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CORRELATION OF RESISTANCE VALUE (R-VALUE) WITH CALIFORNIA BEARING
RATIO (CBR) FOR USE IN THE DESIGN OF FLEXIBLE PAVEMENTS

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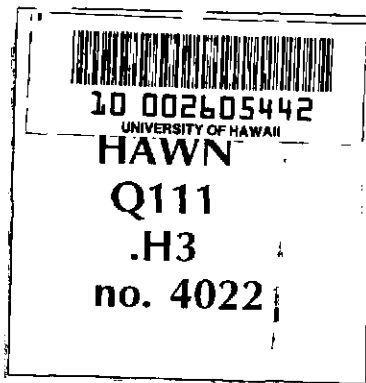
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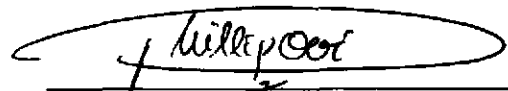
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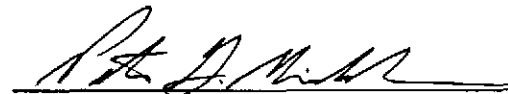
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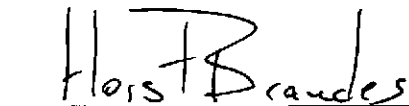
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ABSTRACT

The Resistance Value (R-value) is commonly used by the Hawaii Department of Transportation engineers to design the thickness of flexible pavements. Direct measurements of the R-value require equipment that is not readily available to most practicing engineers in the State of Hawaii. Typically, the R-value is indirectly based on the results of the California Bearing Ratio (CBR) tests. Knowing the CBR, the R-value is estimated based on published correlations. However, these correlations were established for soils outside the State of Hawaii. Moreover, these correlations were not established for directly relating R-value and CBR, but rather for estimating other parameters such as resilient modulus, soil support value or modulus of subgrade reaction.

CBR, R-value and index tests were performed on tropical residual soils from four locations on the island of Oahu in the state of Hawaii. Based on the test results, five correlations were developed to estimate the R-value. Among these procedures is one relating R-value to index properties alone, without reference to the CBR value. The limitations of each procedure and the choice of method are discussed.

Some tropical residual soils can undergo irreversible changes upon drying. One of the soils sampled had a relatively high natural water content. As a secondary objective, this soil was tested at three different stages of drying: first at its natural or in-situ state, second after oven drying the soil and third after drying the soil to approximately half its natural water content (intermediate). This material can be regarded as three different soils corresponding to the various stages of drying.

The CBR and R-value were observed to increase from the in-situ to the oven-dried samples. The oven-dried samples were excluded from the correlations described above because these soils were dried to temperature extremes that regular soils do not experience, and therefore, are judged to be inappropriate for inclusion in the correlations. The intermediate samples were included in the correlations because soils used as fill material may undergo some drying prior to compaction in the field.

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CHAPTER 1 INTRODUCTION

The resistance value or R-value is used by the Hawaii Department of Transportation (HDOT) engineers to design the thickness of flexible pavements. Direct measurements of the R-value require testing equipment that is not available to most engineers in Hawaii. The typical engineering practice in Hawaii is to estimate the R-value indirectly based on the results of the California Bearing Ratio (CBR) test. Knowing the CBR, the R-value is estimated based on published correlations. These correlations were not established to directly relate R-value and CBR. Rather, they were meant for estimating other parameters such as the resilient modulus, soil support value or modulus of subgrade reaction based on the R-value or CBR. Moreover, these correlations were established for soils outside of the State of Hawaii.

Some tropical residual soils found in Hawaii have been known to exhibit different characteristics than soils from temperate regions on the U.S. continent. According to Mitchell and Sitar (1982), tropical residual soils including those found in Hawaii are likely to be less dense, less plastic, less compressible, stronger and more permeable than temperate soils of comparable liquid limit. One complication to this research program is that tropical soils rich in halloysite can undergo irreversible changes upon drying. Halloysite consists of alternating kaolinite unit cells and one layer of water molecules resulting in a much weaker bond between the kaolinite units (Mitchell and Sitar, 1982). As weathering proceeds, the halloysite content decreases and the kaolinite content increases. The halloysite particles are characterized by a tubular morphology. As a result of heating or air-drying, the water layer in the halloysite is removed irreversibly, i.e., the material will not rehydrate to its former amorphous state. The addition of water

to the dehydrated sample will result in different properties than the same undried soil of equal moisture content. Since this study involved testing tropical soils, every effort was made to preserve the moisture of the soil samples prior to testing. In the event that drying of the soil is required during testing (e.g., during compaction), the soil was tested from wet to dry.

1.1 Objectives

The objectives of this research program included the following:

1. Conduct a literature search to identify existing correlations that link CBR with R-values. Correlations between R-value and other soil parameters were also included in the search.
2. Perform a series of CBR tests and R-value determinations for tropical residual soils. CH soils were not considered in this research program because they are typically not used as subgrade material. Instead, ML and MH soils, which are more commonly found on the Hawaiian islands, were tested in this research program.
3. Perform soil index tests on these soils.
4. Verify that the previously established correlations between CBR and R-value apply to local soils. If not, propose a correlation between CBR and R-value for the soils tested. Perform a study to see if R-values can be correlated to other soil parameters.
5. Develop a database of R-values for the soils tested in the State of Hawaii.

The CBR and R-value tests are briefly reviewed in Chapter 2. A literature review of correlations between CBR and R-value is also included in this chapter. In Chapter 3, the soil sampling locations and results of the soil index tests are presented. The CBR and R-value test results are contained in Chapters 4 and 5, respectively. Correlations developed to estimate the R-value and a discussion on the choice of methods are covered in Chapter 6. To conclude in Chapter 7, a summary of the work, recommendations on the correlations and suggestions for future work are described.

CHAPTER 2 LITERATURE REVIEW

The principles of the CBR and R-value tests are first briefly described.

2.1 CBR Test

The equipment and test procedure are detailed in AASHTO T 193-99 and ASTM D1883-99. There are three stages in a CBR test. First, the specimen is dynamically compacted in a 6-inch (152.4 mm) diameter mold. Second, the specimen is soaked for 4 days with a surcharge load applied. Soaking the sample simulates the worst-case moisture scenario in the field and the surcharge simulates the overburden due to the pavement. Third, with the same surcharge load applied, a standardized piston having an area of 3 in² (19.4 cm²) is used to penetrate the soil in the mold at a rate of 0.05 inch per minute (1.27 mm per minute) (Figure 2.1). Generally, the load at 0.1-inch (2.54 mm) penetration is used to compute the CBR. The CBR is defined as the ratio of the stress at 0.1-inch (2.54 mm) penetration to that of a standard value. Standard values correspond to those for a high-quality crushed stone and are summarized in Table 2.1.

Table 2.1 Standard load for high quality crushed stone material

Penetration (inch)	Standard Load for Crushed Stone (psi)
0.1	1000
0.2	1500
0.3	1900
0.4	2300
0.5	2600

Note: Conversion factors: 1 in = 25.4 mm and 1 psi = 6.895 kPa.

If the CBR at 0.2-inch (5.08 mm) penetration is found to be higher than at 0.1-inch (2.54 mm) penetration, the load value at 0.2-inch (5.08 mm) penetration is used if another test confirms a similar result.

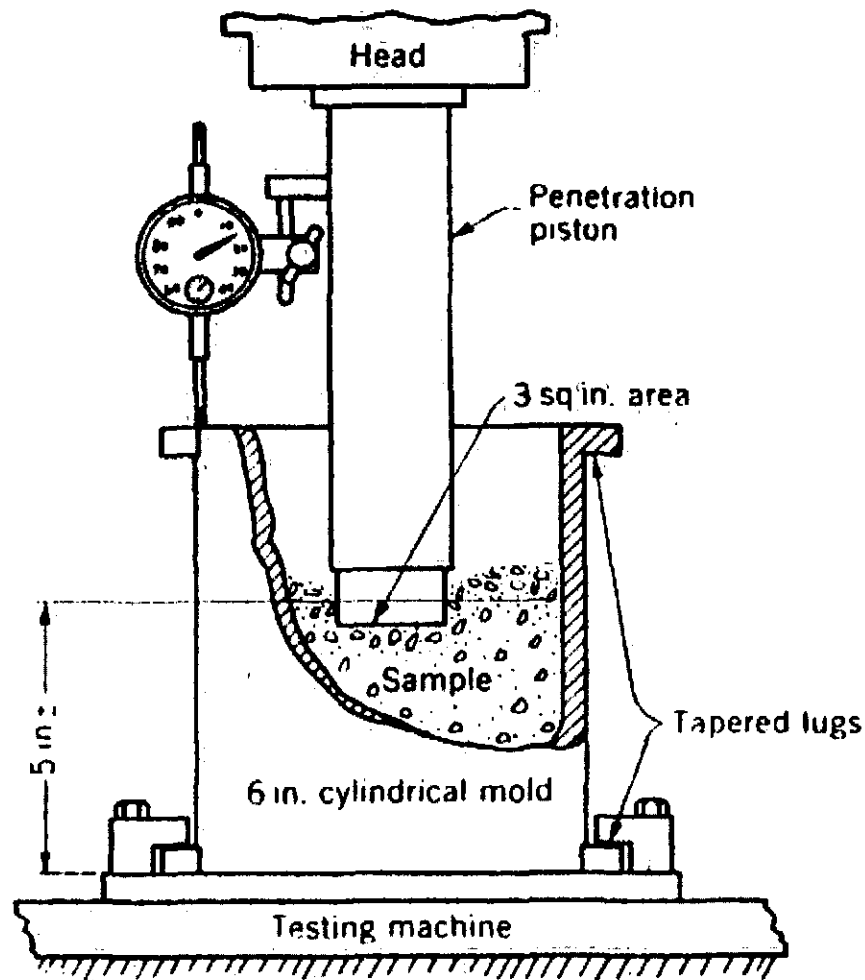


Figure 2.1 Penetration portion of CBR test (Porter, 1949)

2.2 R-value Test

The equipment and test procedure are described in AASHTO T 190-02 or ASTM D2844-01. There are four stages in the R-value test. First, the specimen is compacted in a 4-inch (101.6 mm) diameter steel mold using a kneading compactor, which alternately applies and releases a pressure of 350 psi (2,413 kPa) during the last 100 tamps. Second, the specimen is loaded in a steel mold with a testing press until enough moisture squeezes out of the specimen to light up five of six bulbs on an exudation indicator device. Third, the specimen is soaked for 24 hours. Fourth, the specimen is placed in a

Hveem stabilometer (Figure 2.2) to measure the R-value. This device consists of a cylindrical shell, which has a portion of the inside walls hollowed out and a neoprene rubber diaphragm fixed in position. The annulus behind the diaphragm is filled with a hydraulic fluid and is connected to a pressure gauge. When the 2.5-inch (63.5 mm) high x 4-inch (101.6 mm) diameter sample is loaded from the top, the portion of the vertical load transmitted by the specimen to the liquid annulus can be read on the gauge. The resistance offered by the soil is expressed as a function of the ratio of the lateral transmitted pressure to the vertical pressure of 160 psi (1,103 kPa) applied with a testing press. This ratio provides an indication of the resistance to plastic flow, arranged on a linear scale of 0 to 100. In its simplest form, the R-value is defined as:

$$R = \left(1 - \frac{P_h}{P_v} \right) 100 \quad (2.1)$$

where P_h and P_v are horizontal and vertical pressures, respectively. The lateral pressure varies inversely with the internal resistance of the soil. For example, an R-value of 100 indicates a material that does not deform under the vertical load. On the other hand, an R-value of 0 indicates that the material offers no shear resistance and behaves like a liquid.

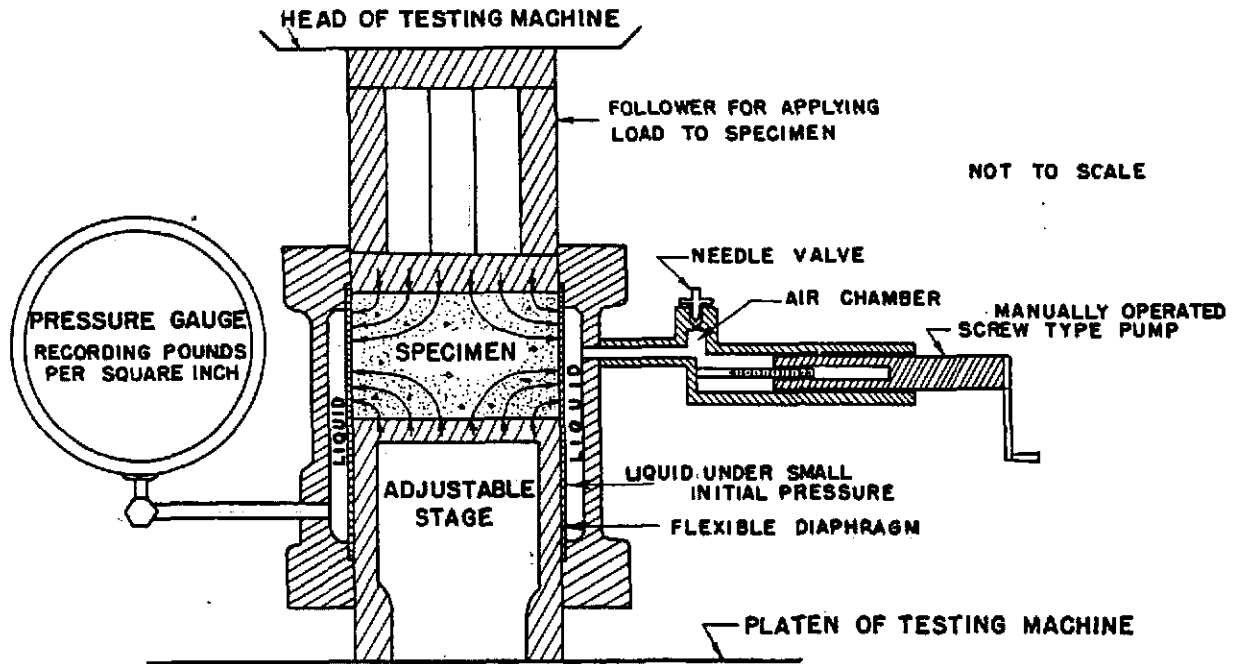


Figure 2.2 Schematic of a Hveem stabilometer (Howe, 1961)

2.3 Correlations between CBR and R-value

In the design of flexible pavements, the value of CBR after construction is desired. If the subgrade soil is subjected to moisture changes, knowledge of the CBR after undergoing a change in the field moisture content is desirable. To facilitate this, a relationship between the moisture content, dry unit weight and CBR is needed. This involves preparing CBR samples at a range of moisture contents and dry unit weights, i.e., developing a "family of curves." Unlike the CBR, the R-value test data do not directly permit selection of field compaction conditions. The R-value test is measured over a range of exudation pressures by varying the moisture content. The R-value specimens are usually prepared wet of optimum (Asphalt Institute, 1982) because of the uniqueness of the exudation portion of the test, where the soil is loaded until moisture is squeezed from the soil specimen. This pressure is called the exudation pressure. The design R-

value is selected based on a value of exudation pressure that best represents the worst condition likely to be reached in place within the subgrade several years after construction (Howe, 1961). The representative exudation pressure is a function of several factors including soil type, climate, drainage and highway construction conditions. For a given soil, the R-value varies with exudation pressure. An exudation pressure of 240 psi (1,655 kPa) is used in California while an exudation pressure of 300 psi (2,068 kPa) is adopted in Washington and Hawaii. As a result of this difference between the CBR and R-value, it is important to know not only the correlation but also under what conditions are the correlations applicable; e.g., CBR at optimum based on Standard Proctor versus R-value at exudation pressure of 240 psi (1,655 kPa).

Direct correlations between CBR and R-value are not commonly available. Often, CBR and R-value are individually correlated with (1) resilient modulus, (2) soil support value or (3) modulus of subgrade reaction. The correlations between CBR and R-value described below are indirectly derived by combining two relationships: one between CBR and say, the resilient modulus and the other between R-value and the resilient modulus. By eliminating the resilient modulus, the relationship between CBR and R-value is obtained. An extensive literature review was performed but no recent literature was found that provided correlations between the CBR and R-value.

2.3.1 Liddle et. al. (1967)

The Utah State Department of Highways Materials and Tests Division proposed the correlation between the R-Value at two exudation pressures, three types of CBR (dynamic, static and AASHTO 3 point) and the soil support value (Liddle et al., 1967) as shown in Figure 2.3. This correlation was later adopted in AASHTO's (1976) Interim Guide for Design of Pavement Structures.

