^2} \left(E_m^2 + I^2 E_k^2 \right) \right]^{1/2}$$

in which

E is probable error of I, inverse slope
E_m is probable error in moisture content
E_k is probable error in K_o

Taking as typical values

$K_2 - K_1 = 0.8 - 0.4 = 0.4$
$E_m = 0.3$ percent
$E_k = 0.027$

We obtain

$$E = 3.5 \ (0.09 + 0.0007 \ I^2)^{1/2}$$

where both I and its probable error E are measured in percent moisture. Using this formula, probable errors will increase from 1.1 to 2.6 as the measured I values move from a minimum of 4.3 to a maximum of 25. Because each I value was determined by striking a line through several data points, the actual probable errors are approximately 0.6 to 1.5 over the range of observed data. While comprehensive error analysis will require data on many soil types, it is believed that the K_o test provides an accurate measurement of soil response to changes in moisture content.

4.4 Comparison of K_o and Soil Index Properties

4.4.1 Atterberg Limits

Because the Atterberg limits have a long history, it would be most useful to find good correlations between PI and the new K_o data. The central portion of the K_o vs moisture content curve includes the moisture regime in which the soil behavior is changing from stiff to fluid. The inverse slope of this section of the K_o curve should be related to P.I. Figure 19 shows that this is the case. The empirical equation

$$I = 9.48 \ln(PI) - 14.48$$

yields a good fit for the K_o inverse slope (I) if the plasticity (PI) is known. The equation can be inverted to estimate PI if I is known. Both cases are shown in Table 9 below.

Table 9 - Correlation Between K_o and Plasticity Index

Measured Values		Estimated from PI		Estimated from I	
PI	I	I	Error	PI	Error
8.0	4.3	5.2	+0.9	7.2	-0.8
8.4	6.9	5.7	-1.2	9.5	+1.1
10.7	8.6	8.0	-0.6	11.4	+0.7
15.5	11.5	11.5	0.0	15.5	0.0
19.0	12.9	13.7	+0.8	17.9	-1.5
30.0	17.0	17.7	+0.7	27.6	-2.1
44.0	20.8	21.5	+0.7	41.2	-3.2
62.0	25.0	24.7	-0.3	64.1	1.9
		mean error	0.13		-0.49
		std. dev. error	0.77		1.73

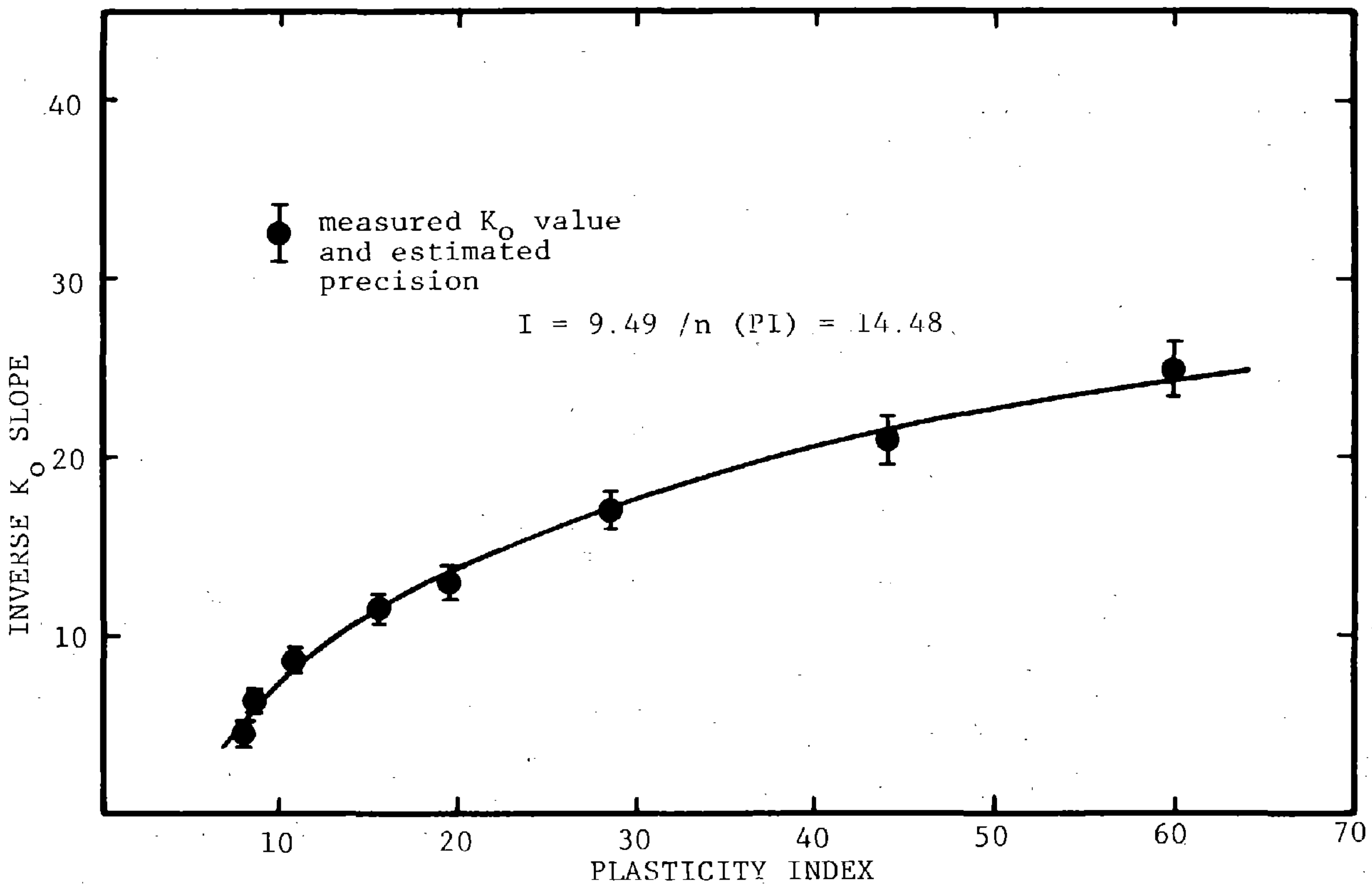


Figure 19. K_o Inverse Slope Vs. Plastic Index

Because the least squares fit was made to ln (PI), the summed square errors are not expected to be zero for the actual PI values. The standard deviation of the error in estimated plasticity index, 1.7 percent moisture, is of the same magnitude as the uncertainty normally expected in the measurement of plasticity index. It remains for future work to test this relationship for many soil types. The good agreement is a convincing argument that the slope of the K_o vs moisture curve is a good measurement of plasticity index.

The straight mid-portion of the K_o vs moisture content curve should also provide a measure of plastic and liquid limits. For this purpose, the line was extended to intersect $K_o = 0$ and $K_o = 1.0$. The moisture contents at these points are compared with the plastic and liquid limits, respectively, in Figures 20 and 21. The data scatter and the coefficients of determination (0.96 and 0.95) indicate a correlation less perfect than was obtained for plasticity index. The data are listed in Tables 10 and 11 below.

Table 10 - Correlation Between Moisture Content (A) at $K_o = 0$ and Plastic Limit

Measured Values		Estimated from PL		Estimated from A	
PL	A	A	Error	PL	Error
34.4	24.1	22.8	-1.3	36.1	1.7
15.2	9.7	7.9	-1.8	17.5	2.3
17.3	7.9	9.5	1.6	15.2	-2.1
16.8	10.0	9.2	0.8	17.9	1.1
17.3	7.3	9.5	2.2	14.4	-2.9
32.0	21.8	20.9	-0.9	32.0	0
19.2	11.5	11.0	-0.5	19.8	0.6
40.2	25.8	27.2	1.4	38.3	-1.9
		mean error	-0.01		-0.15
		error std. dev.	1.51		1.93