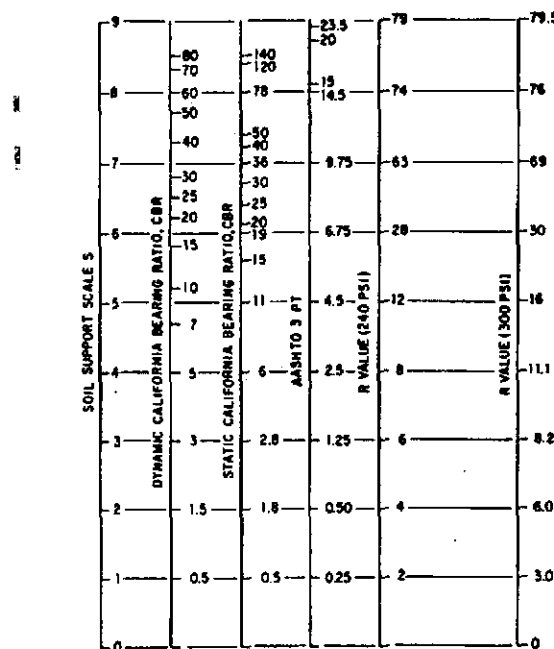


Figure 2.3 Correlation chart for estimating soil support (Liddle et al., 1967)

The various CBR tests differ in the method of compacting the test specimens. Static compaction involves compacting the samples with a static compression load, while the dynamic and the AASHTO 3 point test utilizes a vertical moving rammer dropped onto the sample. The compaction method affects the soil structure in fine-grained soils and therefore, the strength and stiffness characteristics (Seed et al., 1960). The current

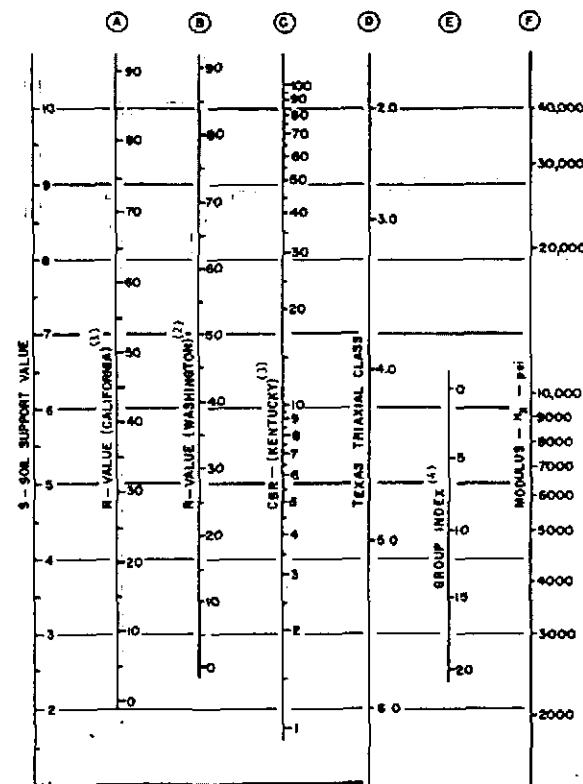
AASHTO T 193-99 and ASTM D1883-99 specifications require dynamic compaction of test specimens.

Liddle et al. (1967) performed CBR and R-value tests on four Utah soils. CBR tests were performed over a wide range of dry unit weights and moisture contents. The resulting CBR is widely variable and the mean was used in the correlation. This methodology was repeated for each soil sample. The CBR obtained in this fashion is not specific to a particular combination of moisture content and dry unit weight. Multiple R-value tests were performed to determine the R-value at exudation pressures of 240 and 300 psi (1654 and 2068 kPa). Each mean CBR and R-value for the four soils was then correlated to the soil support value by first determining its equivalent dynamic CBR. This dynamic CBR value was then correlated to the soil support number using a previous relationship that was obtained from the Utah State Material's Manual. AASHTO (1972) cautioned against using this correlation for soils other than those found in Utah.

2.3.2 Van Til et al. (1972)

Van Til et al. (1972) indicated that “the vertical compressive strain on the subgrade was the most significant factor affecting the performance of the roads at the AASHO Road Test.” They also recognized the importance of layer theory and wanted to develop a rational approach to estimate the soil support value based on the resilient modulus. They proposed a correlation between the soil resilient modulus, R-value (at 240 and 300 psi (1654 and 2068 kPa) exudation pressures) and the CBR as shown in Figure 2.4. In this chart, the CBR is measured in accordance with the method proposed by Drake and Havens (1959). Developed in Kentucky, this method requires that the soil specimen be molded at or near the optimum moisture content as determined by the standard Proctor

(AASHTO T 99-01 or ASTM D698-91) compaction test. Then the soil is compacted in a CBR mold using dynamic compaction, where a 10-pound (4.54 kg) hammer is dropped from a height of 18 inches (46 cm). The soil is compacted in five equal layers with each layer receiving 10 blows. The soil is then soaked for 4 days prior to testing. Note that the Kentucky CBR test procedure is not the same as the CBR test in AASHTO T 193-99 or ASTM D1883-99. This chart was adopted in the 1986 AASHTO Guide for the Design of Pavement Structures.



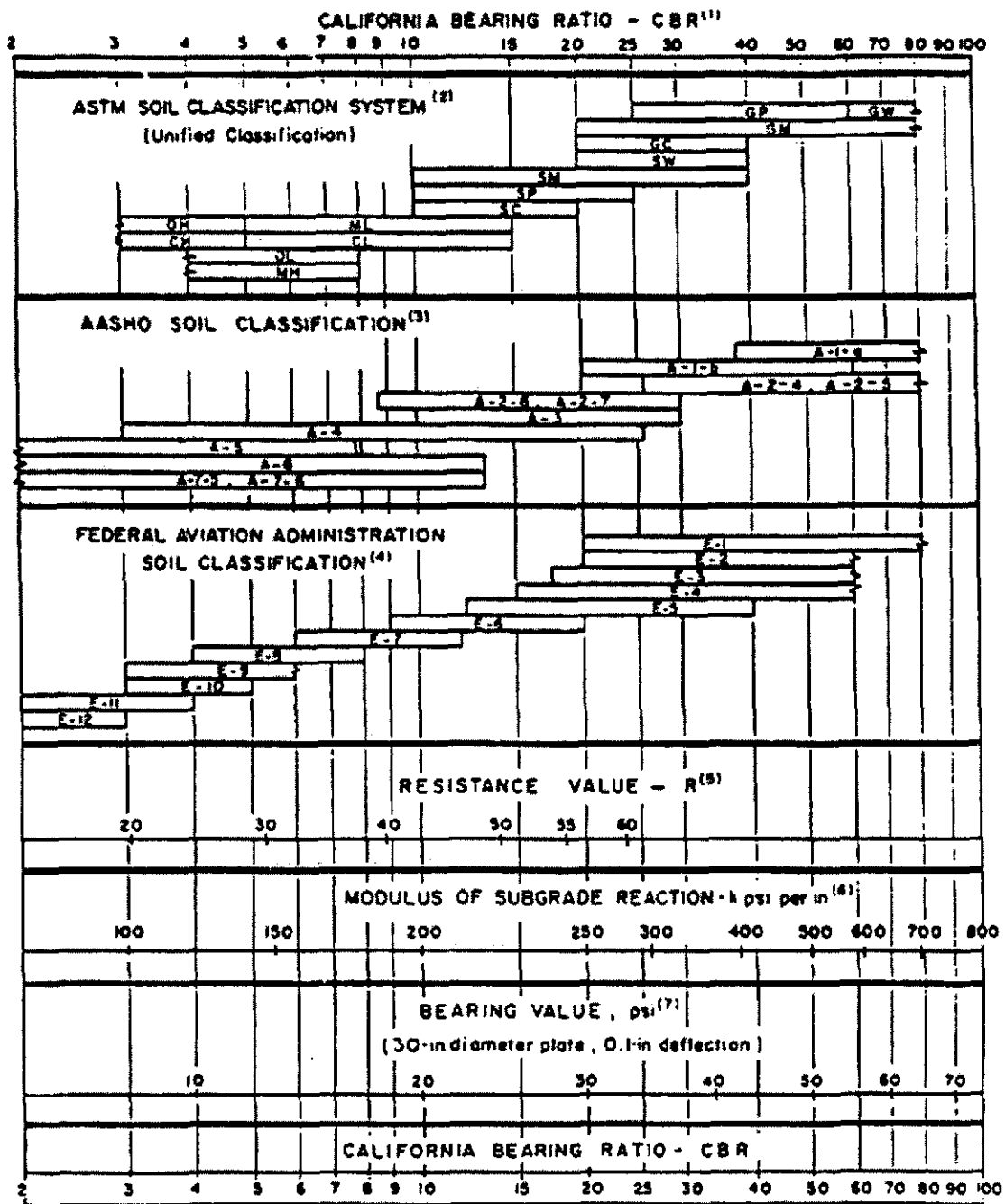
- (1) The correlation is with the design curves used by California: AASHTO designation is T-173-40, and exudation pressure is 240 psi. See Eves, F.M., and Garmann, R.M., "The Factors Underlying the Rational Design of Pavements," *Proc. HRS*, Vol. 28 (1948) pp. 101-136.
- (2) The correlation is with the design curves used by Washington Dept. of Highways; exudation pressure is 300 psi. See "Flexible Pavement Design Correlation Study," *HRB Bull.* 133 (1956).
- (3) The correlation is with the CBR design curves developed by Kentucky. See Drake, W.B., and Havens, J.R., "Re-Evaluation of Kentucky Flexible Pavement Design Criterion," *HRB Bull.* 222 (1959) pp. 13-56. The following conditions apply to the laboratory-modified CBR: specimen is to be soaked at or near the optimum moisture content as determined by AASHTO T-99; dynamic compaction is to be used with a hammer weight of 10 lb dropped from a height of 18 in.; specimen is to be compacted in five equal layers with each layer receiving 10 blows; specimen is to be soaked for 4 days.
- (4) This scale has been developed by comparison between the California R-value and the Group Index determined by the procedure in *Proc. HRS* Vol. 25 (1945) pp. 376-392.

Figure 2.4 Correlation chart for estimating soil support (Van Til et al., 1972)

2.3.3 Packard (1984)

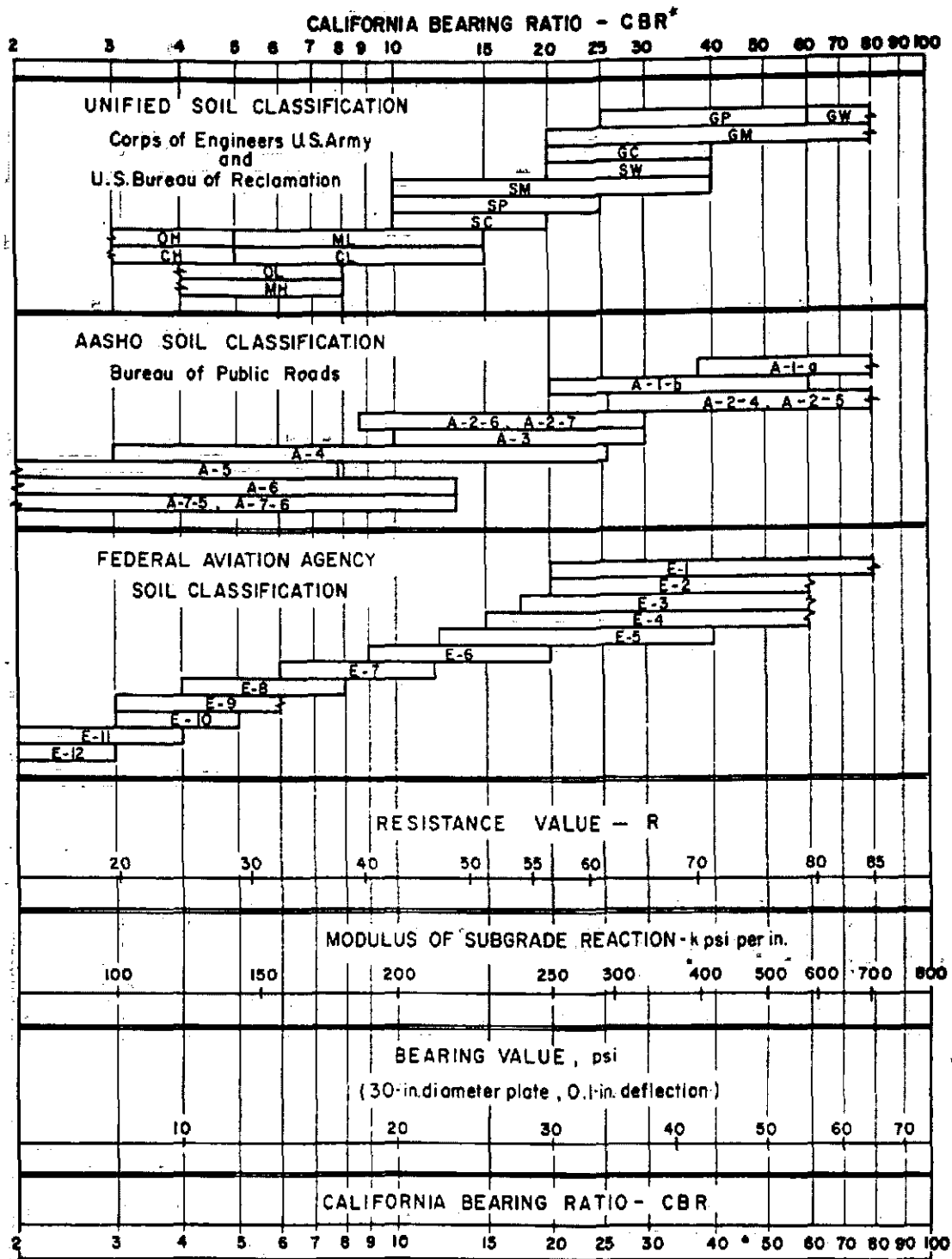
Based on unpublished data of R-value varying with the modulus of subgrade reaction, and based on the relationship between the modulus of subgrade reaction and CBR, Packard (1984) developed an empirical chart where the soil R-value may be indirectly obtained from CBR test results (Figure 2.5). An earlier version of this chart was published by the Portland Cement Association in 1966, a copy of which is shown in Figure 2.6. Since the soil strength parameters were related to modulus of subgrade reaction, these charts were developed for use in the design of rigid or concrete pavements. These charts are useful in that they provide CBR and R-values for various soil types. However, the soil physical state (combination of water content and dry unit weight) at which the CBR is based on was not provided.

A recent publication by Hall et al. (1997) indicated that while there is a noticeable relationship between the modulus of subgrade reaction and CBR, there is little or no correlation between the modulus of subgrade reaction and R-value. Therefore, Packard's correlation, if used to relate R-value to CBR, should be used with caution.



- (1) For the basic idea, see O.J. Porter. "Foundations for Flexible Pavements." Highway Research Board Proceedings of the Twenty-second Annual Meeting. 1942, Vol. 22, pages 100 - 136.
- (2) ASTM Designation D2487.
- (3) "Classification of Highway Subgrade Materials." Highway Research Board Proceedings of the Twenty-fifth Annual Meeting. 1945, Vol. 25, pages 376-392.
- (4) Airport Paving. U.S. Department of Commerce, Federal Aviation Agency. May 1948. pages 11-16. Estimated using values given in FAA Design Manual for Airport Pavements (Formerly used FAA Classification. Unified Classification now used.)
- (5) C. E. Warnes. "Correlation Between R Value and k Value." Unpublished report. Portland Cement Association. Rocky Mountain-Northwest Region. October 1971 (best fit correlation with correction for saturation)
- (6) See T.A. Middlebrooks and G.E. Bertram. "Soil Tests for Design of Runway Pavements." Highway Research Board Proceeding of the Twenty-second Annual Meeting. 1942. Vol. 22, pages 152.

Figure 2.5 Soil classification related to strength parameters (Packard, 1984)



*Sources for values shown are given in PCA Soil Primer

Figure 2.6 Earlier version of Figure 2.5 (Portland Cement Association, 1966)

2.3.4 Equations Relating CBR and R-value to Resilient Modulus

The R-value can also be indirectly related to the CBR using an equation relating the resilient modulus and CBR, and using an equation relating the resilient modulus and R-value. A relationship between the resilient modulus of the soil and the CBR value was proposed by Heukelom and Klomp (1962) as follows:

$$M_r = 1500\text{CBR} \quad (2.2)$$

where M_r = resilient modulus of the soil in psi. Figure 2.7 illustrates the data used to develop the correlation. The constant of proportionality of 1500 can vary quite considerably from 0.5 to 2 times that amount. Heukelom and Klomp (1962) obtained field measurements of the resilient modulus based on vibratory loading.

A relationship between the resilient modulus and the R-value was derived from data collected in the San Diego County Experimental Base Project (Asphalt Institute, 1982) as follows:

$$M_r(\text{psi}) = 772 + 369R \quad (2.3)$$

This equation was subsequently revised to the following (Asphalt Institute, 1982):

$$M_r(\text{psi}) = 1155 + 555R \quad (2.4)$$

By equating the right-hand sides of equations 2.2 and 2.4 to eliminate M_r , the R-value can be related to the CBR as follows:

$$R = \frac{1500\text{CBR} - 1155}{555} \quad (2.5)$$

AASHTO (1993) indicated that equations 2.2 and 2.4 are valid for a limited range of CBR and R-values, respectively. Equation 2.2 is valid for CBR values of less than about 10, while equation 2.4 is valid for R-values of less than about 20.