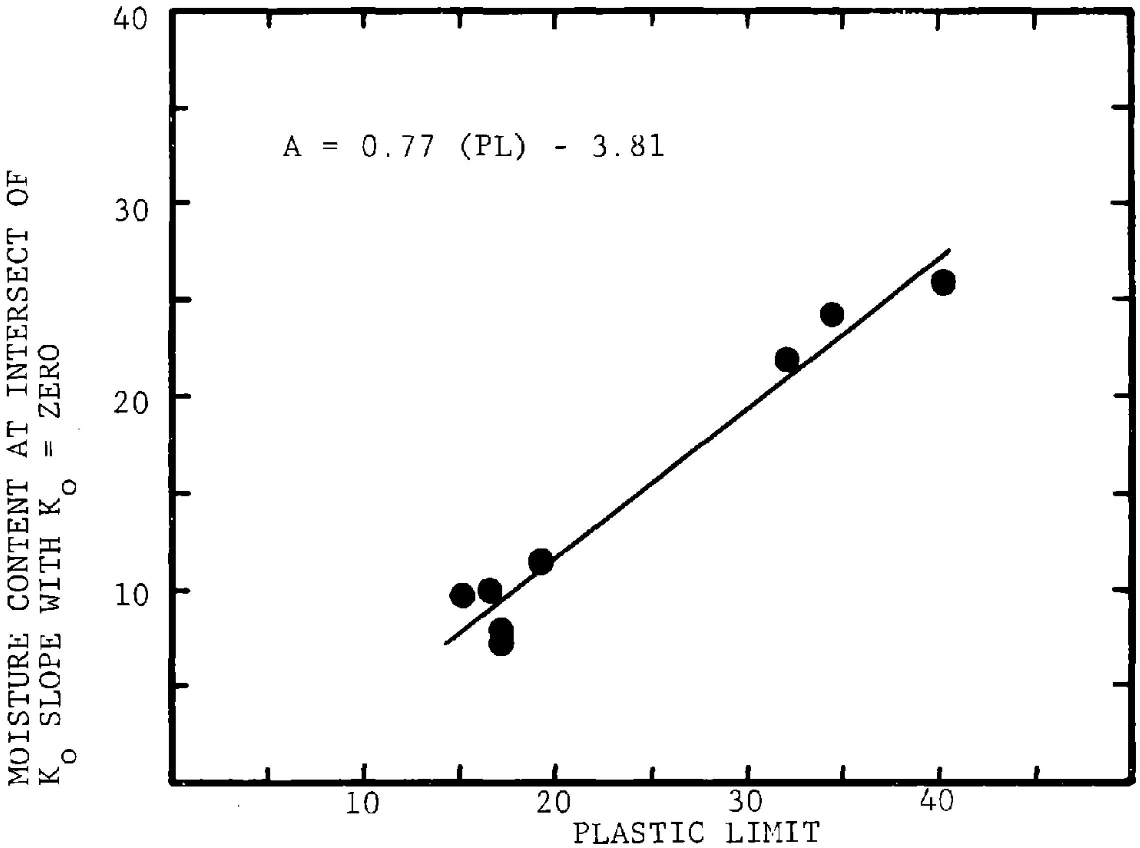


Figure 20. K_o Parameter A Vs. Plastic Limit

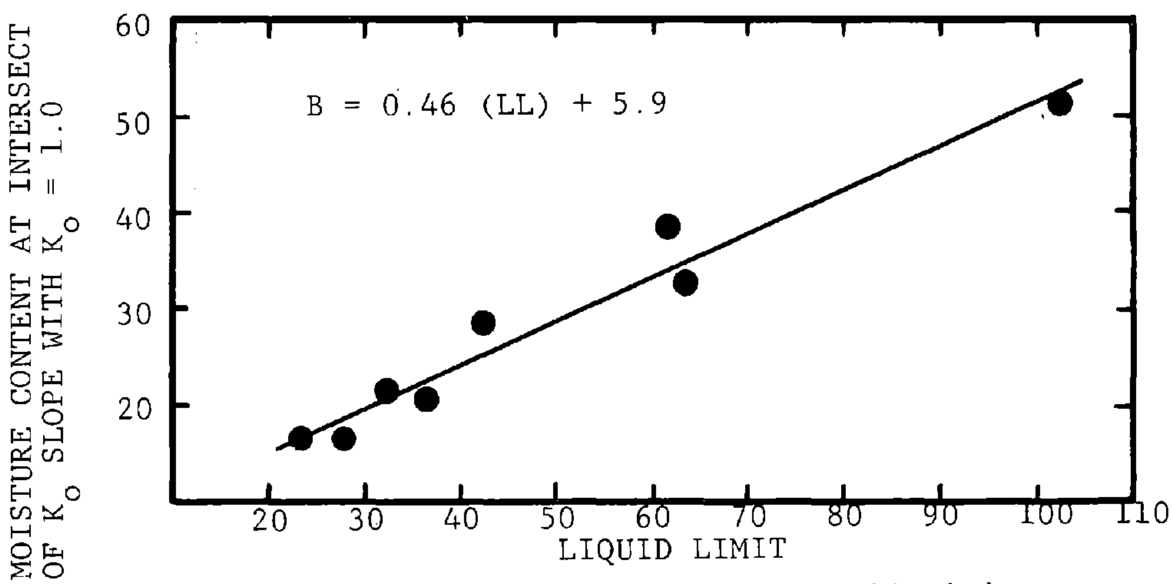


Figure 21. K_o Parameter B Vs. Liquid Limit

Table 11 - Correlation Between Moisture Content at $K_o = 1$ (B) and Liquid Limit

Measured Values		Estimated from LL		Estimated from B	
LL	B	B	Error	LL	Error
42.4	28.5	25.3	-3.2	49.4	7.0
23.6	16.7	16.7	0.0	23.6	0.0
28.0	16.4	18.7	2.3	22.9	-5.1
32.3	21.5	20.7	-0.8	34.1	1.8
36.7	20.3	22.7	2.4	31.5	-5.2
61.7	38.8	34.1	-4.7	71.9	10.2
63.6	32.2	35.0	2.8	57.5	-6.1
102.4	51.6	52.8	1.2	99.5	-2.9
		mean error	0.00		-0.04
		error std. dev.	2.76		6.03

It can be seen that the plastic and liquid limits may also be extimated from K_o data, but with less precision than can the plasticity index. This section is summarized in the following table.

Parameter	Equation	Standard Deviation
plastic index	$PI = \exp \frac{I+14.48}{9.49}$	1.73
plastic limit	$PL = \frac{A+3.81}{0.77}$	1.93
liquid limit	$LL = \frac{B-5.9}{0.46}$	6.03

The fact that the PI and PL can be estimated within 2 percent moisture, and the liquid limit to within 6 percent, is gratifying, inasmuch as there was no intent to design a new Atterberg limits device. It was expected that any new device designed to measure soil consistency would correlate well with the Atterberg limits for soils not having large amounts of plus No. 40 material. The correlations could be improved by making use of the redundancy contained in the relationships

$$PI = LL - PL$$
$$I = B - A$$

applying to the two test methods. The correlation equations for PL and LL should be changed to the semi-log representation used for PI for this transform to be valid.

4.4.2 Plate Bearing

The plate bearing tests conducted with the 1.95 in. (4.95 cm) diameter plate provide convenient strength and stiffness data against which to compare the K_o data.

The ultimate plate load, as given by the hyperbolic curve fit, is shown as a function of K_o in Figure 22. As is expected, each soil decreases uniformly in strength as K_o increases. At K_o values in the general vicinity of 0.9, the ultimate loads abruptly plunge. The slopes on the upper portions of these curves are quite uniform. These soils all are reduced in ultimate load by 10 to 25 percent for each increase of 0.1 in K_o.

The initial plate modulus (lb/inch deflection) is plotted in Figure 23. The variation in absolute stiffness is large for this range of soils, but again the reduction in stiffness is consistently between 10 to 25 percent for each increase of 0.1 in observed K_o.

These two plots indicate that a soil must be maintained at a K_o value less than 0.8 to 0.9. At the moisture content that results in a K_o of 0.95, stiffness is on the order of several hundred pounds per inch, and ultimate loads for the three square inch plate are below 100 pounds (450 Newtons).