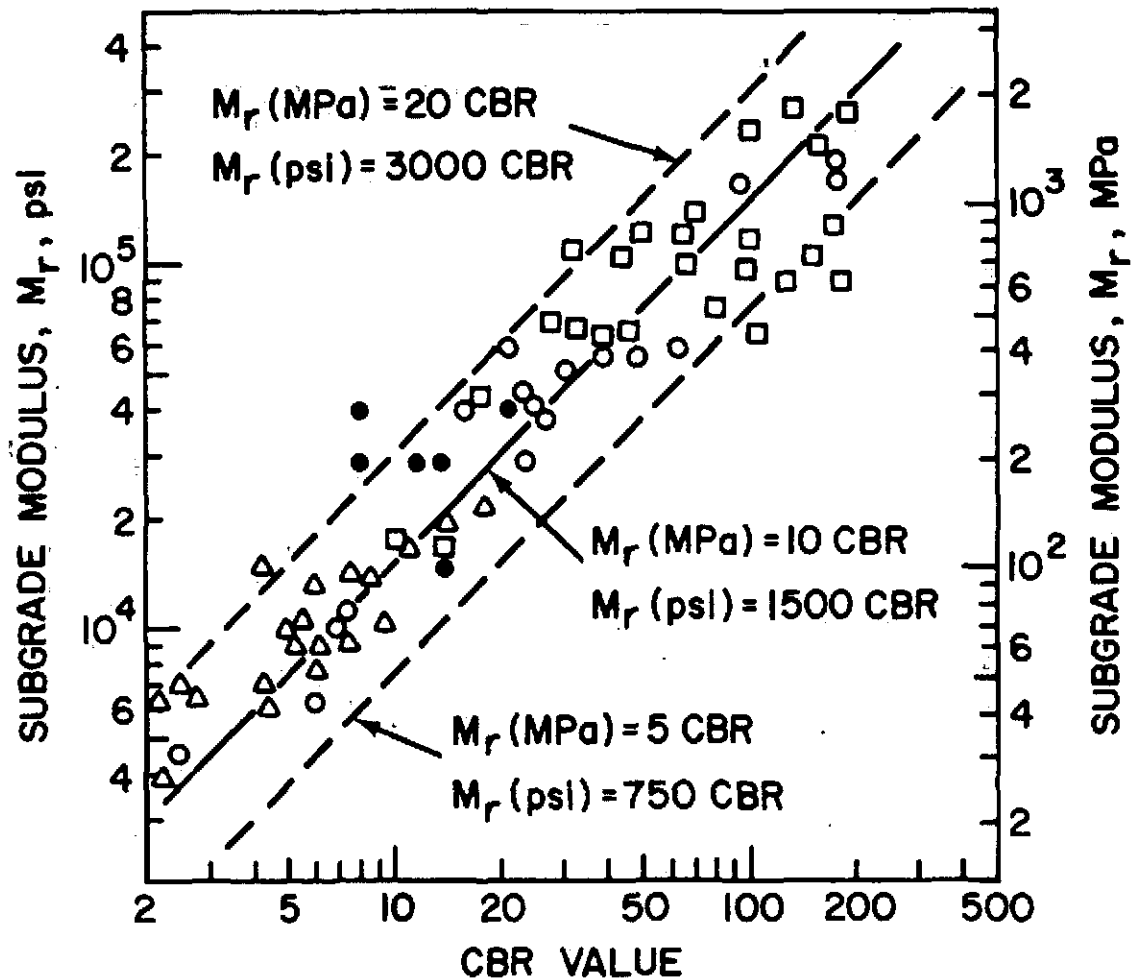


Figure 2.7 Resilient modulus as a function of CBR (Heukelom and Klomp, 1962)

Another relationship between CBR and resilient modulus was proposed by Powell et al. (1984) for British soils as follows:

$$M_r(\text{psi}) = 2552 \text{CBR}^{0.64} \quad (2.6)$$

In the U.K., the CBR is measured on samples that are prepared at the dry unit weight and water content that are likely to be in the field without soaking (Croney and Croney, 1998). Combining equations 2.4 and 2.6, and eliminating M_r , R-value can be related to CBR as follows:

$$R = \frac{2552\text{CBR}^{0.64} - 1155}{555} \quad (2.7)$$

A comparison of the effects of using equations 2.5 and 2.7 is shown in Figure 2.8. At CBR values less than 5, the two methods yield similar R-values. When the CBR value exceeds 5, equation 2.7 is found to be more conservative.

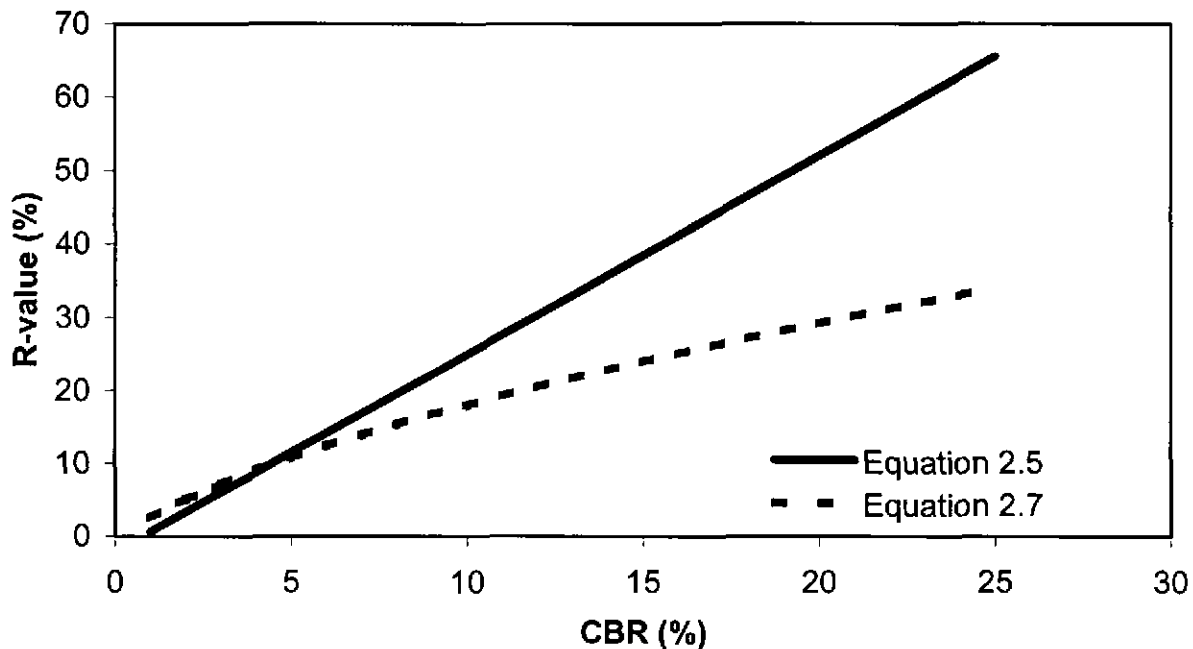


Figure 2.8 Comparison of R-value vs. CBR relationship derived indirectly from Heukelom and Klomp's (1962 – Equation 2.5) and Powell et al.'s (1984 – Equation 2.7) equations

For a given soil, the resilient modulus is a function of the soil stress state (confining and deviatoric stresses) as well as the soil physical state (water content and dry unit

weight). The CBR is a function of the surcharge loads and soil physical state. The R-value is a function of exudation pressure (which is related to the soil physical state). Correlations between the resilient modulus, CBR and R-value will be most useful if these other variables are included or addressed but yet, they are excluded in many of the correlations in the literature.

2.3.5 Correlation Between R-value and Index Properties

The R-value has been directly related to soil index properties. One such correlation is provided in Table 2.2 used by the Arizona Department of Transportation (Miyashiro, 2000) where the R-value is related to the plasticity index of the soil and the percent passing the #200 sieve.

PERCENT PASSING #200 SIEVE

19

CHAPTER 3 SOIL INDEX TESTING

3.1 Soil Sample Locations

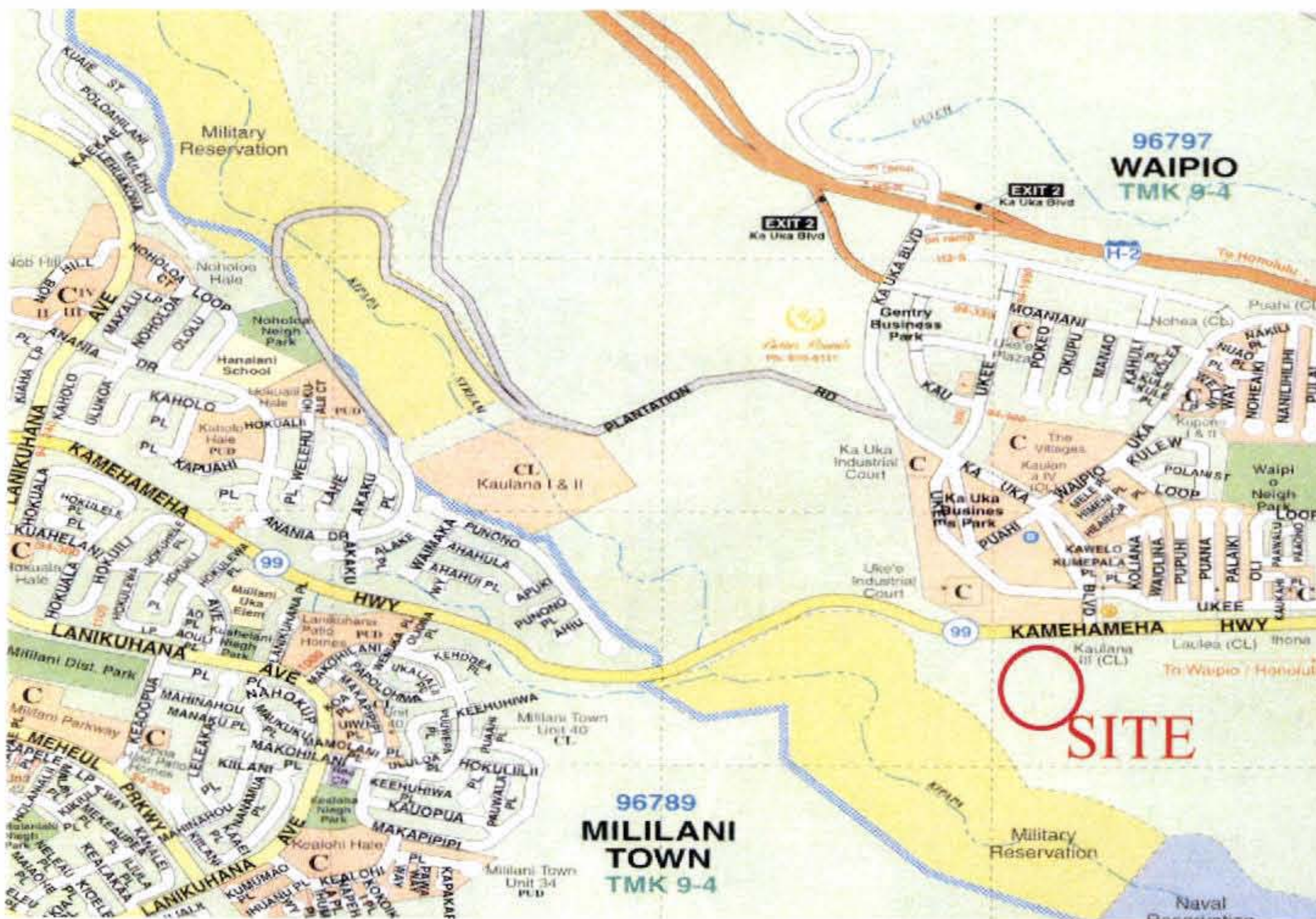
Soil samples from four different locations on the island of Oahu were collected for testing. The site locations (Figure 3.1) and the date of sampling are as follows:

- 1) Waipio - February 1, 2001
- 2) Kapolei - May 24, 2001
- 3) Mililani Mauka - September 25, 2001
- 4) Wahiawa - February 7, 2002

A trench was dug at each site to expose the less desiccated soil for sampling. To preserve the in situ moisture content, the soil samples were placed into plastic bags, which were then placed into 5-gallon plastic buckets. Prior to storage, moisture contents were recorded on the day of sampling. After heat-sealing the plastic bags, each bucket was sealed with a lid containing an O-ring, which provided a watertight seal. The buckets were then stored in a 100%-humidity curing room located in the structures laboratory in Holmes Hall at the Department of Civil and Environmental Engineering, University of Hawaii. These steps were necessary to avoid possible irreversible changes in the soil properties that could occur upon drying.