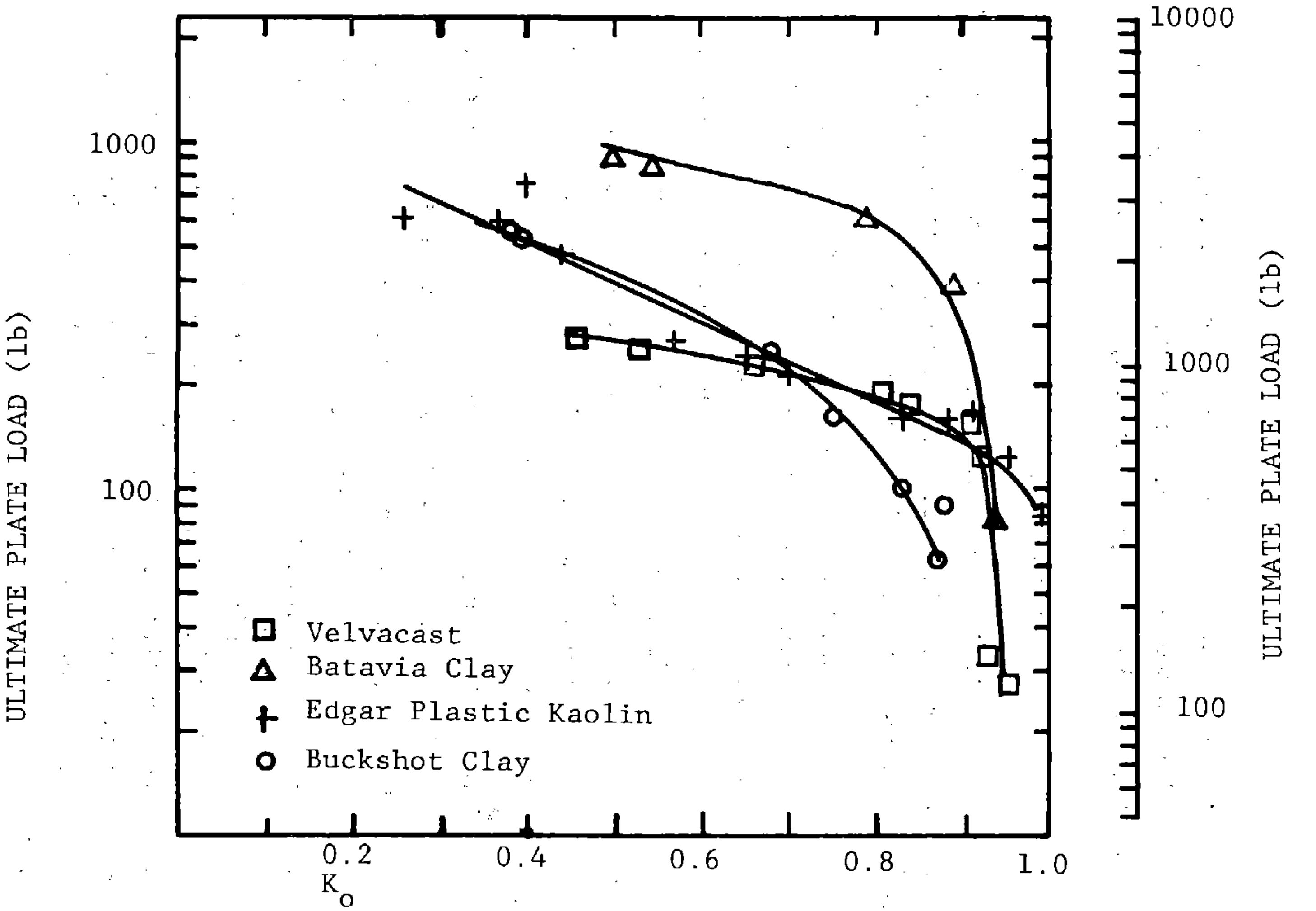


Figure 22. Ultimate Plate Bearing Load as a Function of K_o

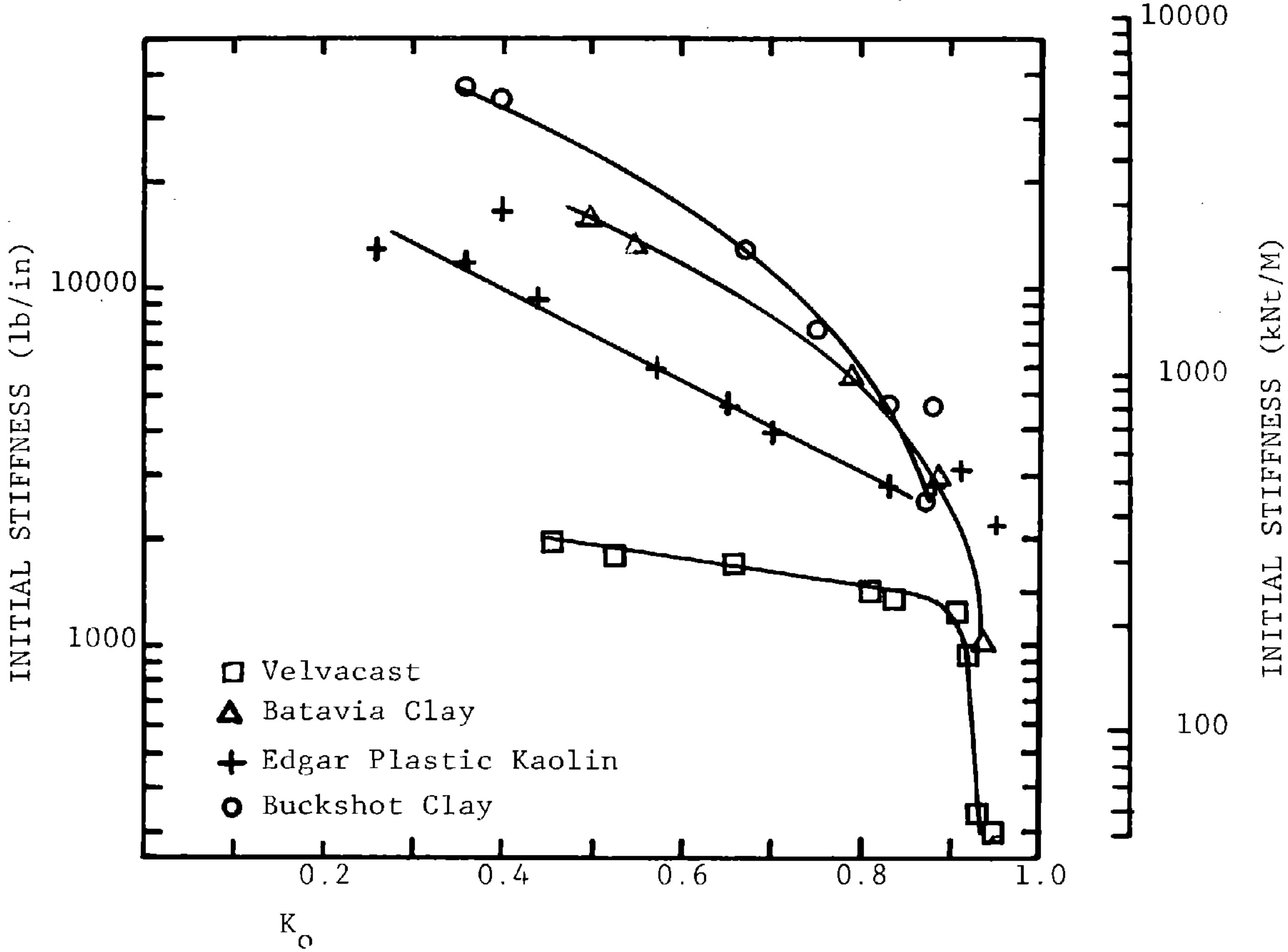


Figure 23. Bearing Plate Initial Stiffness As A Function of K_o

4.4.3 California Bearing Ratio

The California Bearing Ratio (CBR) test is a plate bearing test conducted under specific overburden and moisture conditions. The test conditions are selected to conform with the planned use of a soil. The CBR test, like any soil shear test, is moisture sensitive, and may be compared with K_o values measured at corresponding moisture contents. The CBR tests reported in Table 3 "Standard Tests" were conducted at single moisture-density conditions provide only limited data. However, several soils were tested over a range of moisture contents using the 1.95 in (49.5 mm) plate bearing apparatus. From these data, zero overburden unsoaked CBR values may be computed.

The variation of CBR with moisture content is shown in Figure 24. The transition from stiff to fluid behavior is displayed by the straight portions of each curve, generally in the range $0.5 < CBR < 10$. For this portion, a strength decrement can be computed as the slope of the CBR-moisture curve. This decrement measures the increase in moisture content that will result in a factor of 10 reduction in CBR. Like the K_o inverse slope, this is a moisture susceptibility indicator. In Figure 25, CBR strength decrement is shown as a function of K_o inverse slope. In both parameters, low values indicate that soil behavior changes rapidly with small changes in moisture, while high values indicate that a large change in moisture is required for a given change in behavior. The expected trend is apparent: the smaller I values are associated with soils that lose strength rapidly as moisture content increases.

In figure 26 we consider the relationship directly between K_o and CBR. The frictional materials are not susceptible to moisture content. For these materials, K_o is always near the value 1/3, and CBR depends upon density and overburden. The silts and clays display a

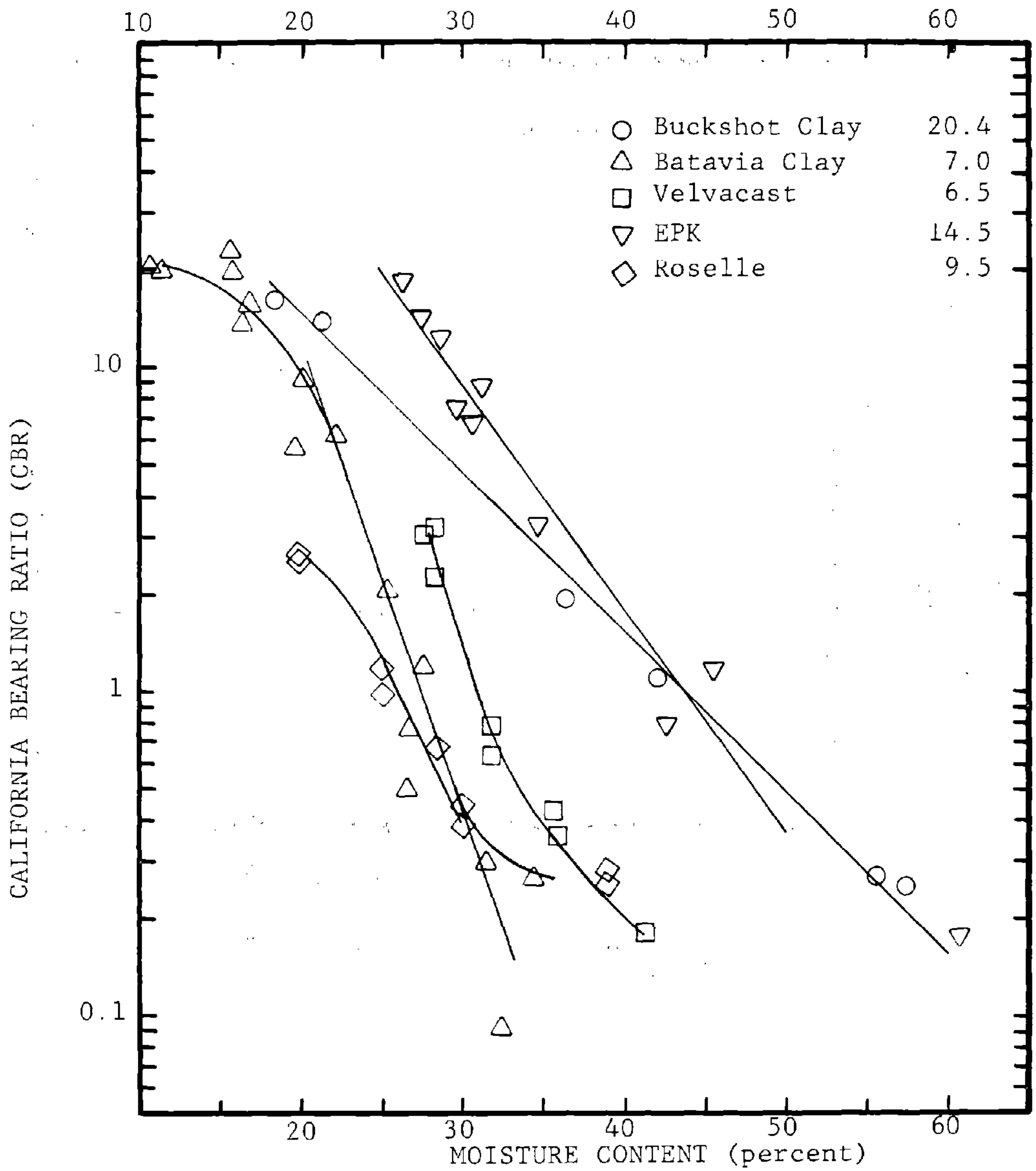


Figure 24. Decrease of California Bearing Ratio with Increasing Moisture At Compaction

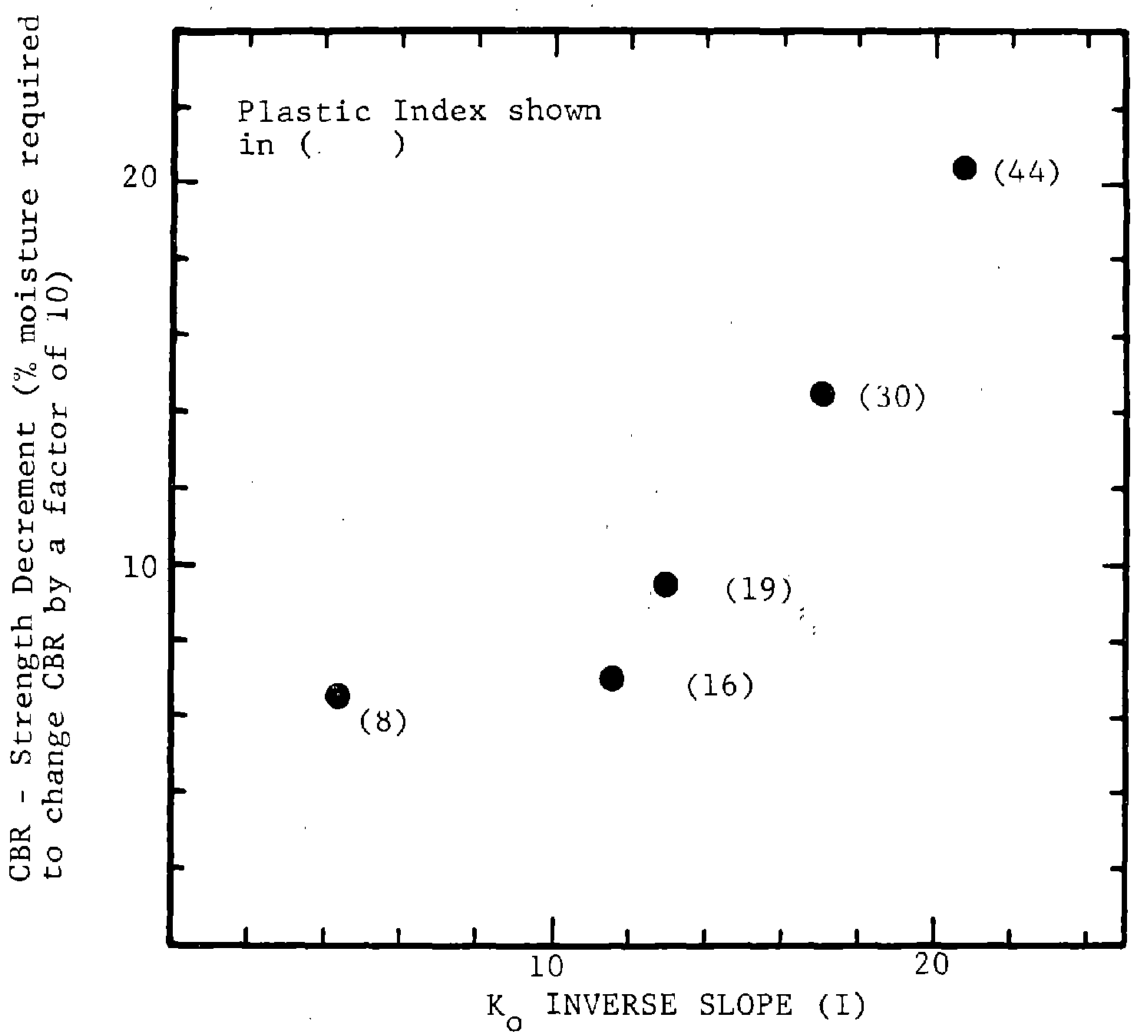


Figure 25. Comparison of Moisture Susceptibility Measured by K_o test and CBR test.