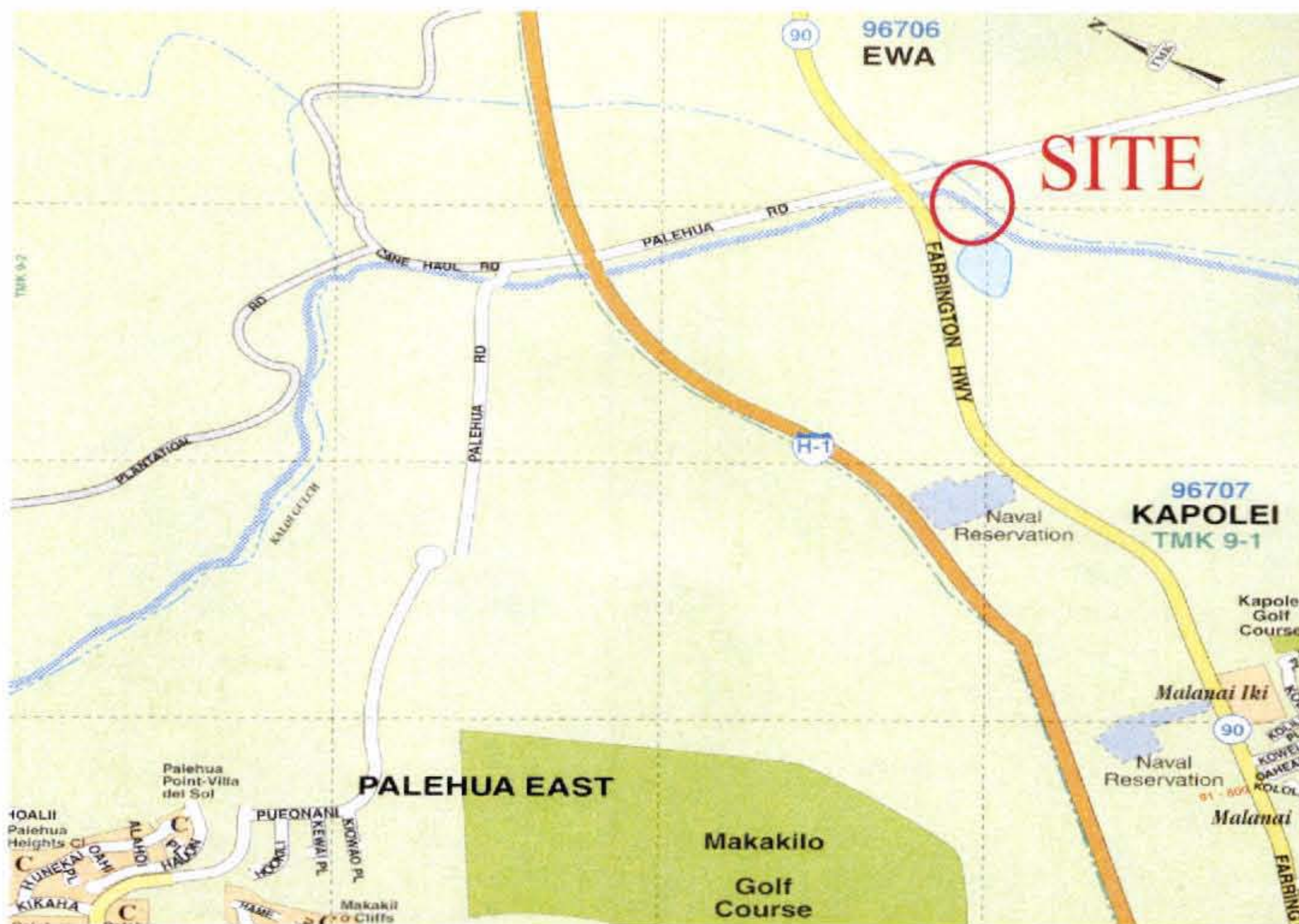


(a) Island map

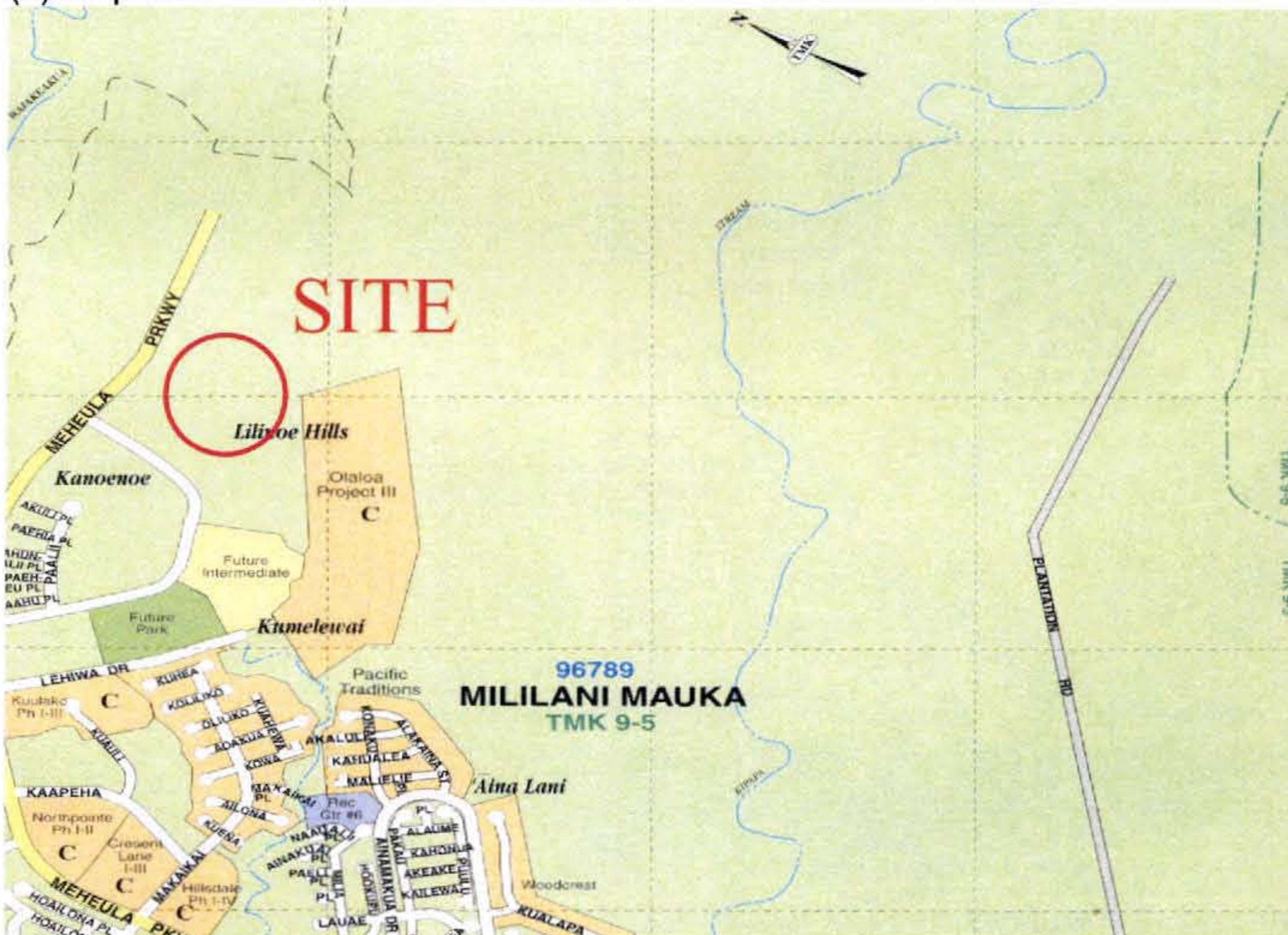


(b) Waipio

Figure 3.1 Soil sampling locations

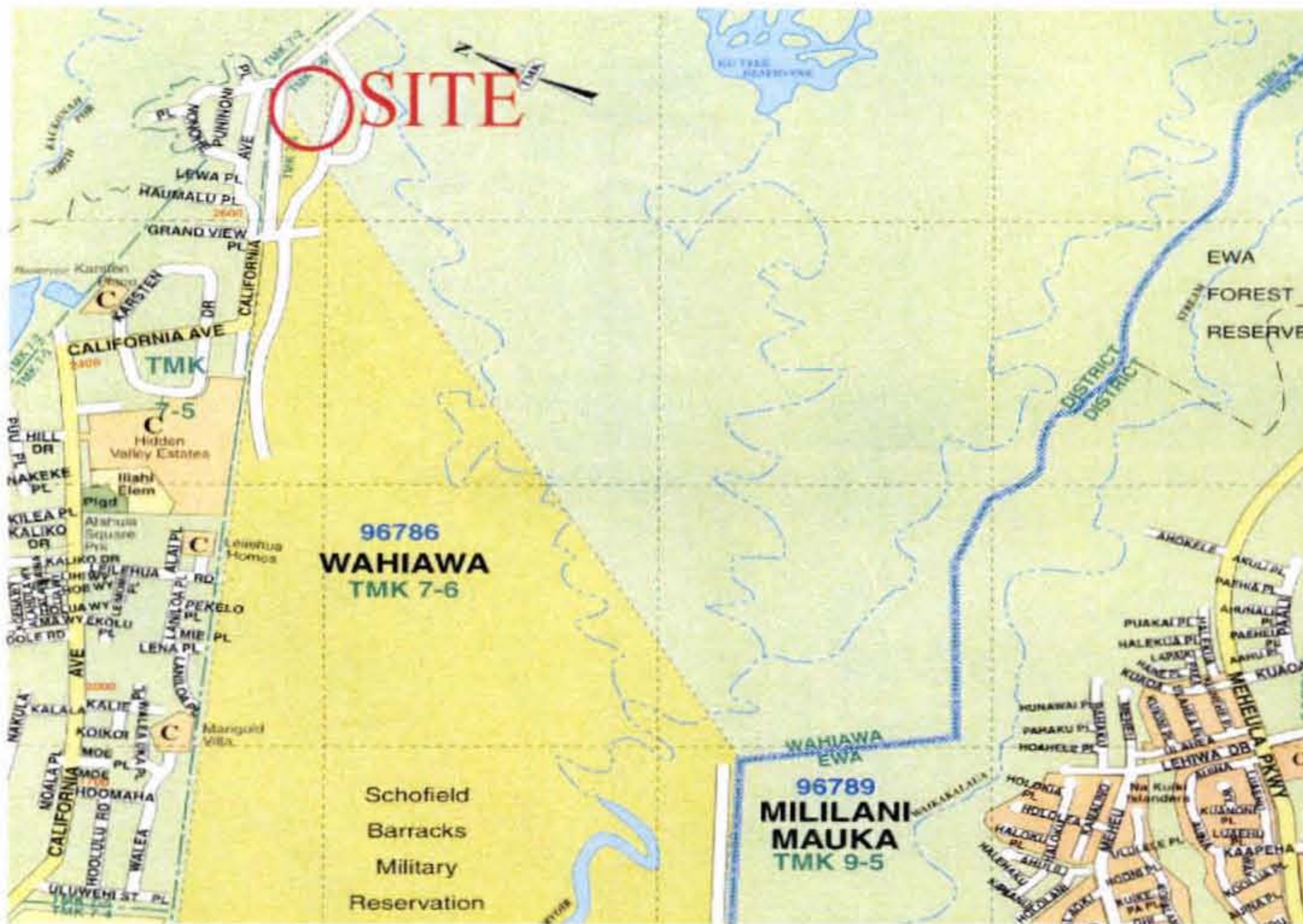


(c) Kapolei



(d) Mililani Mauka

Figure 3.1 Soil sampling locations (continued)



(e) Wahiawa

Figure 3.1 Soil sampling locations (continued)

Nuclear gauge (courtesy of Geolabs, Inc.) and sand cone testing were performed at each site to measure the in place dry unit weight and moisture contents. Only the results of the in situ moisture contents are presented herein. The in situ moisture contents are summarized in Table 3.1 and Figure 3.2.

Table 3.1 Summary of in situ water contents

Soil	Water Content ¹ (%)			
	Sand Cone		Nuclear Gage	
	Range	Mean	Range	Mean
Waipio	26.5 to 28.9	27.9	25.2 to 28.5	27.0
Kapolei	18.9 to 21.3	20.1	22.2 to 26.2	23.6
Mililani Mauka	28.1 to 33.4	30.5	30.9 to 37.7	34.5
Wahiawa	50.6 to 56.9	52.5	54.1 to 63.7	58.3

Note: (1) In this report, in situ moisture contents refer to those obtained from the in situ material that was removed during sand cone testing. Nuclear density testing at Kapolei, Mililani Mauka and Wahiawa were performed in a trench whereby the trench walls could affect the moisture content as the standard count was obtained from outside the trench.

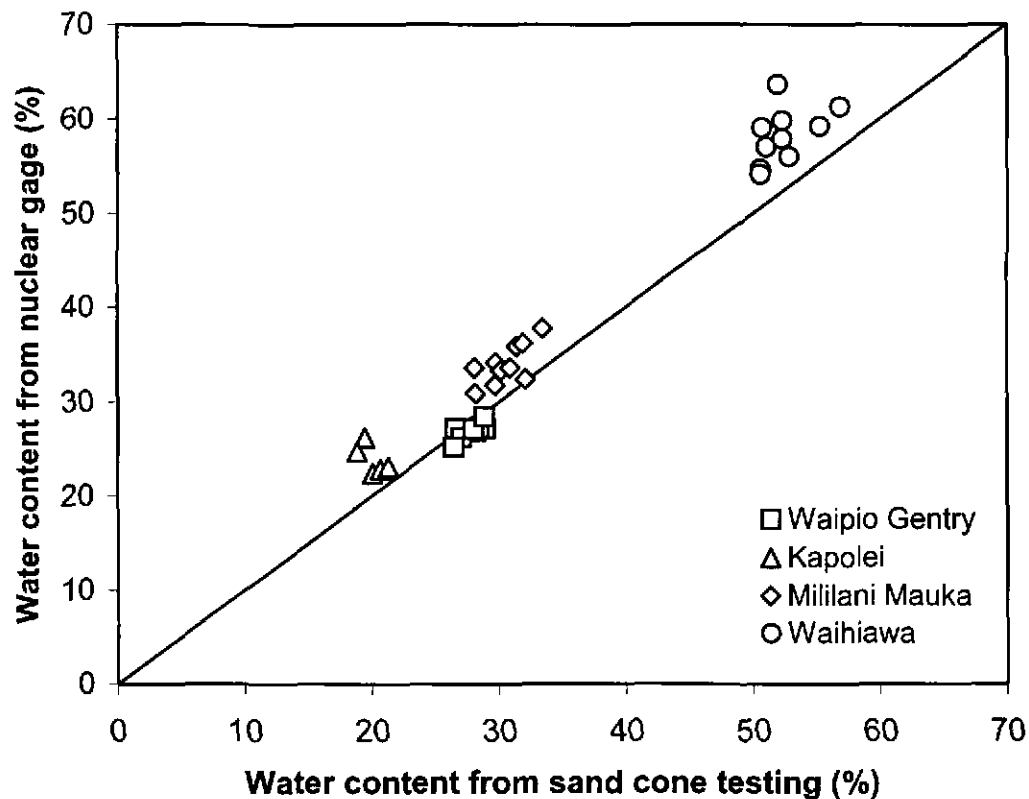


Figure 3.2 In situ water contents of sampled soils

Compared to the other soils, the Waihiawa soil was found to have the highest in situ moisture content ranging from 51% to 57%. This soil was tested at various stages of drying to study the effects drying had on the various soil properties. The Waihiawa soil was subjected to the following stages of drying:

1. Testing at the in situ moisture content (Samples were required to be tested at lower moisture contents than the in situ moisture content. These samples were tested from wet to dry to reduce the effects of drying on the soil; i.e., the samples were never rewetted after drying down). These samples are referred to as Waihiawa in situ.

2. Drying to approximately half the in situ moisture content or ~ 26% (Samples were then tested from dry to wet if the target moisture contents were higher. If the target moisture contents were lower, then they were tested from wet to dry). These samples are referred to as Wahiawa intermediate.
3. Testing after oven-drying the soil (Samples were tested from dry to wet). These samples are referred to as Wahiawa oven-dry.

Waipio, Kapolei and Mililani Mauka soils had lower in situ moisture contents compared to the Wahiawa soil. Increasing in situ moisture contents generally are characteristic of soils from higher elevations and wetter climates on Oahu. The soils were tested from dry to wet if the target moisture contents were higher than the in situ or from wet to dry (without rewetting) if the target moisture contents were lower. Also, a few tests were performed on oven-dried samples to see if they underwent irreversible changes upon drying. Test results are presented next.

3.2 Index Tests and Results

The following laboratory index tests were performed on each soil sample:

- Atterberg limits
- Grain size distribution
- Sand equivalent
- Specific gravity

3.2.1 Atterberg Limits

Liquid and plastic limits were determined in accordance with ASTM Standard D4318-00 and are summarized in Table 3.2. The test results are also summarized in a plasticity chart shown in Figure 3.3. Based on the Unified Soil Classification System

(USCS), soils from Waipio and Kapolei are classified as low plasticity silt, or ML. Soils from Mililani Mauka and Wahiawa are classified as high plasticity silt, or MH. Based on the AASHTO classification system, the Waipio and Kapolei soils are A7-6 while the Mililani Mauka and Wahiawa soils are A7-5.

Upon drying, the Atterberg limits generally trend down the A-line, with the shift more pronounced for the high plasticity soils.

Table 3.2 Atterberg limits test results

Soil	Atterberg Limits								
	Type	Determinations						Avg.	Ovendry
		1	2	3	4	5	6		
Waipio	Liquid Limit	45.4	43.4	47.6	46.0	--	--	45.6	42.8
	Plastic Limit	25.0	26.7	37.7	29.3	--	--	29.7¹	30.9 ¹
	Plasticity Index	20.3	16.7	9.8	16.7	--	--	15.9	12.0
Kapolei	Liquid Limit	42.2	40.4	41.7	41.4	41.2	--	41.4	36.6
	Plastic Limit	26.2	28.5	26.4	27.6	27.8	--	27.3	24.9
	Plasticity Index	16.0	12.0	15.3	13.7	13.4	--	14.1	11.7
Mililani Mauka	Liquid Limit	96.9	88.4	95.5	100.0	--	--	95.2	57.7
	Plastic Limit	46.8	43.9	38.8	47.2	--	--	44.2	37.5
	Plasticity Index	50.1	44.4	56.7	52.7	--	--	51.0	20.2
Wahiawa In situ	Liquid Limit	93.7	97.4	96.5	108.6	96.6	--	98.6	--
	Plastic Limit	44.1	48.5	49.4	49.1	44.6	--	47.1	--
	Plasticity Index	49.6	48.9	47.1	59.5	52.1	--	51.4	--
Wahiawa Intermediate	Liquid Limit	89.7	84.7	87.6	--	--	--	87.3	--
	Plastic Limit	41.1	42.1	43.6	--	--	--	42.3²	--
	Plasticity Index	48.6	42.6	44.0	--	--	--	45.1	--
Wahiawa Owendry	Liquid Limit	71.0	60.3	68.6	54.6	64.6	62.5	63.6	--
	Plastic Limit	47.5	43.5	43.6	42.2	43.0	45.4	44.2²	--
	Plasticity Index	23.5	16.8	25.1	12.4	21.7	17.2	19.4	--

Note

1. Plastic limit of the oven dry soil is higher than the average for the in situ soil. The difference is not significant, it's well within the margin of error and may be attributable also to variability in the soil.
2. The average plastic limit of the oven dry soil is higher than the intermediate soil. The difference is not significant, it's well within the margin of error and may be attributable also to variability in the soil.

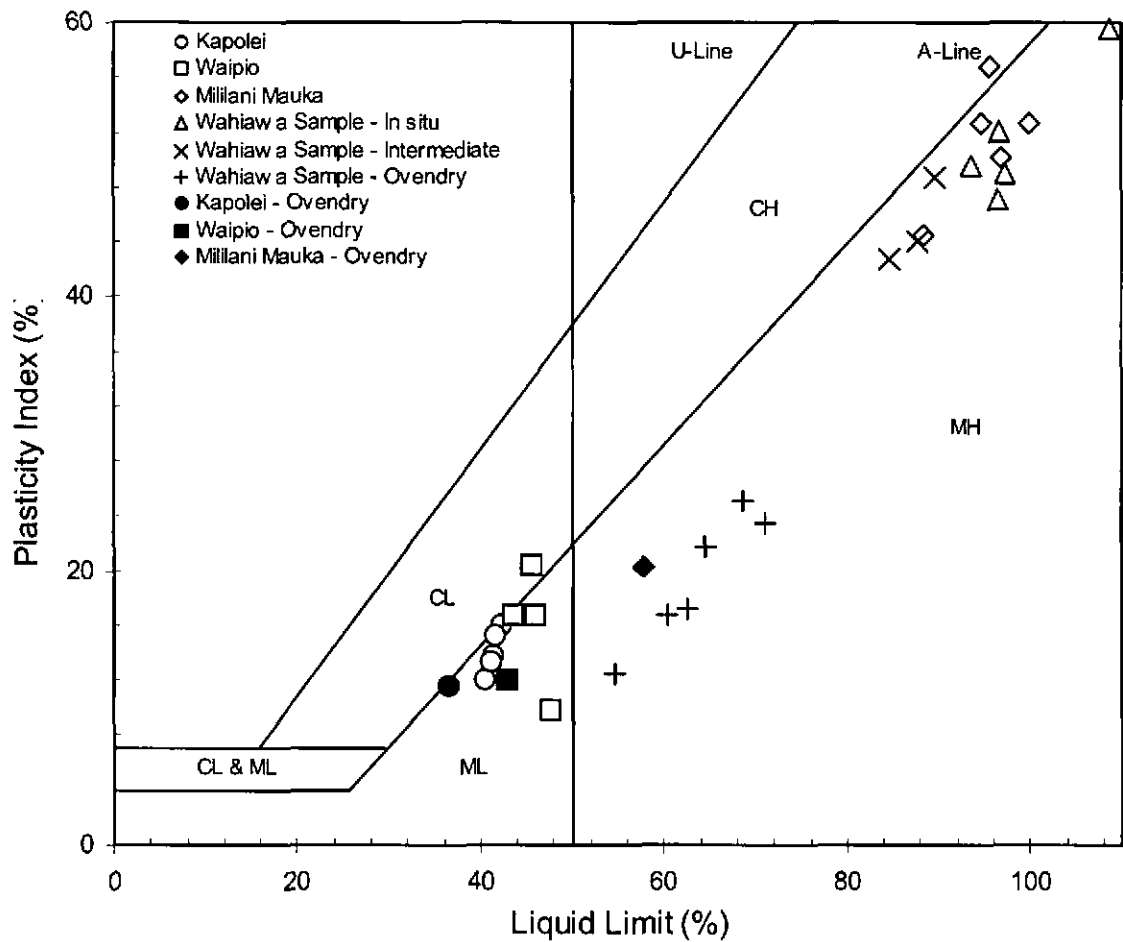


Figure 3.3 Atterberg limits and plasticity chart

The liquidity index (LI) relates the natural moisture content of the soil in the ground to the plastic limit and plasticity index. It is defined as

$$LI = \frac{w - PL}{PI} \quad (3.1)$$

where w = natural moisture content, PL = plastic limit and PI = plasticity index. Values of LI are summarized in Table 3.3.

Table 3.3 Liquidity index

Soil	Natural Moisture Content (%)	Plastic Limit (%)	Plasticity Index (%)	Liquidity Index
Waipio	27.9	29.7	15.9	-0.11
Kapolei	20.1	27.3	14.1	-0.51
Milliani Mauka	30.5	44.2	51.0	-0.27
Wahiawa	52.5	47.1	51.4	0.11

The LI provides an indication of the soil's consistency and sensitivity. If LI is approximately equal to 0, the natural moisture content is close to the plastic limit. This indicates that the soil sensitivity (undisturbed strength divided by remolded strength) is low and the soil consistency may be relatively stiff. On the other hand, if LI approaches unity, the soil is close to the liquid limit. This is an indication that the soil is sensitive. If LI is less than 0, this is an indication that the soil is desiccated and hard. Three of the four soils had negative liquidity indices. Only the Wahiawa soil had a positive LI. However, its LI is relatively low (0.11).

3.2.2 Grain Size Distribution

Grain size distributions were obtained by performing hydrometer testing and wet sieve analyses in accordance with ASTM Standard D422-63 (2002). Three variations of the wet sieve/hydrometer tests were used on the Kapolei soil to assess the sensitivity of each method:

Method 1.

1. Soil from a bucket were divided into two 100g (0.22 lbs) portions.
2. Several moisture contents of the soil were then measured on each portion.
3. Using the moist weight from (1) and the moisture content from (2), the total dry weight was then calculated.

4. One portion was wet sieved through a stack of sieves (No. 40, 60, 100 and 200). The material retained on the sieves was oven-dried to determine the dry weights.
5. The portion passing the No. 200 sieve was not collected but the dry weight of the percentage passing the No. 200 sieve can be estimated by subtracting the sum of all the dry weights from (4) from the total dry weight from (3).
6. The second portion of the soil from (1) was wet sieved through the No. 200 sieve and the fines and water were collected.
7. The collected soil/water mix from (6) was then dried to a moisture content that is near, but not less than the in situ value.
8. After determining the moisture content, a portion of the moist fines equivalent to a dry weight of approximately 50g (0.11 lbs) was subjected to hydrometer testing. The actual dry weight of soil used in the hydrometer test was determined at the conclusion of the hydrometer test.
9. The results from the wet sieve analyses and the hydrometer test were then combined to yield the complete grain size distribution.

Method 2.

This method is identical to method 1 except for steps 1 and 5. In step 1, only one portion of sample was used for wet sieving. In step 5, all the fines passing the No. 200 sieve were collected for the hydrometer test.

Method 3.

This method is identical to method 2 except that the soil retained on the No. 40, 60, 100 and 200 sieves were mixed with a 100 ml standard sodium hexametaphosphate solution for several hours and stirred in a mechanical mixer. The deflocculated material was wet sieved through the stack of the four finest sieves again. The material retained on the sieves was oven-dried to determine the dry weights while the fraction passing the No. 200 sieve was collected and dried to a moisture content near the in situ value. After determining the moisture content, a portion of the moist fines equivalent to a dry weight of 50g (0.11 lbs) was subjected to hydrometer testing.

The results from all three methods are plotted in Figure 3.4b for the Kapolei soil. Methods 2 and 3 are the most reliable but they are the most tedious to perform because a significant amount of water had to be evaporated prior to hydrometer testing. When the results from the methods were compared, they all yielded similar results, although method 3 resulted in the finest grain size distribution because of the use of the deflocculant prior to wet sieving through the four smallest sieves. Because the differences are relatively insignificant and because methods 2 and 3 are time consuming, the grain size distributions of the soil from the other three locations were obtained using method 1.

The grain size distributions for all four soils are plotted in Figures 3.4a through 3.4d.

3.2.3 Sand Equivalent

The sand equivalent test is used to determine the characteristics of the finer grained portion of cohesionless soils. Typically, clays have sand equivalents between 0 and 5, silty clays between 6 and 10, clayey silts between 11 and 30, clayey fine sands between 30 and 40, and silty fine sands above 40. Sand equivalent tests were performed in accordance with AASHTO T 176-02, the results of which are summarized in Table 3.4. The test results below confirm that the soils tested were predominantly clayey silts.

Table 3.4 Sand equivalent test results

Soil	Sand Equivalent						
	Determinations					Average	Ovendry
	1	2	3	4	5		
Waipio	8	11	13	17	--	12	12
Kapolei	8	8	7	10	--	8	12
Mililani Mauka	11	9	10	16	11	11	10
Wahiawa In situ	14	14	13	18	--	15	--
Wahiawa Owendry	21	21	19	19	20	20	--

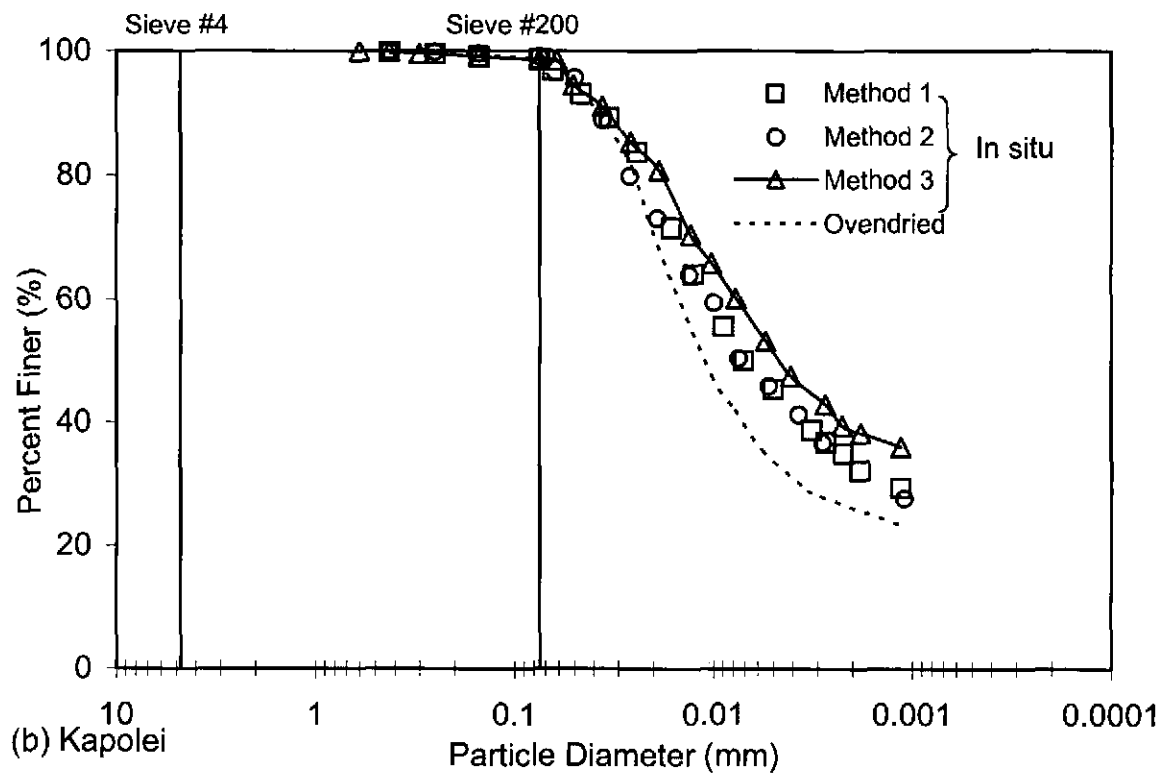
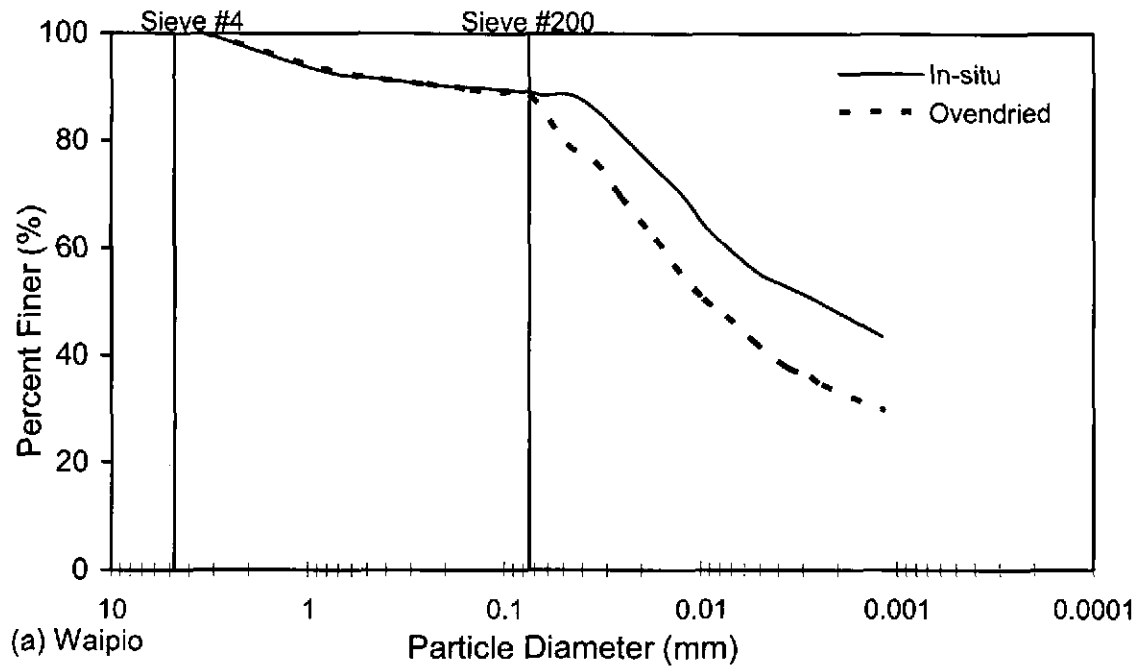


Figure 3.4 Grain size distribution for soils from (a) Waipio; (b) Kapolei; (c) Mililani Mauka; and (d) Wahiawa

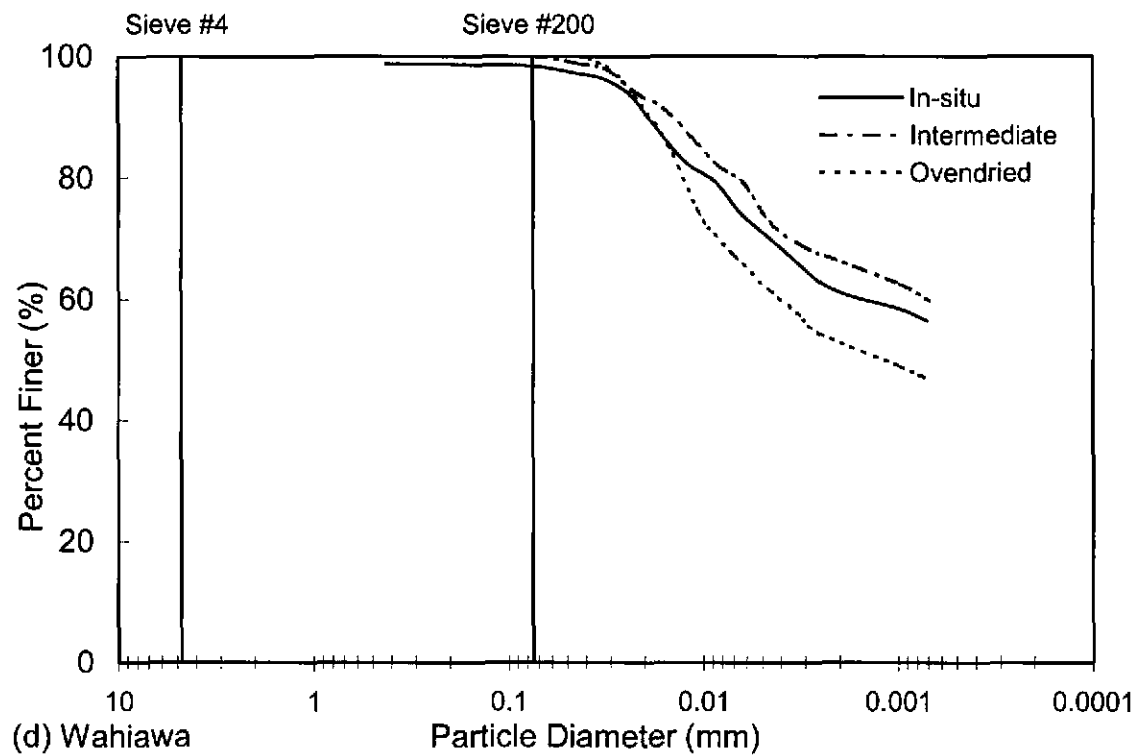
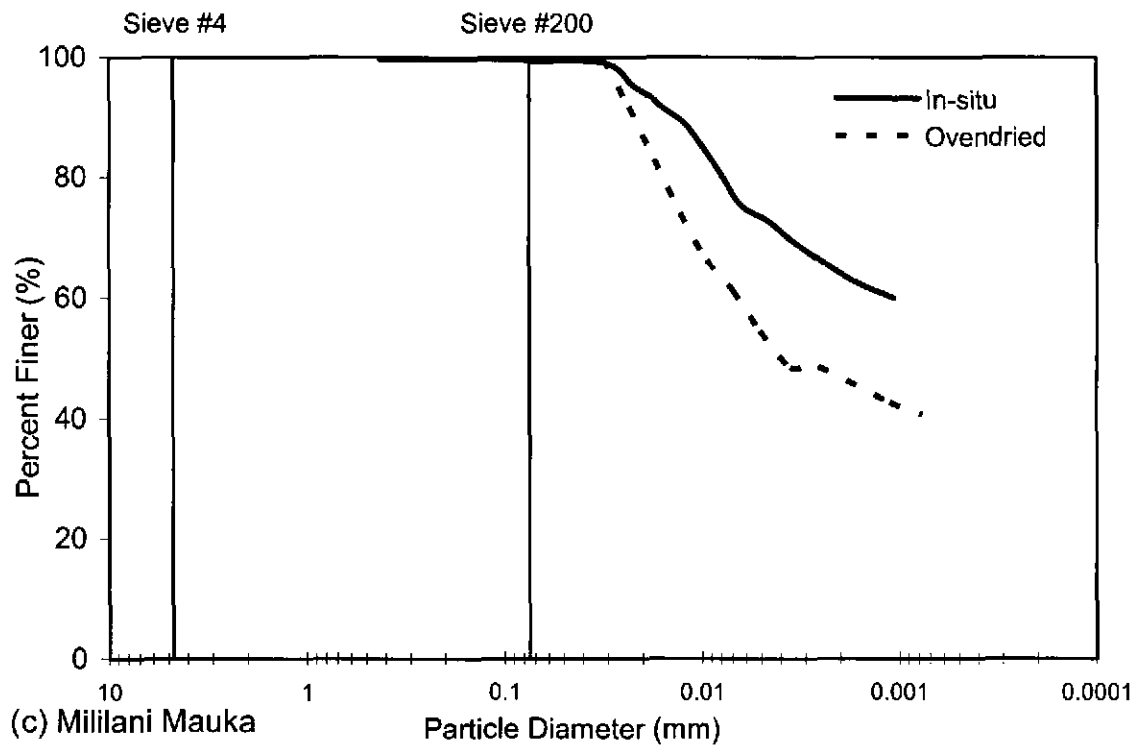


Figure 3.4 Grain size distribution for soils from (a) Waipio; (b) Kapolei; (c) Mililani Mauka; and (d) Wahiawa (continued)

3.2.4 Specific Gravity

The specific gravity of the soils was measured in accordance with ASTM Standard D854-98, the results of which are summarized in Table 3.5.

Table 3.5 Specific gravity test results

Soil	Specific Gravity						
	Determinations					Average	Ovendry
	1	2	3	4	5		
Waipio	2.82	2.99	2.90	2.90	--	2.90	2.90
Kapolei	2.96	2.90	3.09	3.04	--	3.00	3.06
Mililani Mauka	2.96	2.94	3.01	3.01	--	2.98	3.00
Wahiawa In Situ	2.99	3.06	3.20	3.22	2.94	3.08	--
Wahiawa Owendry	3.09	2.94	3.17	3.25	--	3.11	--

In general, oven drying the soils did not lead to significant changes in the specific gravity.

3.2.5 Activity

Activity is the ratio of plasticity index to % clay (% finer than 0.002 mm). The plasticity index is related to both the mineralogy and the amount of clay present. For example, a soil rich in kaolinite may have a similar plasticity index as another soil with little montmorillonite. The effects can be separated by the activity of the soil, which is related to the specific surface area of the clay mineral. According to Mitchell (1993), the activity is approximately 0.5 for kaolinite, between 0.5 and 1 for illite and between 1 and 7 for montmorillonite. The activity of the soils sampled is summarized in Table 3.6.

The activity of the ML soils increased minimally after oven drying while the activity of the MH soils decreased significantly after oven drying. At its natural state, the activity of the Wahiawa soil is 0.83. It decreased by about 18% when the natural water content was halved and it decreased by about 55% after oven drying.

Table 3.6 Activity of soils tested

Soil	Plasticity Index (%)	Clay Fraction (%)	Activity
Waipio	15.9	48.0	0.33
Waipio oven-dry	12.0	33.0	0.36
Kapolei	14.1	38.0	0.37
Kapolei oven-dry	11.7	26.0	0.45
Mililani Mauka	51.0	64.0	0.80
Mililani Mauka oven-dry	20.2	47.0	0.43
Wahiawa in situ	51.4	62.0	0.83
Wahiawa intermediate	45.1	67.0	0.67
Wahiawa oven-dry	19.4	52.5	0.37

3.2.6 Swell Potential

The Waterways Experimental Station (WES) provides a useful classification for identifying in situ soils with a swell potential based on Atterberg limits. The swell potential can be classified as low, marginal or high as summarized in Table 3.7. These classifications are based on volume change measured from oedometer testing of undisturbed soils.

Table 3.7 WES method of classifying swell potential of undisturbed soils (after Reese and O'Neill, 1988)

LL (%)	PI (%)	Suction Pressure (tsf)	Potential Swell (%)	Potential Swell Classification
> 60	> 35	> 4	> 1.5	High
50 – 60	25 - 35	1.5 - 4	0.5 – 1.5	Marginal
< 50	< 25	< 1.5	< 0.5	Low

Based on the Atterberg limits and Table 3.7, the potential swell classifications for the ML soils (Waipio and Kapolei) and MH soils (Mililani Mauka and Wahiawa) are low and high, respectively. The swell potential of compacted soils is provided in the 1997 Uniform Building Code in Table 3.8. These classifications are also based on volume change measured from oedometer testing.

Table 3.8 Swell potential classification of compacted soils (Uniform Building Code, 1997)

Percent Swell (%)	Potential Swell Classification
> 13	Very High
9.1 – 13	High
5.1 – 9	Medium
2.1 – 5	Low
0 - 2	Very Low

CHAPTER 4 CBR TESTING AND RESULTS

Highlights of the CBR test procedures that deviate from AASHTO T 193-99 and ASTM D1883-99 or features that pertain to this project are discussed below.

4.1 Test Program

CBR tests were performed on the soil samples prepared using several compactive efforts and a variety of physical states. They are as follows:

1. 5 layers at 56 blows per layer (compaction effort equivalent to the Modified Proctor or AASHTO T 180-01 Method B or ASTM D1557-02 Procedure C)
2. 5 layers at 25 blows per layer
3. 5 layers at 10 blows per layer
4. 3 layers at 56 blows per layer (compaction effort equivalent to the Standard Proctor or AASHTO T 99-01 Method B or ASTM D698-00 Procedure C)
5. Kentucky CBR

For the first four test series, the CBR was measured on at least 5 samples with varying physical states along the compaction curve (one at or close to the optimum moisture content, two dry-of-optimum and two wet-of-optimum). The Kentucky CBR was measured at only one physical state. This method required that the soil specimen be molded at or near the optimum moisture content as determined by the Standard Proctor (AASHTO T 99-01 or ASTM D698-00) compaction test. The soil is then compacted in a standard 6" CBR mold using dynamic compaction, where a 10 pound (4.536 kg) hammer is dropped from a height of 18 inches (45.72 cm). The soil is compacted in five equal layers with each layer receiving 10 blows.