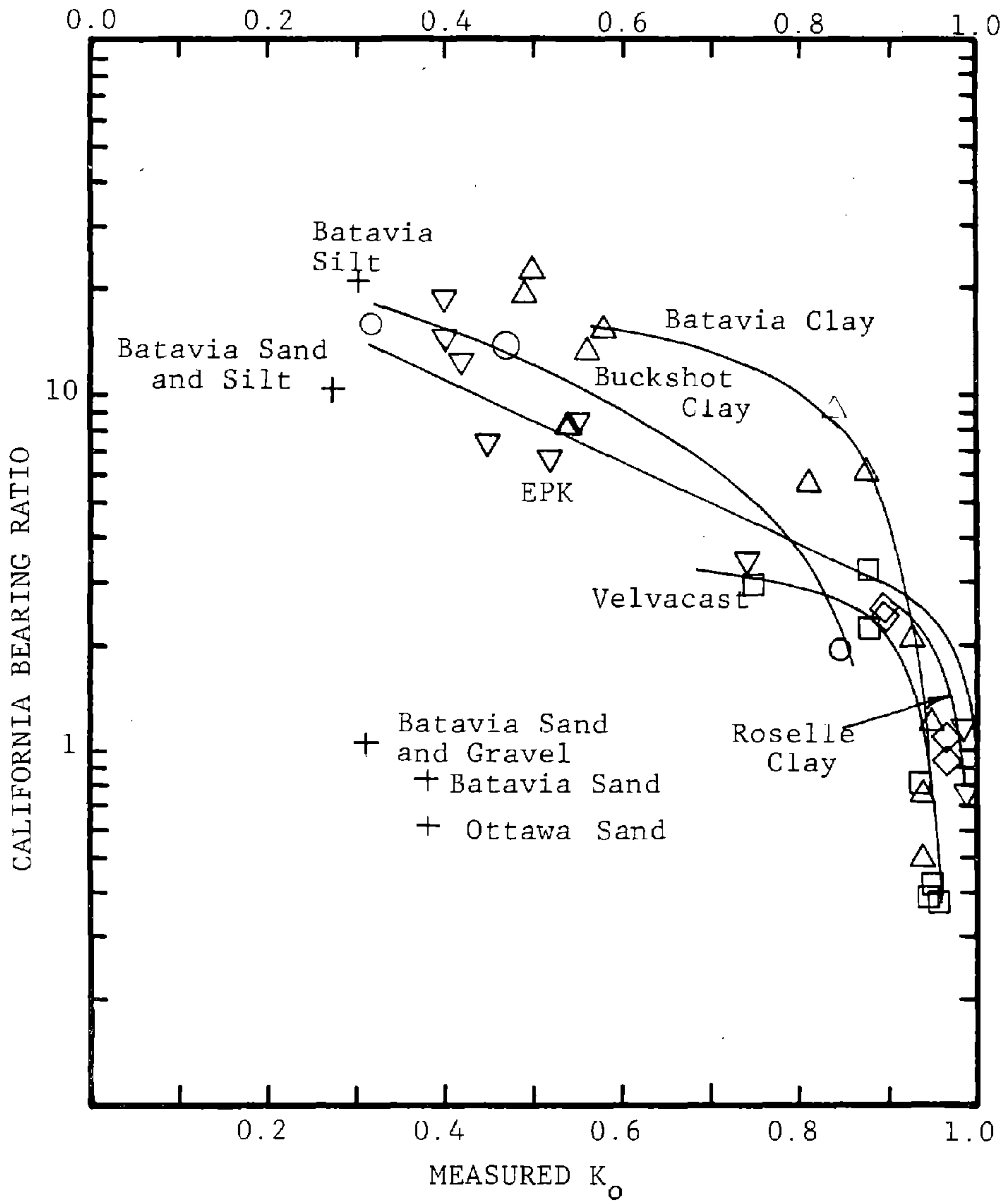


Figure 26. Comparison of K_o Values With California Bearing Ratio

broad range of K_o values, as shown by the curves. These materials consistently lose 10 to 25 percent of their strength for each increase of 0.1 in K_o. Strength drops rapidly for K_o values above 0.8 to 0.9. For those soils such as Velvacast that change rapidly from stiff to fluid behavior, small changes in moisture content may result in significant strength loss.

5.0 APPLICATIONS

There are several possible applications for a soil consistency test that provides a rational measurement of soil behavior with varying moisture content. These include soil classification, design of earth structures, and construction control.

5.1 Classification Systems

Classification systems using the new test method may be developed either by use of the correlation equations to translate established systems or by creating completely new systems.

The current AASHTO system employs liquid limit, plasticity index and particle size analysis. This may be translated directly by replacing PI with I and LL with B as follows:

PI less than 6 implies I less than 2.5
PI less than 10 implies I less than 7.8
LL less than 40 implies B less than 24.5

The correlations of I with PI and B with LL are based on only eight soil types and should be regarded as tentative until the data base is increased. The particle size analysis would still be required for a direct translation. The use of such a translated classification system would permit the direct application of the large backlog of experience and data available from long usage of the Atterberg limits. A similar translation for the group index could also be made. Such use of K_o parameters would provide a safe mechanism for the creation of a new classification system, being backed up by previous state of the art. Prior to such use, the correlation equations should be defined by additional testing.

A revolutionary, as opposed to evolutionary, approach is to redefine soil classification in terms of mechanical response rather than blow counts. The physical characteristics that are desirable in earth structures (strength, stability, lack of sensitivity to moisture changes) would be established as low K_o at optimum moisture. Such a classification system would have the advantage that it would incorporate within itself parameters for design and construction. The variation of stiffness and strength with varying moisture content are parameters that enter directly into rational design methods. This test method develops the critical data on the loading portion. The test could easily be modified to hold a constant load for as long as desired to develop the creep data necessary to define the visco-elastic soil properties that may be used in future rational design methods.

5.2 Design and Construction

The availability of soil classification data that gives stiffness directly and strength by close correlation should prove to be of considerable value in design. Also, the ability to measure any change in soil classification resulting from special treatments that may be used to improve the soil in the field will be useful. The applications possible in design seem broad and limited only by the design method itself.

Construction specifications typically accept or reject soil-aggregate materials on the basis of classification tests that place the soil on one side or the other of a sharp boundary between soil groups. This may lead to dispute in borderline cases. If we instead require that the soil, as compacted, have a minimum strength or stiffness, we will have established a performance specification that recognizes the fact that soils are not found clustered in separate classes, but rather display a smooth spectrum of behavior from good to bad. Under this format, small variations in a soil type, or random variations in test results would not cause abrupt changes in the soil classification, but would indicate instead the need for slightly more or less compactive effort or other treatment. This is more desirable than a rigid yes/no acceptance criterion that may be triggered by random test variations in borderline cases. The primary specifications would be derived from the K_o and dry density data generated by the K_o test. The control test would be the same density test as is currently used in earthwork, but with acceptance levels based upon the primary specification.

Such specifications would be easier to administer than the current accept/reject materials criteria, and would relate more directly to rational design methods based on soil strength and stiffness. The development of such specifications would seem to be worthy of sustained effort on the part of the highway community.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The objective of this program was to develop a practical, automated methodology for measuring soil and soil-aggregate response to moisture changes. To meet this objective, an apparatus was designed and fabricated which is capable of measuring the coefficient of earth pressure at rest (K_o) on specimens compacted in standard 4 inch (101.6 mm) compaction molds. Defined under conditions of zero lateral strain ($\varepsilon_2 = \varepsilon_3 = 0$), K_o may be expressed as a stress ratio (σ_3/σ_1) or as an elastic property ($\nu/1-\nu$). For any material not containing residual stress, K_o must lie between zero (perfectly rigid behavior) and one (perfect fluid). Values usually expected for soils at normal field moisture and density conditions range between 1/3 and 1/2.

The K_o test hardware developed under this program includes a test cell sized to accept a standard density specimen and a control box containing pneumatic controls and strain gage bridge circuitry. An X-Y recorder was used to record the data. Tests were routinely run by loading at 2 psi (14 kPa) per second up to 60 psi (400 kPa), except when particularly strong specimens required higher pressure. Axial and lateral stresses are recorded continuously and automatically. K_o is obtained from the slope of the X-Y recorder plot of axial vs lateral stress. The K_o test requires approximately 8 minutes in addition to the time required to produce moisture-density specimens. Approximately half of the required extra time is available to begin compaction of the subsequent specimen.

During this program, eight plastic and five non-plastic soils were compacted at different moisture contents for determination of K_o. The fine-grained plastic soils displayed the full range of behavior; stiff, fluid and an intermediate transition stage. The non-cohesive soils displayed no fluid behavior, and at most only a small portion of the transition stage. In none of the five non-plastic soils was a K_o value greater than 1/2 observed. The K_o vs moisture content data, shown in Figures 9 to 18, provide a direct measurement of soil consistency and its behavior as compaction moisture content changes.

One of the soils tested on this program contained small amounts of aggregate up to 1-1/2 inch (38 mm) in size. All K_o tests were conducted on whole soils - no aggregate was removed. The K_o cell fabricated on this project is probably adequate for material such as crushed base containing large amounts of 3/4 inch (19 mm) aggregate. If it is desirable to test soils containing much aggregate larger than this, a test cell based on the 6 inch (152 mm) compaction mold could be used. The K_o test itself is scale-invariant and hardware of any practical size desirable might be fabricated.

The K_o test is designed to be run on specimens compacted for moisture-density testing. For cases in which moisture-density data would normally be generated, the K_o test can be incorporated at the cost of several minutes per test. Because the device is semi-automated, K_o tests on one specimen can be conducted while mixing and compacting the subsequent specimen.

The K_o test data were compared to Atterberg limits, plate bearing and CBR tests. The correlations with the Atterberg limits were sufficiently good to warrant the formulation of tentative predictive relationships. The comparison between K_o and the strength tests show that both plate resistance and CBR drop rapidly as K_o increases above 0.9. In the transition range, where K_o is between 0.4 and 0.8, the CBR, ultimate plate load and plate stiffness all decrease 10 to 25 percent for each increase of 0.1 in K_o.

When considered as a soil classification test, the K_o test has the following advantages:

* tests are run without drying the soil to near zero moisture content.
* tests can be conducted on soils following stabilization or other treatment.
* tests include the response of the soil to compaction.
* tests are run on the whole soil - no size fractions are removed.
* moisture-density data are generated as a product of the K_o test method.

The test data may be used to classify soil according to the existing AASHTO system by applying the tentative correlations with liquid limit and plastic index. It is believed, however, that the combination of consistency plus moisture-density data produced by the K_o test may form the basis of a more desirable soil classification scheme for use with rational design methods.