4.2 Equipment

The CBR testing equipment consisted of a loading frame supporting a piston that penetrated the soil within the mold (Figure 4.1). A data acquisition system was used to record the load and displacement during the penetration portion of the CBR test. These readings were checked with manual readings of the load using a 6000-lb (2721 kg) rated proving ring and displacements using a Soiltest Inc. LC-8 dial gauge for quality assurance. The data acquisition system consisted of the following equipment:

1. 3000-lb (1360 kg) rated load cell (Sensortronics 60001-3K)
2. Two linear variable differential transducers (LVDT) with a range of ± 1 inch (25.4 mm) (Schaevitz 1000MHR)
3. Signal conditioner (PMG Precision Instruments SC-5B AC Transducer)
4. Computer with analog to digital (A/D) board (Metrabyte)
5. ATS software (Version 3.1)

During testing, the voltage output from the load cell and LVDT's were transmitted to the signal conditioner, which converted the voltage to an analog output (in bytes). The analog output is then translated by the A/D board to load and displacement units. The load cell and LVDT's were calibrated periodically to ensure that the correct load and displacement were recorded by the ATS software. The LVDT's were placed diametrically opposite, and the measured displacement was taken as the average of both LVDT readings.

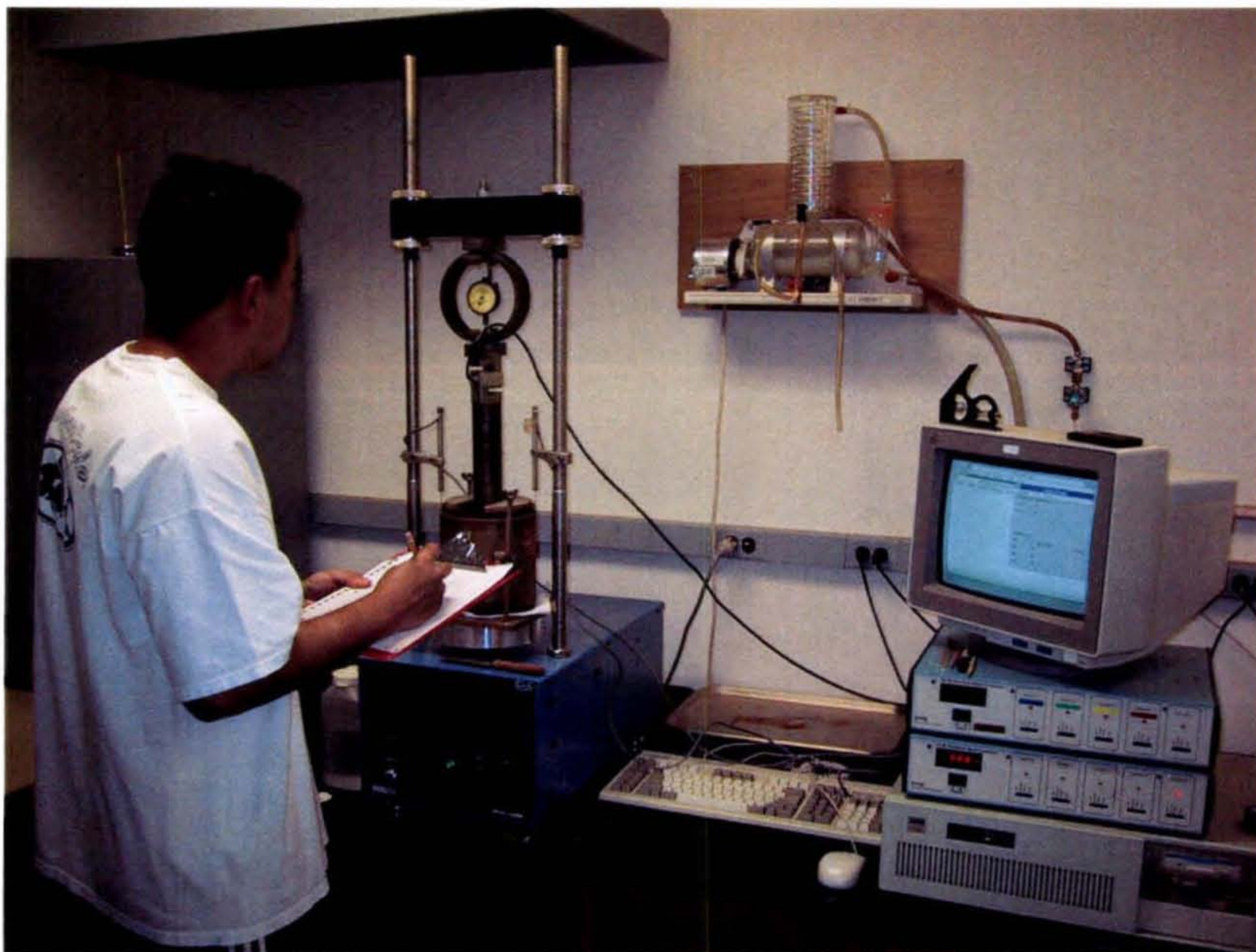


Figure 4.1 CBR penetration test apparatus and data acquisition system

4.3 Test Procedure

The CBR tests were performed in accordance with ASTM D1883-99 and AASHTO T 193-99. A few modifications were made to the testing procedure and sample preparation. These are described below.

4.3.1 Sample Preparation

As stated in ASTM D1883-99, if the material passed the $\frac{3}{4}$ in (19-mm) sieve then the entire sample shall be used for testing. All of the soil samples tested passed the No.

4 sieve. Extraneous materials such as roots and other materials which may alter the results of the CBR test were removed prior to testing.

Based on the moisture contents determined during the initial field sampling, two 5-gallon buckets of soil with similar moisture contents were thoroughly hand mixed, passed through a No. 4 sieve, placed back into the plastic bags which were then heat sealed, and restored in the 100%-humidity concrete curing room to ensure a uniform soil and moisture distribution. Moisture content checks on the samples were taken to ensure no moisture lost occurred during sample preparation.

Two 5-gallon buckets (approx. 40 lbs) of moist soil were required for a series of CBR tests at each compactive effort. Prior to testing the soil from both buckets were again emptied into a large pan and mixed thoroughly. Also, any soil clumps were broken apart at this time and another series of moisture content checks were performed to ensure no moisture loss occurred after soil mixing.

4.3.2 Compaction

Compaction of the CBR test samples were performed in accordance with ASTM D1883-99 and AASHTO T 193-99 with several minor modifications. The soil required for each lift was prepared separately rather than in a single batch. For each lift, the required amount of water and soil was mixed thoroughly in a pan prior to compaction. Moisture contents were determined using soil samples from lifts 1, 3 and 5 for soils compacted in 5 layers, or every lift for soils compacted in 3 layers. Additional soil was added to each batch to allow moisture content determinations to be made. Soil for the subsequent lifts were prepared during compaction of a lift to minimize drying of the soil.

A Boart Longyear S-335 mechanical compactor (Figure 4.2) was used to prepare the soil for the following series of tests:

1. 5 Layers at 56 Blows per layer.
2. 5 Layers at 25 Blows per layer.



Figure 4.2 Mechanical rammer used for compaction of CBR samples

The mechanical compactor used a pie-shaped rammer, and was bolted to concrete floor. Prior to testing, the mechanical compactor was calibrated in accordance with ASTM D2168-02 by comparing the deformation of lead cylinders using both the mechanical compactor and a manual compactor.

For the remaining test series (5 Layers at 10 blows per layer using a 10-lb (4.54 kg) hammer, 3 Layers at 56 blows per layer using a 5.5-lb (2.5 kg) hammer and the Kentucky CBR), the samples were compacted manually. These test series were compacted manually because it was determined that the mechanical compactor did not provide equal compactive effort to each lift when the number of blows per lift is low (i.e. 10) and when using the 5.5 lbs (2.5 kg) rammer. In these instances, pockets of uncompacted soil were observed when using the mechanical compactor.

4.3.3 Soaking of Samples

Volume change below road pavements occur upon loading as well as upon changes in moisture content. The focus of this section is on wetting-induced volume change rather than the load-induced variety. Volume change, especially in expansive and collapsing subgrades, can cause pavement distress, and should ideally be minimized. Generally, volume change tends to be higher when soils are compacted dry of optimum (Seed, 1959 and Lawton et al., 1989).

After compaction, each sample was placed into a tub of water and soaked for four days. The soaking was necessary to simulate the worst-case scenario in the field. A 15-lb (6.8 kg) surcharge load was placed on the specimens during soaking (Figure 4.3) to simulate the effect of the pavement overburden stress. Measurements of swell were taken for each sample. If the sample swelled within the first hour, measurements were

taken every hour for four hours. Measurements were then taken on a daily basis during the four day soaking period. Using swell measurements after the 4-day-soaking period, the volumetric expansion was calculated as the swell divided by the original sample height for each point, and contour lines of percent volume change were generated as shown in Figure 4.4. In general, volume change decreased as the molding water content increased. Also the maximum volume change occurred dry of optimum. To minimize volumetric expansion in compacted soils, they should ideally be compacted on the “wet side”. However, using too high a moisture content can compromise the strength (CBR) of the soil (see later).



Figure 4.3 Soaking of CBR specimens and monitoring of swell

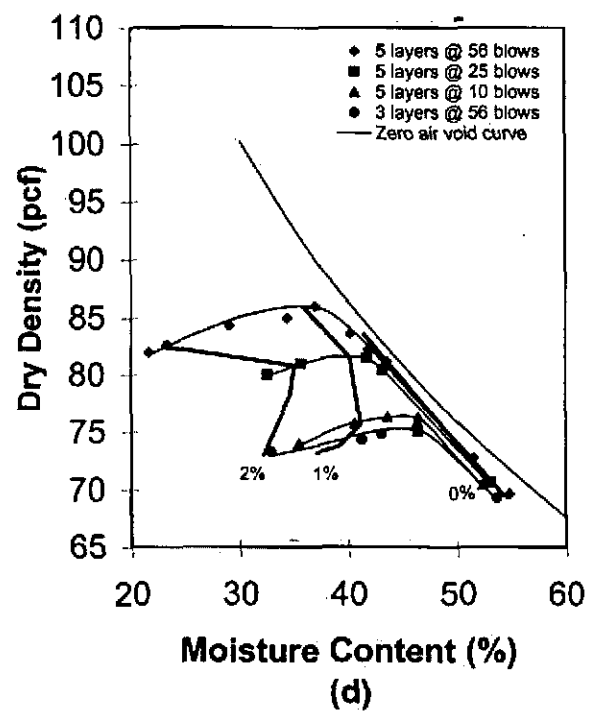
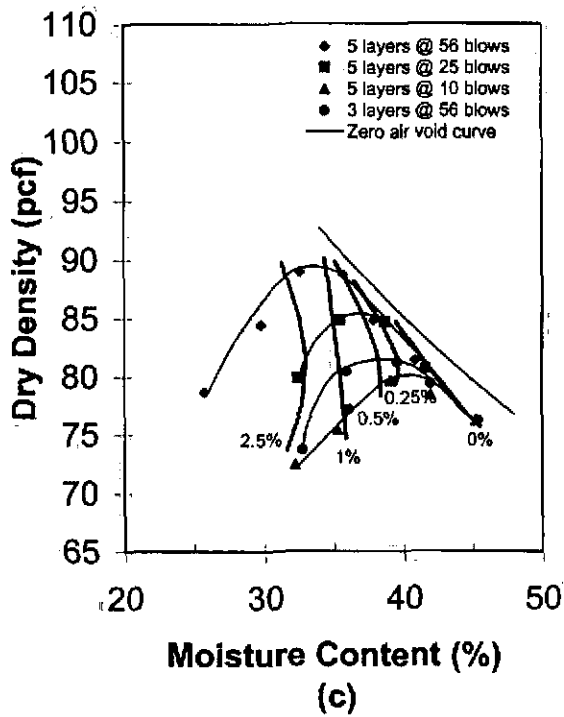
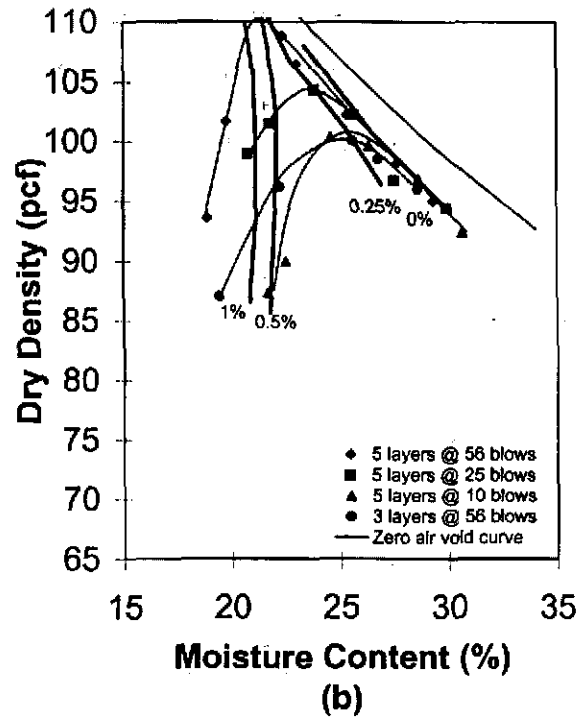
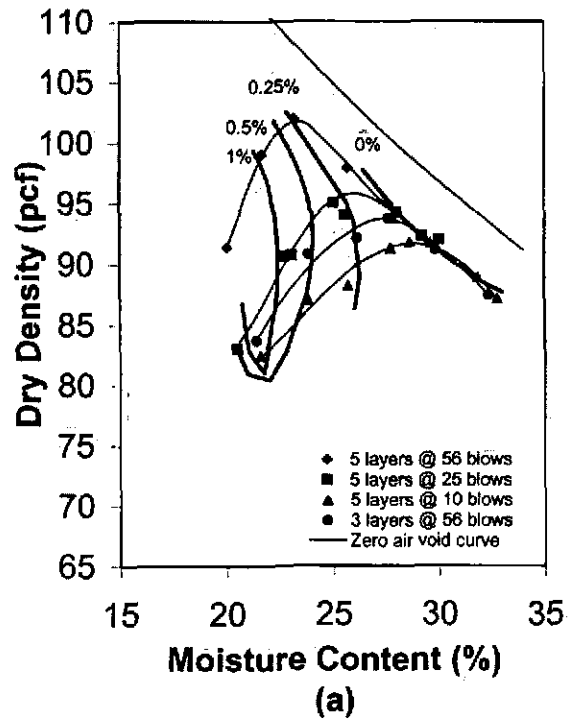


Figure 4.4 Swell contours for (a) Waipio; (b) Kapolei; (c) Mililani Mauka and (d) Wahiawa in situ

A plot of percent swell versus CBR is shown in Figure 4.5. The swell is higher for the high plasticity soils (i.e., Wahiawa and Mililani Mauka). The maximum values recorded for the percent swell were: 7.5%, 6.9%, 2.9% and 2.2% for Wahiawa, Mililani Mauka, Kapolei and Waipio, respectively. The maximum values recorded for the percent swell were: 7.5%, 5.1% and 3.0% for Wahiawa in situ, intermediate and ovoidry. According to the UBC method of classifying swell potential of soils, the Kapolei and Waipio soils that swelled 2.9% and 2.2%, respectively, can both be classified as having a low swell potential. While the Wahiawa and Mililani Mauka soils that swelled 7.5% and 6.9%, respectively, have a medium swell potential. The UBC potential swell classifications are based on swells measured in oedometer testing. It is expected that CBR swells will be less than those measured from oedometer testing as the CBR samples are significantly thicker and larger in diameter. Hee (2005) indicated that it is not uncommon to assume that CBR swells are approximately half of those from oedometer testing. Using this assumption, the potential swell classification based on CBR swells can be approximated as follows:

Potential Swell (%)	Swell Classification
0 – 2.5	Low
2.5 – 4.5	Medium
4.5 – 6.5	High
> 6.5	Very High

Therefore, the Kapolei and Waipio soils that swell 2.9% and 2.2% can be classified as having a medium and low swell potential, respectively, while the Wahiawa and Mililani Mauka soils that swell 7.5% and 6.9%, respectively, have a very high swell potential.