6.2 Recommendations

It is believed that the K_o test method has considerable potential for use in soil classification and the design and construction of soil structures. The ultimate implementation of the method will require additional development, some of which is recommended below:

a) A soil classification system should be based on many more tests than could be conducted within the context of a single project in one lab. Consideration should be given to the accumulation of many data points. This should be a coordinated effort which includes testing the same soils in several laboratories.

b) Test data should be extended to consider the effects of stabilization and changing moisture content after compaction.

c) User agencies should assist in the development of new design procedures and construction specifications on a long range basis as more data becomes available.

d) Modifications in the hardware and methods should be considered as the backlog of experience with the test method builds. Several such are recommended in the appendix. These will not change the test results, but will act to further reduce cost of hardware and testing.

We recommend, then, that effort in the immediate future be directed to the development of a large body of data including K_o vs moisture-content, strength data and Atterberg limits for 50 to 100 soil types. These should include the natural soils as well as soils made by mixing increasing portions of silt or silica flour into highly plastic clays to develop a smooth spectrum of soil consistency. The effort will firmly establish the correlations between the three parameters, so that future development can take advantage of past experience.

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Three lists of references are included below. These include group A references cited in the report, group B cone penetrometer citations, and group C earth pressure at rest citations. A given citation appears only in that group that seems most appropriate to its use in this report.

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APPENDIX - K_o DEVICE AND TEST METHOD

A.1 K_o Hardware

The K_o device consists of a test cell and reaction frame, and a control/instrumentation package. Supporting needs include gas pressure, X-Y recorder with associated electrical power and a D.C. voltage source. In the field, 110 volt line current is usually available for the recorder. Gas pressure can be supplied by compressor or small tanks of compressed air or nitrogen, while a 12 volt auto battery is unexcelled as a D.C. voltage source for strain gage circuitry.

A.1.1 K_o Device

The K_o device, shown in Figure 27, includes the cell body and two loading heads. It is designed to test soil specimens compacted in a standard 4 in. (102 mm) compaction mold. The cell body is a honed stainless steel cylinder with a plastic cover for eacy handling. On a production run of many units, the covers would be segments of plastic pipe. The cell body senses lateral pressure by strain gages mounted on the exterior of the thin wall section at mid-height. Bleed holes near the ends of the body permit any air trapped under the membrane to escape when testing is initiated. This membrane provides a lubricated surface between the soil and the cell wall as well as protecting the honed finish.

The cell wall thickness was selected to provide a radial strain of 1 micro-strain per psi of internal lateral stress. Standard pressure vessel equations were used to select the wall thickness. For internal pressures of several tens to several hundreds of psi, routine strain bridge

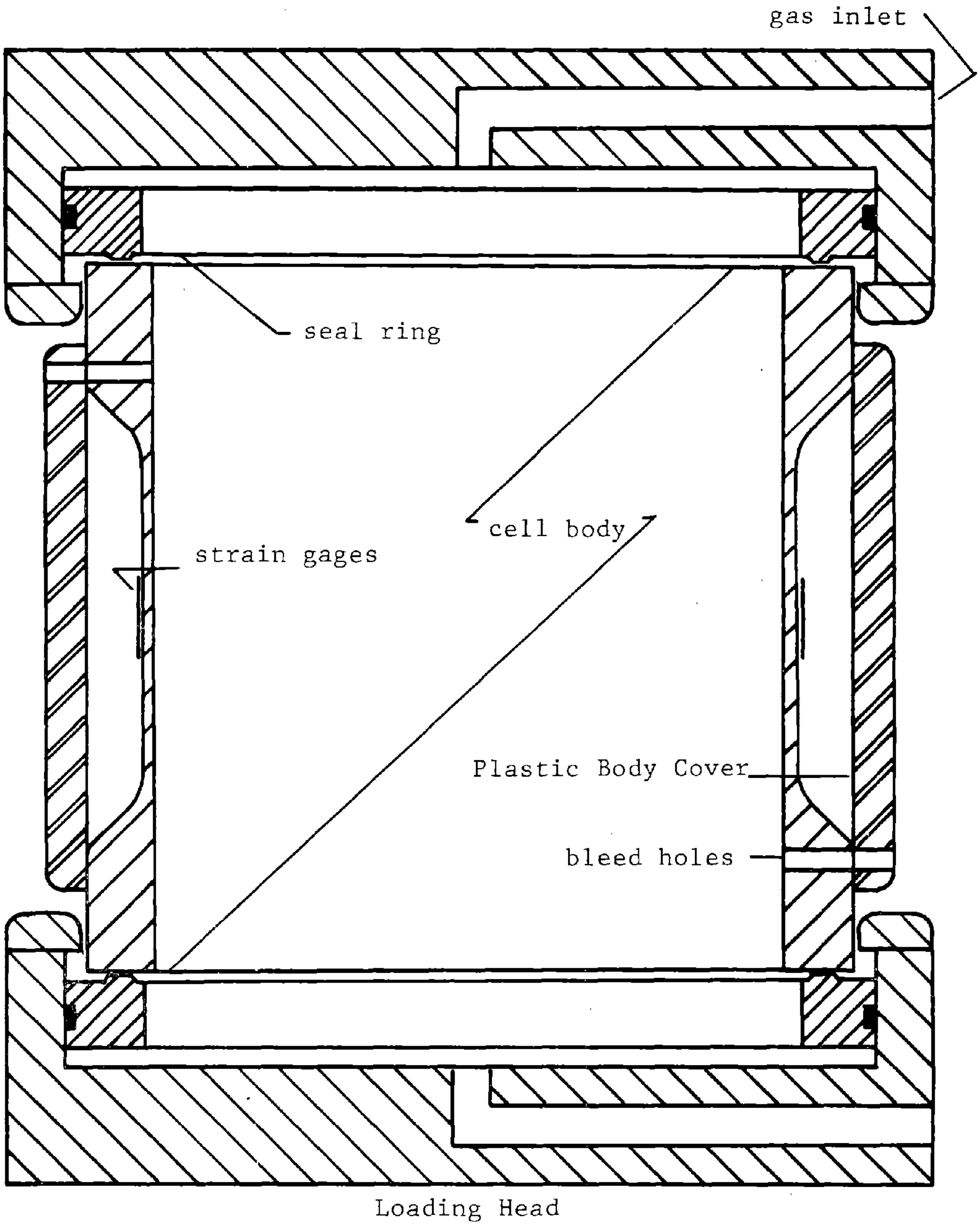

Figure 27. Assembly Drawing of K_o Device

circuitry will provide adequate signal levels.

The loading head assemblies consist of loading head, seal ring, six 1/4 by 3/4 compression springs to load the seal ring, and a seal retainer. The seal ring carrier is brass, and carries a captive "O" ring on its outer diameter. The seal against the end of the cell body is accomplished by a small protrusion on the seal carrier that seals against a flat latex membrane which covers the end of the soil specimen and cell body. This membrane is trapped under the seal carrier retainer drum-head fashion, and prevents the gas pressure within the loading head from entering the soil specimen. Axial load is provided by gas pressure which enters the side of the loading head as shown. Both loading heads are pressurized so as to reduce the effective specimen length to 2-1/4 inches (57 mm).

The entire device, when loaded, is mounted in a simple reaction frame fabricated from rolled channel and threaded rod.

A.1.2 Pneumatic Circuitry

The K_o device is operated by supplying air or gas pressure to both loading heads. This provides a uniform axial stress to the soil. Testing is accomplished by turning a single valve to "load" or "unload" as appropriate. Secondary controls permit fine control over load and unload rates, rapid dump and isolation of the K_o cell. Air pressure is regulated and monitored by two small bourden tube gages. The pressure signal is read by a commercial transducer. A dead pressure volume is used to buffer the loading/unloading rates, maintaining uniformity regardless of soil stiffness. Pneumatic circuitry is shown in Figure 28, a photograph of the system in Figure 29. Test control being the function of a single valve and pressure gage, the test is rapidly learned by inexperienced personnel.

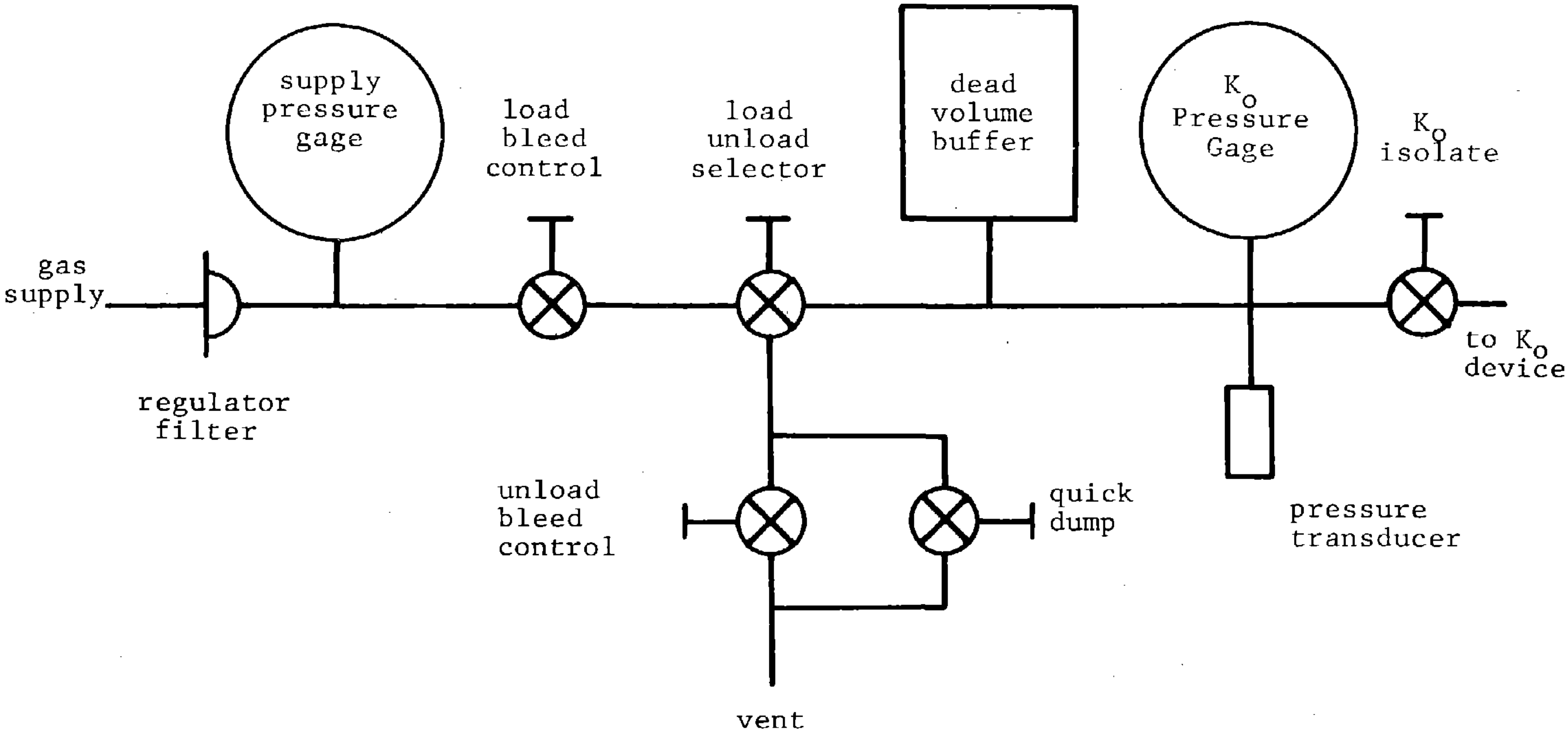

Figure 28. Pneumatic Controls

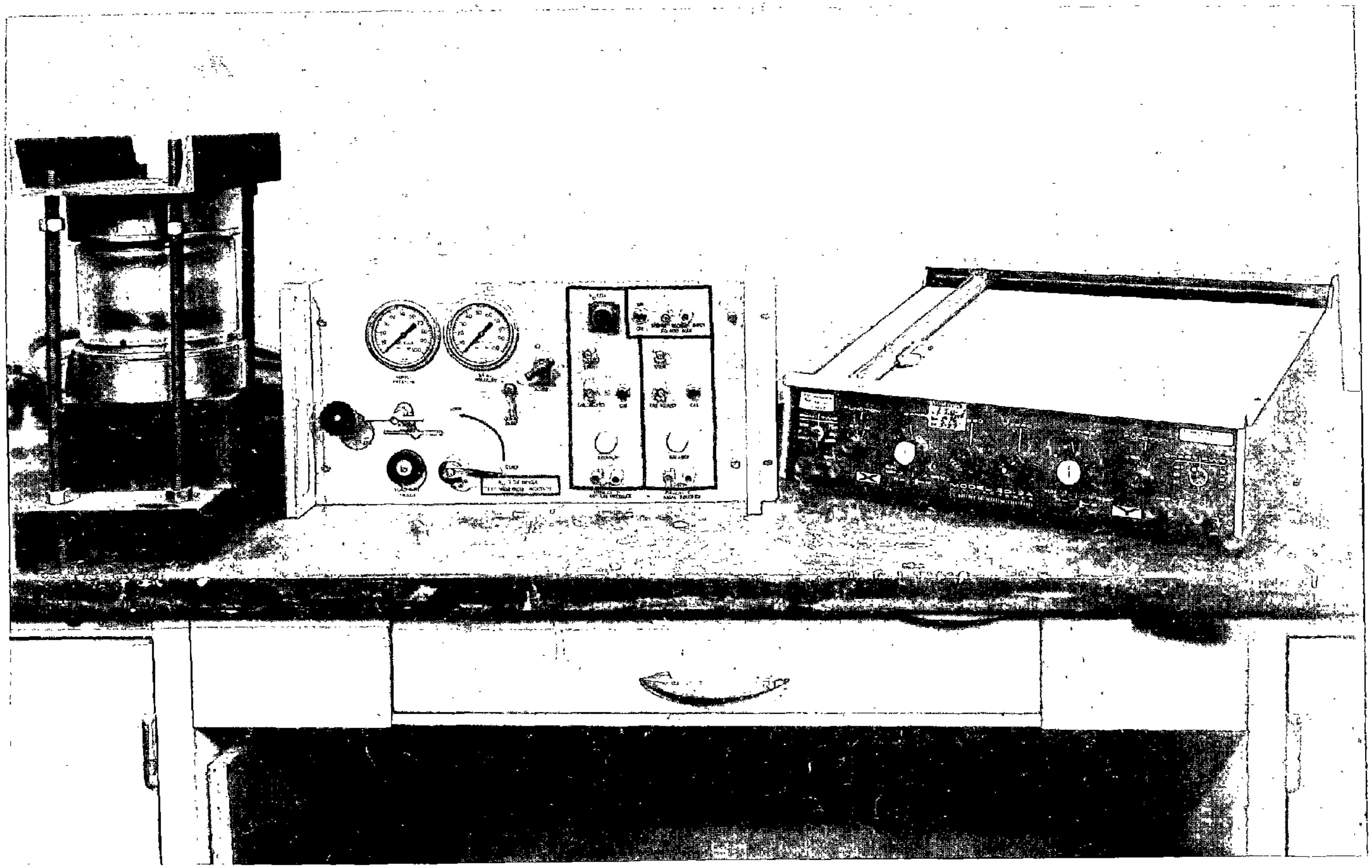

Figure 29. K_o Test Hardware

A.1.3 Bridge Circuitry

The bridge circuitry required to drive the two signal channels is shown in Figure 30. This circuitry housed in a 5 by 8 minibox mounted in the control package. Each channel has separate span control calibration pots.

The axial pressure signal is provided by a commercial strain gage type pressure transducer. The one selected for this unit is a BLH model DHF 350 psi strain-gage transducer that operates at up to 15 VDC input.

The lateral stress signal is provided by foil strain gages mounted on the K_o cell body. To obtain high signal levels at low strains, a large area, high resistance strain gage driven at higher than standard voltages is used. We selected the Micro-measurements 350 Ω 250 TB gage pattern on the basis of off-the-shelf availability at the time the K_o cell was fabricated. Any high resistance gage with equal or greater power dissipation would work equally well. Four gages were assembled on the K_o cell body, oriented to sense circumferential strain and connected in two series pairs on opposite arms of an equal arm Wheatstone bridge. Precision resistors within the balance package completed the bridge. This 750 Ω configuration can be driven at 10 to 20 VDC without stability problems.

The balance circuitry includes adjustable calibration pots and momentary contact switches for both channels. These were set at a signal level of 60 psi, (4.1×10^5 Pa) and calibration tics shown on all data. All data were recorded by X-Y recorders.