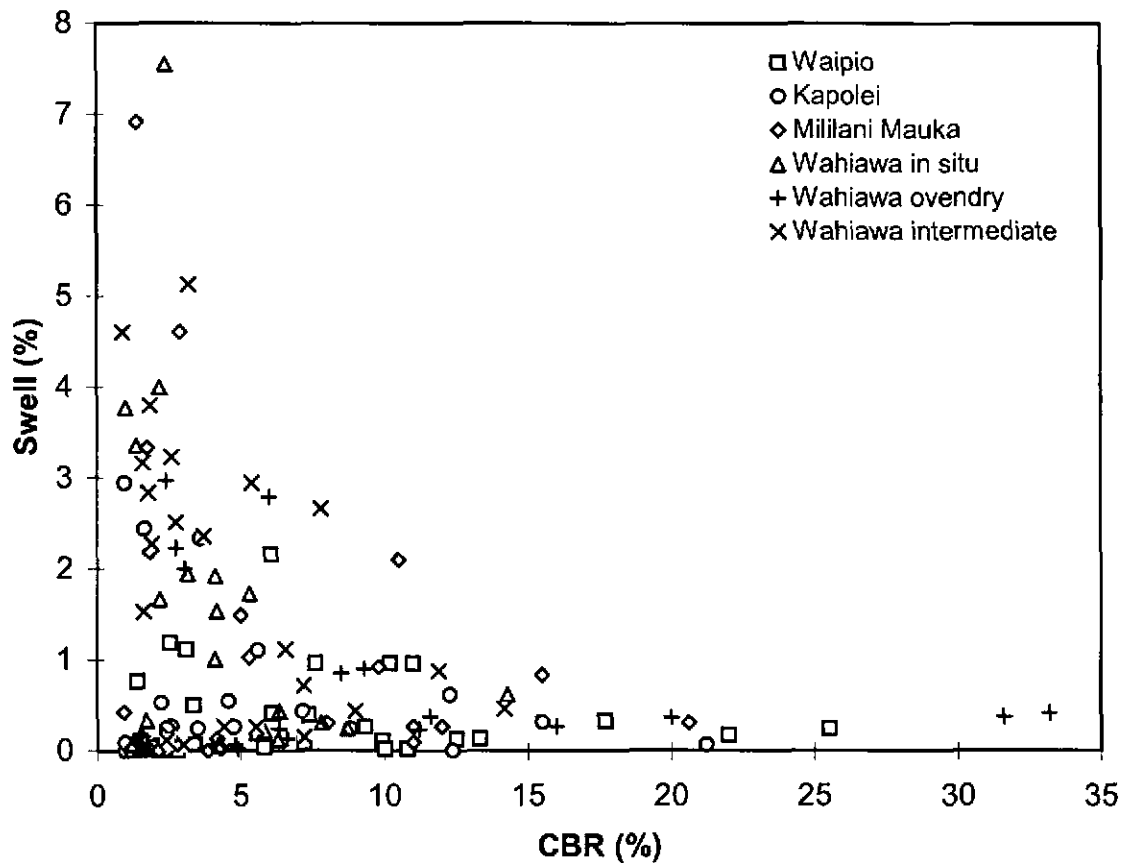


Figure 4.5 Swell versus CBR

4.3.4 Penetration Test

The 15-lb (6.8 kg) surcharge load remained in place during the CBR test. Two sets of readings were recorded: one set was taken manually and the second was recorded using the data acquisition system. Overall, the readings from both sets were very consistent. Upon completion of the CBR test, a moisture content determination was made.

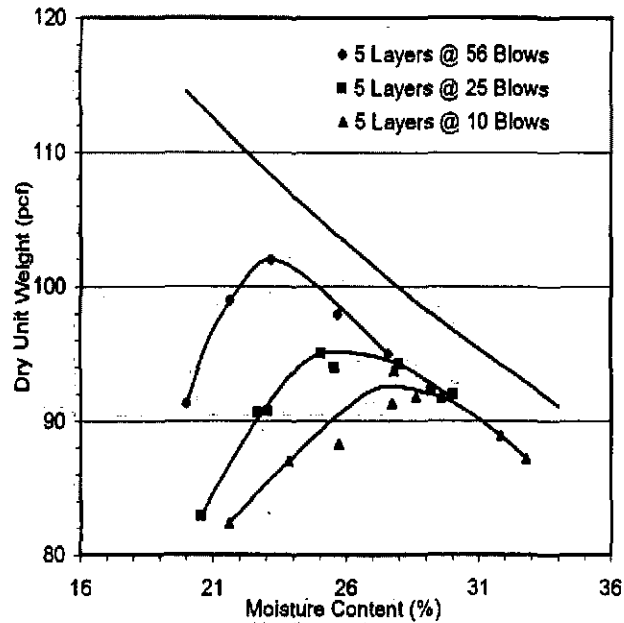
4.4 Analysis of Test Results

The load versus displacement curves were plotted to determine if corrections are needed. If the initial portion of the load-deflection curves concaved upward, a zero correction as specified in ASTM D1883-94 was made. Then, the bearing ratio was

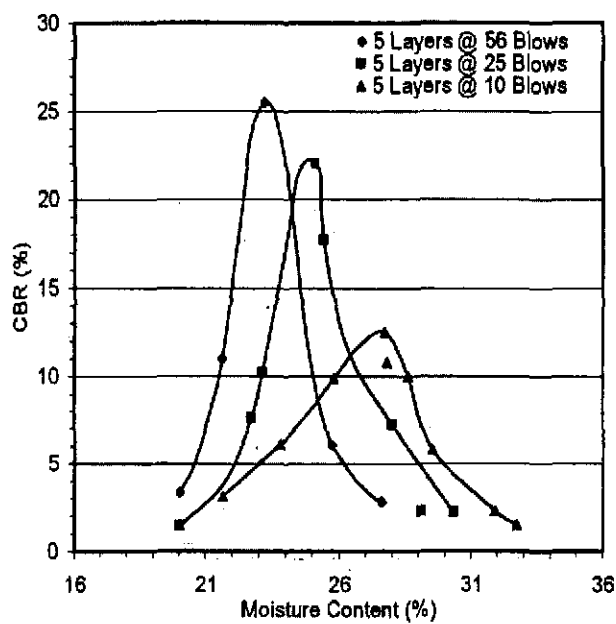
calculated at 0.1- (2.54 mm) and 0.2-inch (5.08 mm) deflections, and the greater of the two was recorded as the CBR.

Compaction curves were also plotted based on the measured dry unit weight and moisture content. A family of CBR curves was then generated for each soil. These curves are contained in Figs. 4.6 through 4.11. It should be noted that the soils, compacted in accordance with standard Proctor (56 blows in 3 layers), were not used to generate the family of curves. From Figs. 4.6b through 4.11b, CBR increases with increasing dry density until the optimum moisture content is reached, where a peak CBR is observed. Wet of optimum, the CBR decreases with decreasing dry density. In Figs. 4.6c through 4.11c, the CBR is plotted against dry density at constant moisture content. At low moisture contents, the CBR increases with increasing dry unit weight. At high moisture contents, the reverse is true where the CBR decreases with increasing dry density. This reduction is associated with water contents that are wet of the peak CBR. Therefore, a decrease in CBR can occur as a result of over-compaction (too large a compaction effort resulting in excessive dry unit weight) especially at high moisture contents.

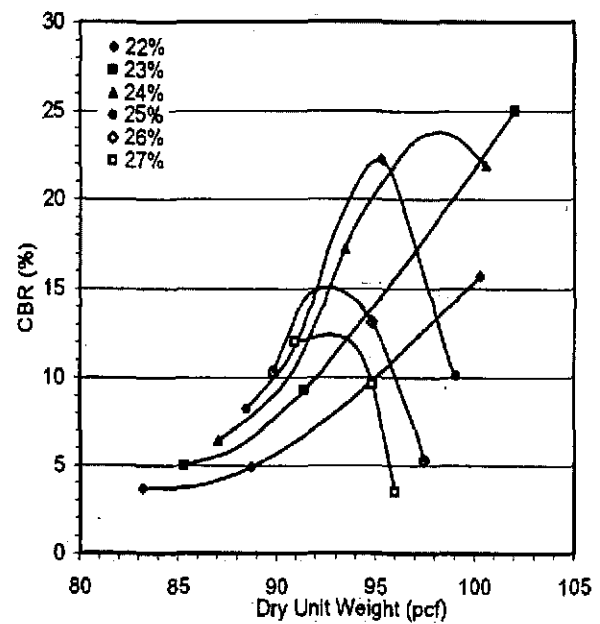
Two interesting observations on the Wahiawa soil can be made. First, drying results in a shift of the compaction curve up and to the left, with the exception of the soils compacted in 5 layers at 56 blows per layer (Fig. 4.12). In this case, the maximum dry unit weight for the "intermediate" soil is higher than the oven-dry soil. Second, as the sample is dried out, the CBR values tend to increase (see Figs. 4.9 through 4.11). For example, the peak CBR for the 56-blow, 5-layer soils increased from about 15 for in situ to 19 for intermediate to 38 for oven-dry.



(a)



(b)



(c)

Figure 4.6 CBR family of curves for Waipio (a) Dry unit weight versus moisture content; (b) CBR versus moisture content; and (c) CBR versus dry unit weight at constant moisture content

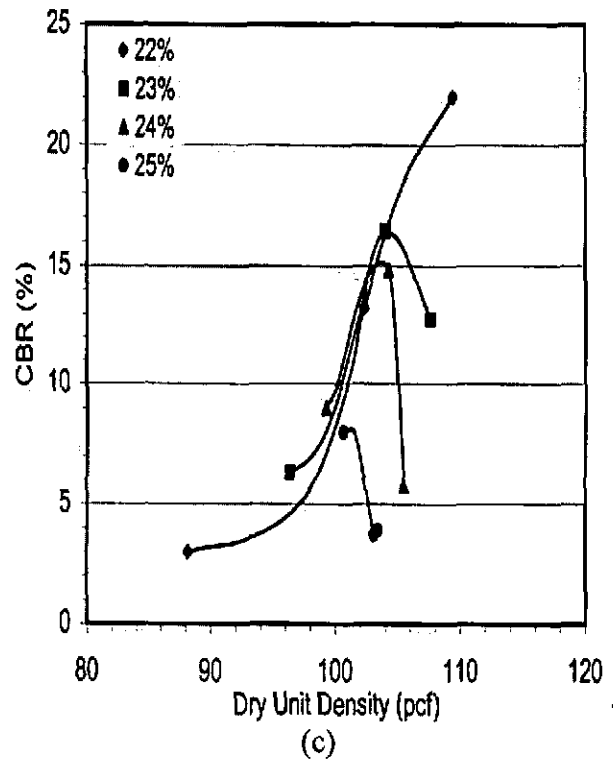
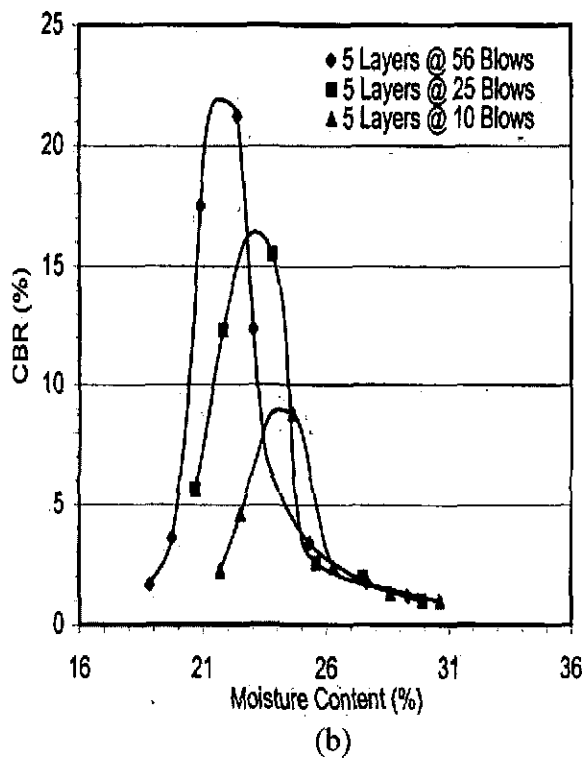
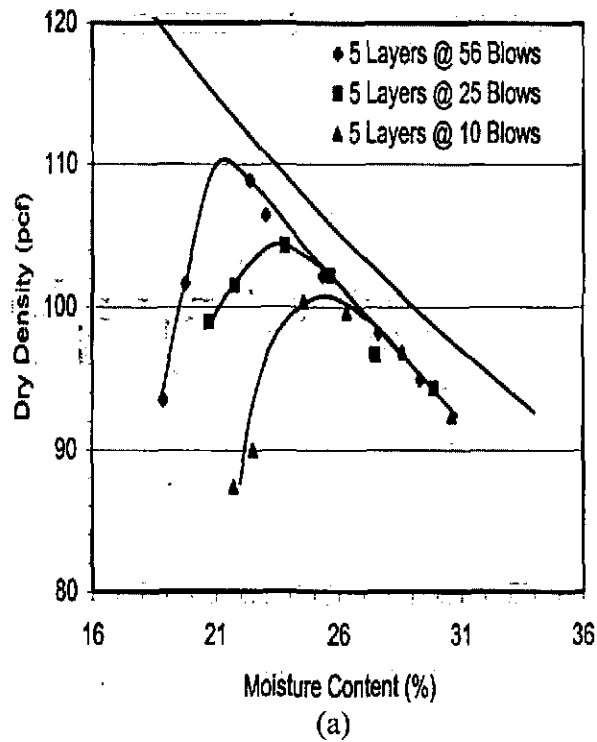


Figure 4.7 CBR family of curves for Kapolei (a) Dry unit weight versus moisture content; (b) CBR versus moisture content; and (c) CBR versus dry unit weight at constant moisture content

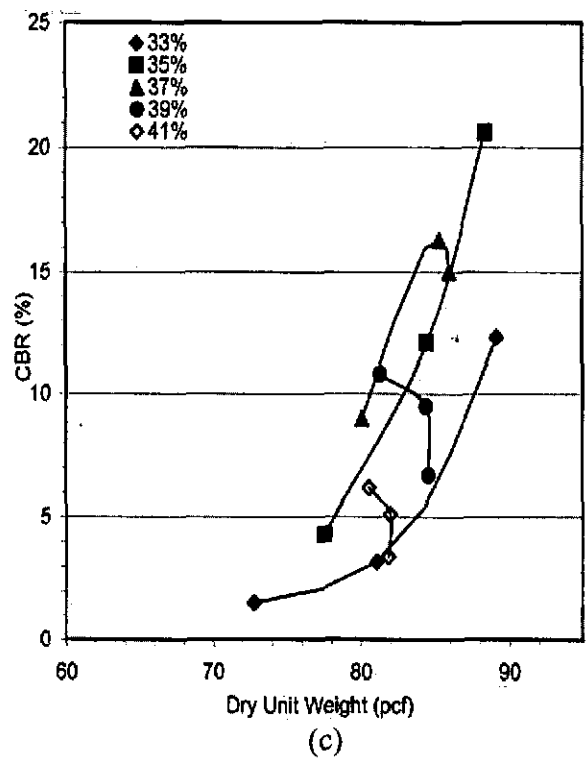
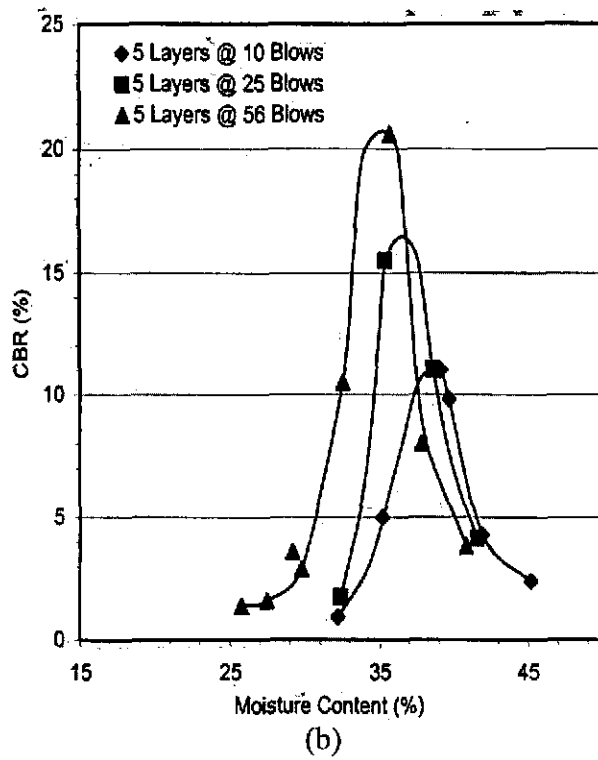
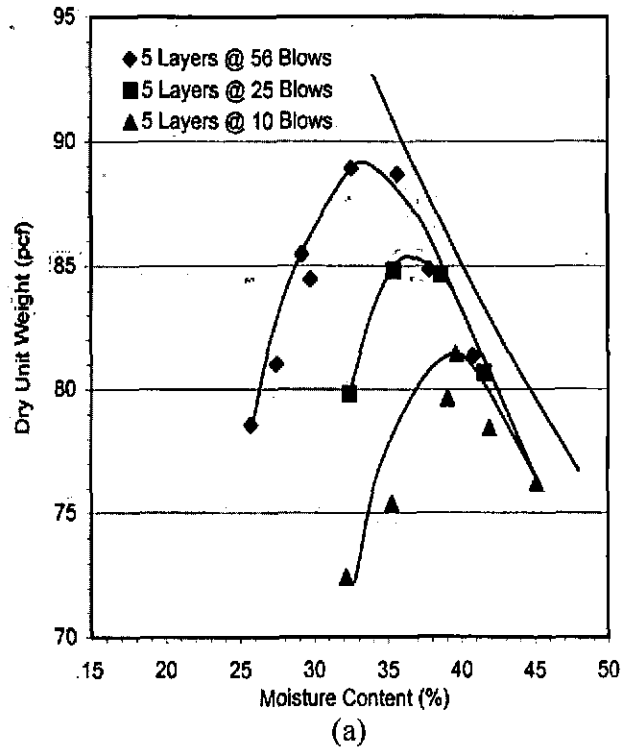
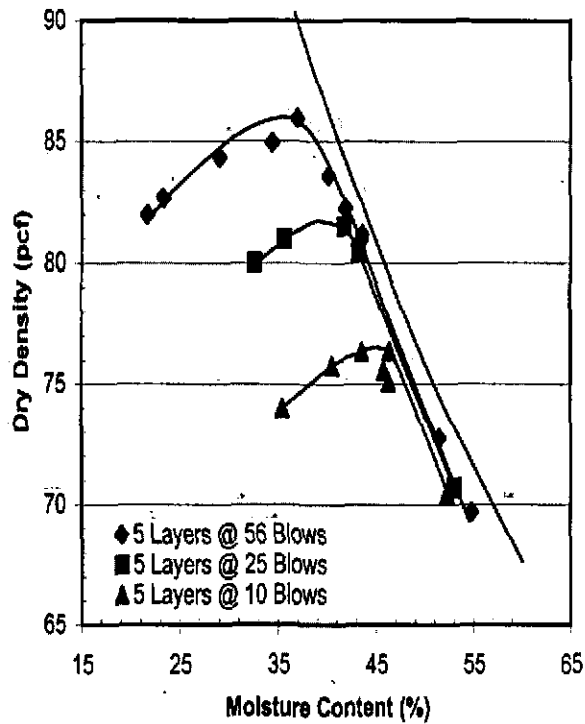
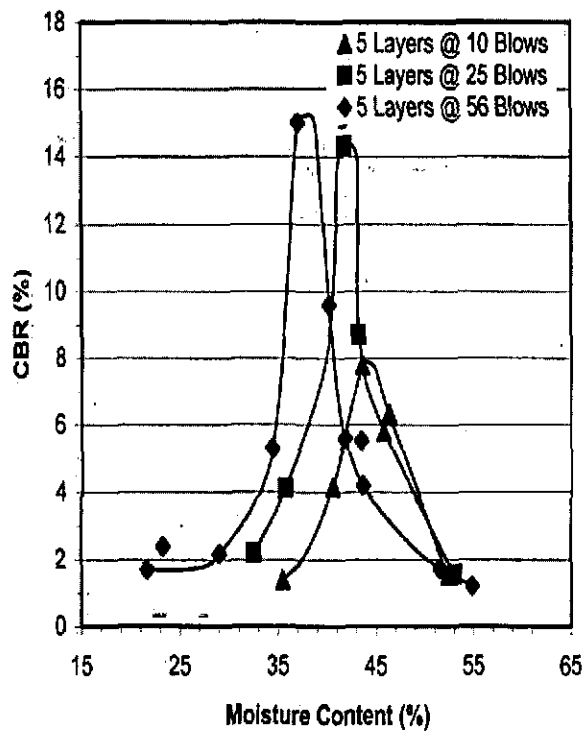