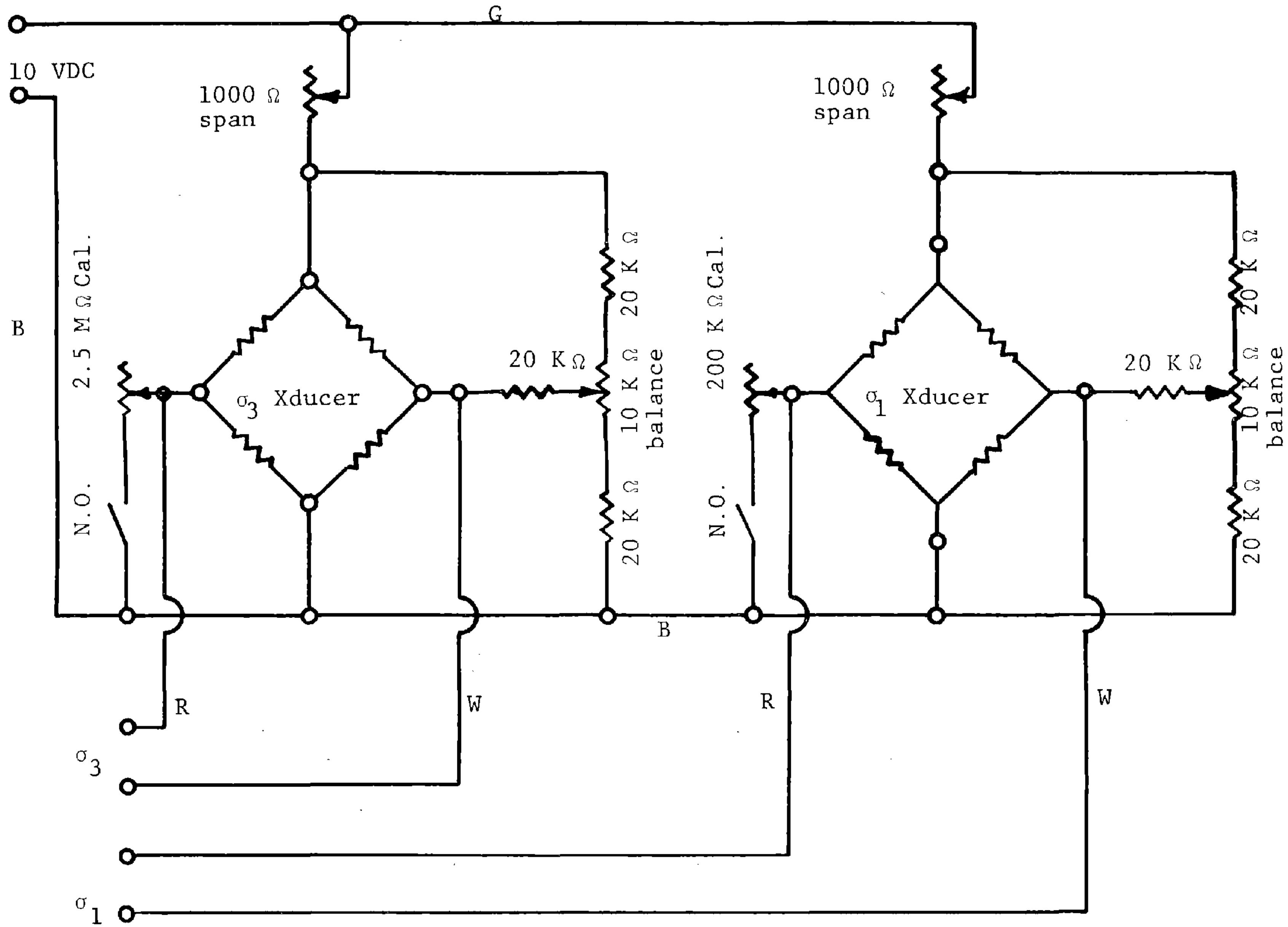

Figure 30. Balance Circuitry

A.2 Test Method

A.2.1 Specimen Preparation

Test specimens are prepared in accordance with AASHTO T-99 (ASTM D-698) with the following exceptions:

a) Soil is not dried from its as-received condition except for that degree of air-drying that might be necessary to reach desired test moistures.

b) Soil is not scalped. Aggregate larger than No. 4 is retained in the specimen.

The soil is mixed in an appropriate mixer that will provide mixing without degradation of aggregate. Mixers used on this project includes a Zee blender for soil having 100 percent passing the No. 40, a Muller for cohesive soils with aggregate and a Hobart blender for cohesionless soils.

A.2.2 Test Protocol

The K_o test device is designed for use on specimens compacted in a standard 4 inch (102 mm) compaction mold. If moisture-density specimens are to be prepared for another test method, the test protocol below should be entered at step d.

a) Place 4.5 lb (2 Kg) of soil in the appropriate mixer. On the basis of experience, estimate the amount of water necessary to bring the soil to a moisture content that will provide a K_o between 0.4 and 0.8. Add water in even 50 or 100 ml. increments, recording the amount added.

b) Mix the soil and water to a uniform state in the most appropriate mixer. The mixing conditions should be uniform and representative of field conditions for all specimens.

c) Compact the soil in a standard 4 inch (102 mm) mold in three layers as per T-99. After compaction, trim the top, weigh soil and mold for computation of compacted wet density.

d) Remove the compacted soil intact from the compaction mold, place it in a standard triaxial membrane, and slip the K_o Cell Body over the specimen. If the soil is very wet, some care is required. Some soils can be handled in this fashion even at moisture contents above the liquid limit. When specimen and membrane are in the body, roll the protruding triaxial membrane down around the outside of the cell body.

e) Assemble the cell body containing the specimen into the loading heads and reaction frame. If the soil is very wet, this is done by placing the bottom loading cap face down on top of the cell body and soil specimen. The cell body, soil sample and bottom loading head are then inverted as a unit, placing them right side up in the reaction frame. The upper loading head may then be placed on top, and the reaction frame closed.

f) Prepare for testing by putting a new sheet in the recorder and marking calibrations for both lateral stress and axial stress. It is convenient to adjust the recorder gain so that the stress levels may be read directly in pounds.

g) Pen-down the recorder and turn the selector valve to load. The pressure bleed should be set at about 2 psi (14×10^3 Pa) per second. When the pressure reaches 60 psi by the gage, switch the selector valve to unload.

note: It is necessary to exceed the unconfined compressive strength to measure K_o. For soils having strengths above 60 psi, (414×10^3 Pa) permit the pressure to continue increasing until X-Y recorder clearly indicates failure has occured. Subsequent cycles will also be run at high pressure. Because K_o is essentially constant as load increases, the exact pressure reached is less important than the measured stress ratio.

h) Index the recorder to a new location on the paper and reload-unload. In all, 5 loading cycles are taken, each recorded separately. During this time, the experimenter should mark the test ID on the recorder data, clean and assemble the compaction mold, weigh the next specimen and begin its mixing cycle. The moisture added should be changed by 50 to 100 ml. after estimating the approximate K_o value displayed by the specimen being tested.

i) Following cycle 5, unload the specimen, weigh out moisture content samples and put them in an oven for drying. During this period, the next specimen is mixing.

j) Continue testing until at least three specimens having K_o between 0.4 and 0.8 have been tested.

k) K_o is computed by striking a tangent to the X-Y record of the fifth cycle and taking the slope. The device as built has a baseline reading of 4.2 psi (29×10^3 Pa) at an axial stress of 60 psi (414×10^3 Pa). Thus K_o is given by

$$K_o = \frac{R_{60} - 4.2}{60 - 4.2} = \frac{R_{60} - 4.2}{55.8}$$

where R_{60} is the lateral stress reading at an axial stress of 60 psi (414×10^3 Pa).

The base line reading is caused by the seal ring which bears on the K_o cell body, transmitting axial stress which is picked up by cross-sensitivity effects.

The K_o data is presented as a plot of K_o vs. moisture content.

The K_o test as described can be run by one man at the rate of 16 tests per day. Two men working together can more than double this rate. The time to produce a four point K_o-moisture content curve is two hours. One man-year is approximately equal to determinations of moisture-density and moisture-K_o for 800-1000 soils, neglecting down-time for extraneous events.

A.2.3 Calibration

The K_o device is calibrated for hydrostatic and perfect-rigid conditions. Hydrostatic calibration is obtained by permitting the pressurizing air to enter the K_o cell body when empty. The drum-head sealing membrane on at least one loading head is punctured or removed for this purpose. Use reduced flow (load bleed control) to avoid measuring head loss in the lines between the pressure transducer and the K_o device as the air rushes into the empty K_o cell body. The instrumentation easily reads fractions of a pound per square inch.

A.3 Possible Modifications

Several modifications might be recommended for future models of the K_o device.

a) The foil strain gages might be replaced by semiconductor strain gages having higher sensitivity. This would permit thicker cell body walls and lateral strains even lower than the current 1 microinch per psi.

b) The brass seal rings were designed to bear on the ends of the cell body because it was felt that situations requiring flush cell bottom ends might arise. Experience has shown that this is not required, and the cell body can be made longer than the soil specimen. The brass seal carriers can be replaced by a variety of plastic cup seals and wipers which are available off-the-shelf at low cost. The cup seal would ride inside both ends of a lengthened K_o cell body, permitting considerable reduction in the cost and complexity of the loading head assemblies.

c) The K_o cell itself could be designed as a compaction mold, and the soil compacted directly therein. The lubricated membrane could not be used, and great consideration of wall shear effects would be required before making this step. Tests with granular materials would scratch the interior surfaces in the absence of a membrane, but a small fly-hone would easily buff these out. Finally, the strain gages might require occasional replacement if the cell body is used as a compaction mold.

12063 *U.S. GOVERNMENT PRINTING OFFICE : 1977 O-730-871/1694