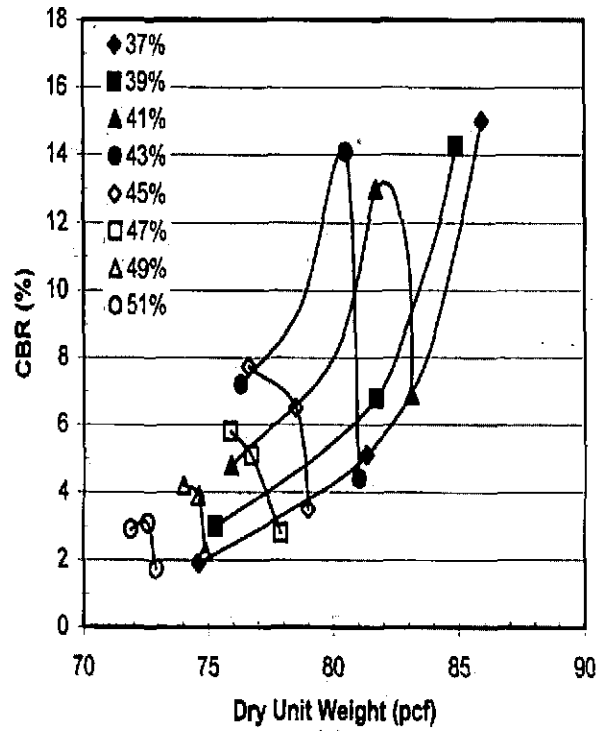
Figure 4.8 CBR family of curves for Mililani Mauka (a) Dry unit weight versus moisture content; (b) CBR versus moisture content; and (c) CBR versus dry unit weight at constant moisture content



(a)

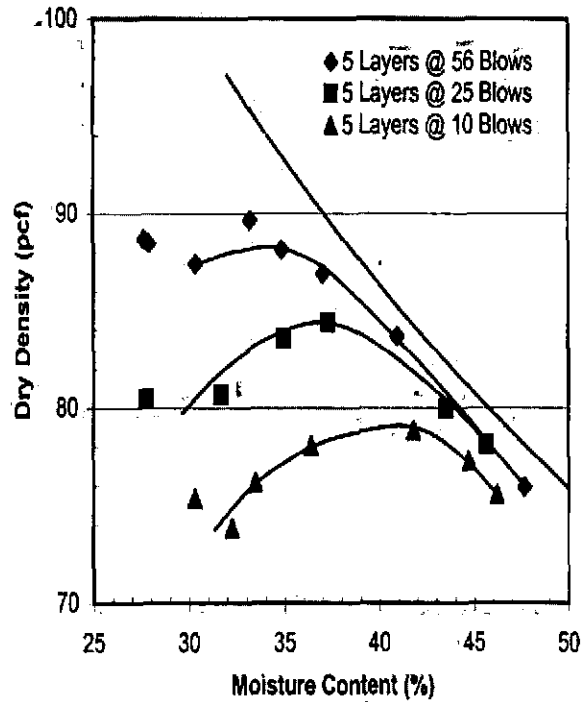


(b)

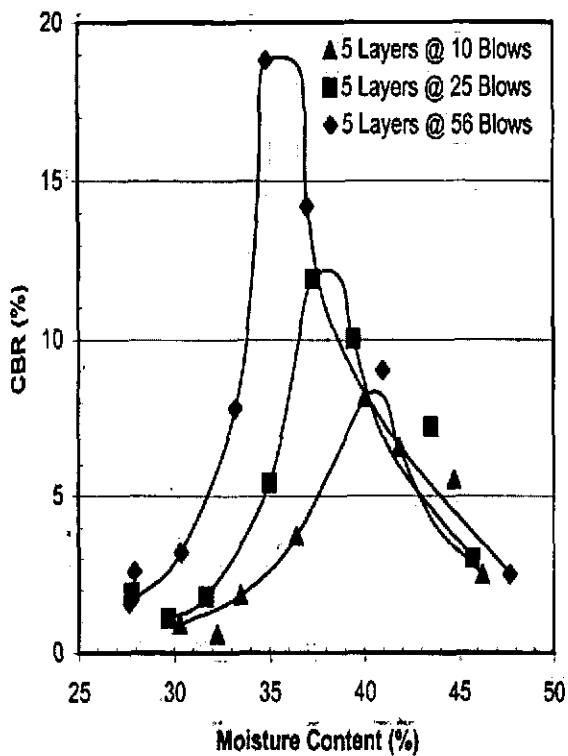


(c)

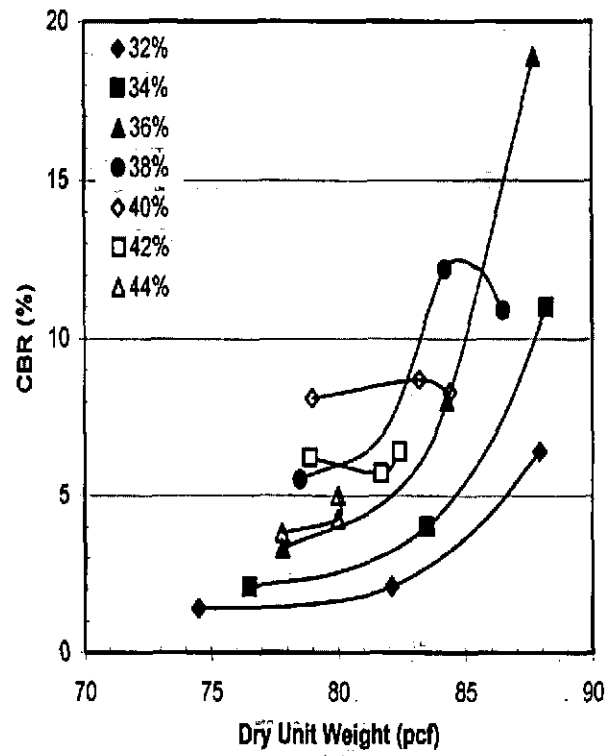
Figure 4.9 CBR family of curves for Wahiawa in situ (a) Dry unit weight versus moisture content; (b) CBR versus moisture content; and (c) CBR versus dry unit weight at constant moisture content



(a)



(b)



(c)

Figure 4.1 CBR family of curves for Wahiawa intermediate (a) Dry unit weight versus moisture content; (b) CBR versus moisture content; and (c) CBR versus dry unit weight at constant moisture content

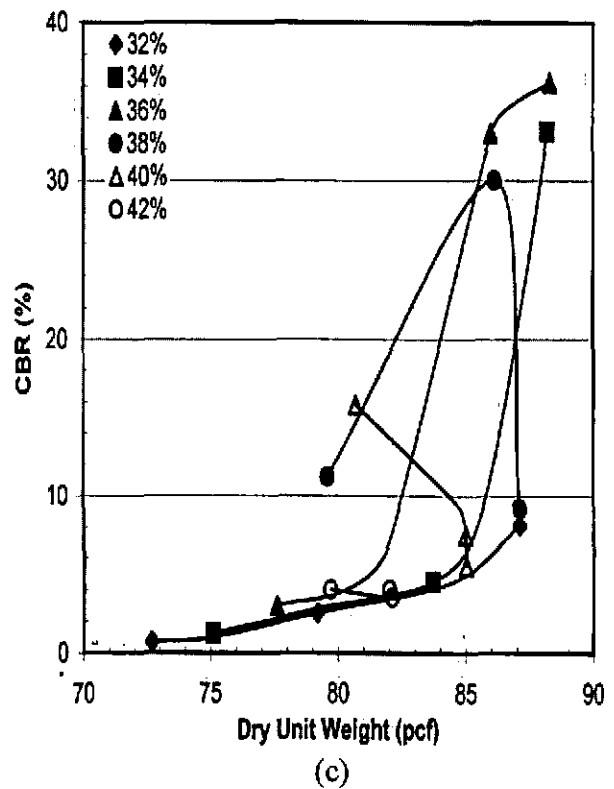
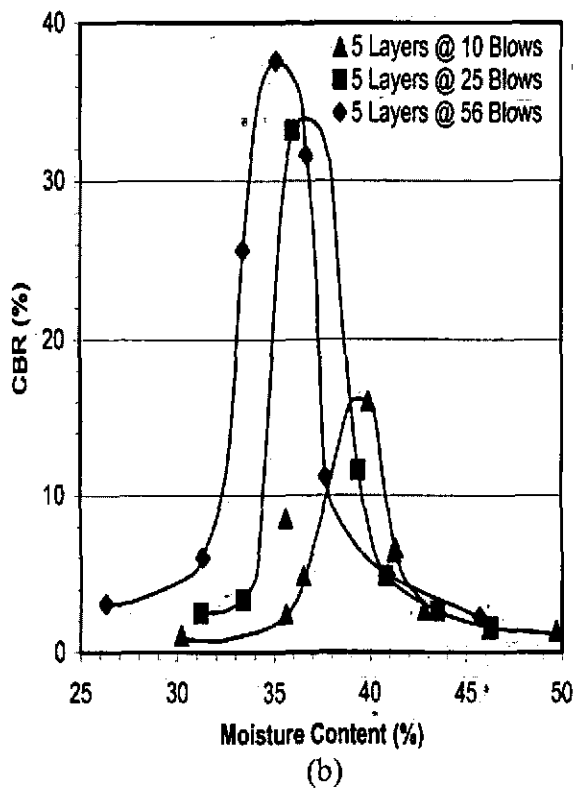
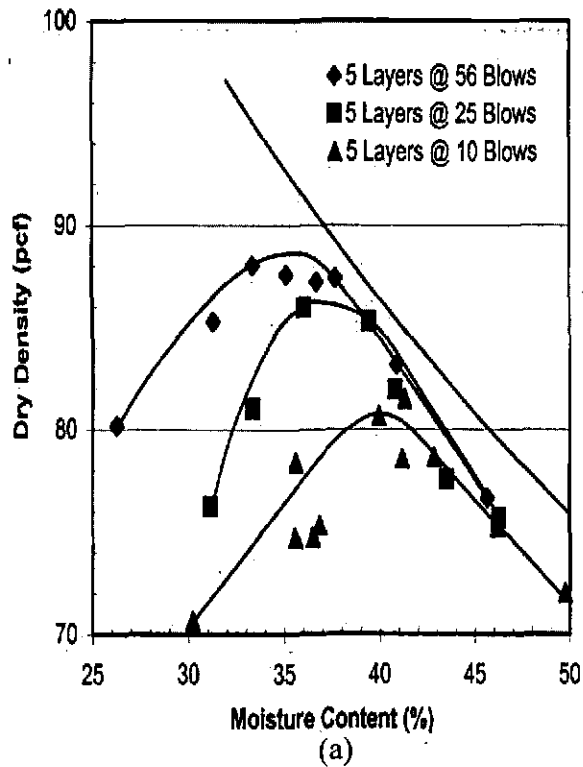


Figure 4.11 CBR family of curves for Wahiawa ovendry (a) Dry unit weight versus moisture content; (b) CBR versus moisture content; and (c) CBR versus dry unit weight at constant moisture content

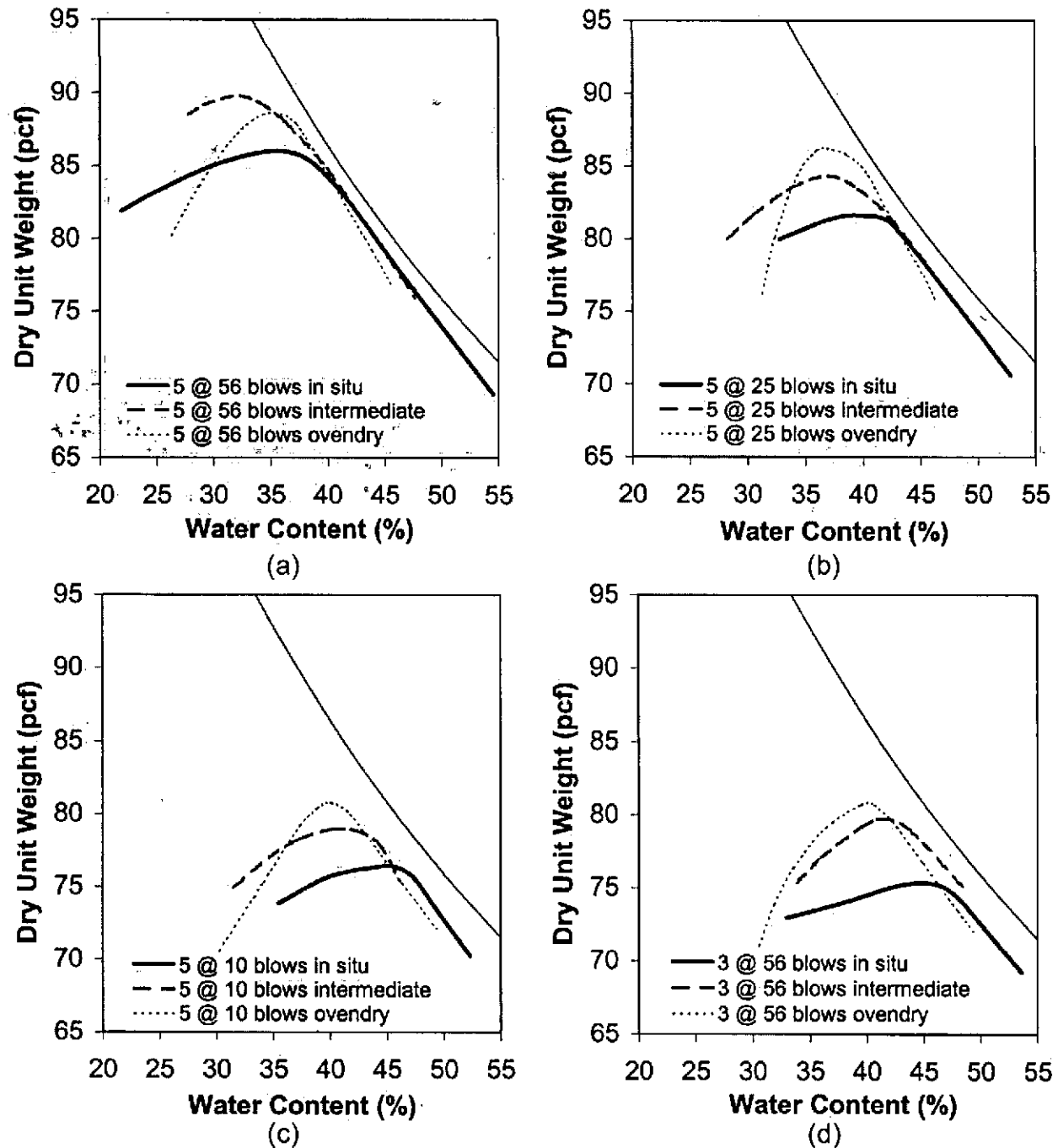


Figure 4.12 Effect of Drying on Compaction Curves for Wahiawa Soil (a) 5 layers @ 56 blows (b) 5 layers @ 25 blows (c) 5 layers @ 10 blows and (d) 3 layers @ 56 blows

CHAPTER 5 R-VALUE TESTING AND RESULTS

5.1 Test Program

R-value tests were performed at the Hawaii Department of Transportation (HDOT) Materials Testing and Research Laboratory. A HDOT certified technician, Mr. Robert Fukuda, performed all R-value tests in accordance with ASTM D2844-01. Between 6 and 15 tests were performed for each soil sample over a wide range of exudation pressures to provide sufficient data to determine R-values at exudation pressures of 240 psi (1,655 kPa) and 300 psi (2,068 kPa). In the following section, adopted procedures that deviate from ASTM D2844-01 or features that pertain to this project are discussed below.

5.2 Test Procedure

5.2.1 Equipment and Sample Preparation

Major components of the R-value test equipment include:

1. kneading compactor (Figure 5.1)
2. exudation indicator device and loading frame with soil press (Figure 5.2)
3. Hveem stabilometer (Figure 5.3)

A single 5-gallon bucket of soil (approx. 40 lbs (18.14 kg) moist soil) was required to perform R-value tests for each soil. Prior to testing the soil was passed through the No. 4 sieve to remove non-soil particles, mostly roots and other debris. After sample preparation was completed, the soil was placed back into the plastic bags re-sealed to minimize moisture loss. Moisture content tests were again performed on the soil sample upon completion of sample preparation to ensure no moisture loss occurred.

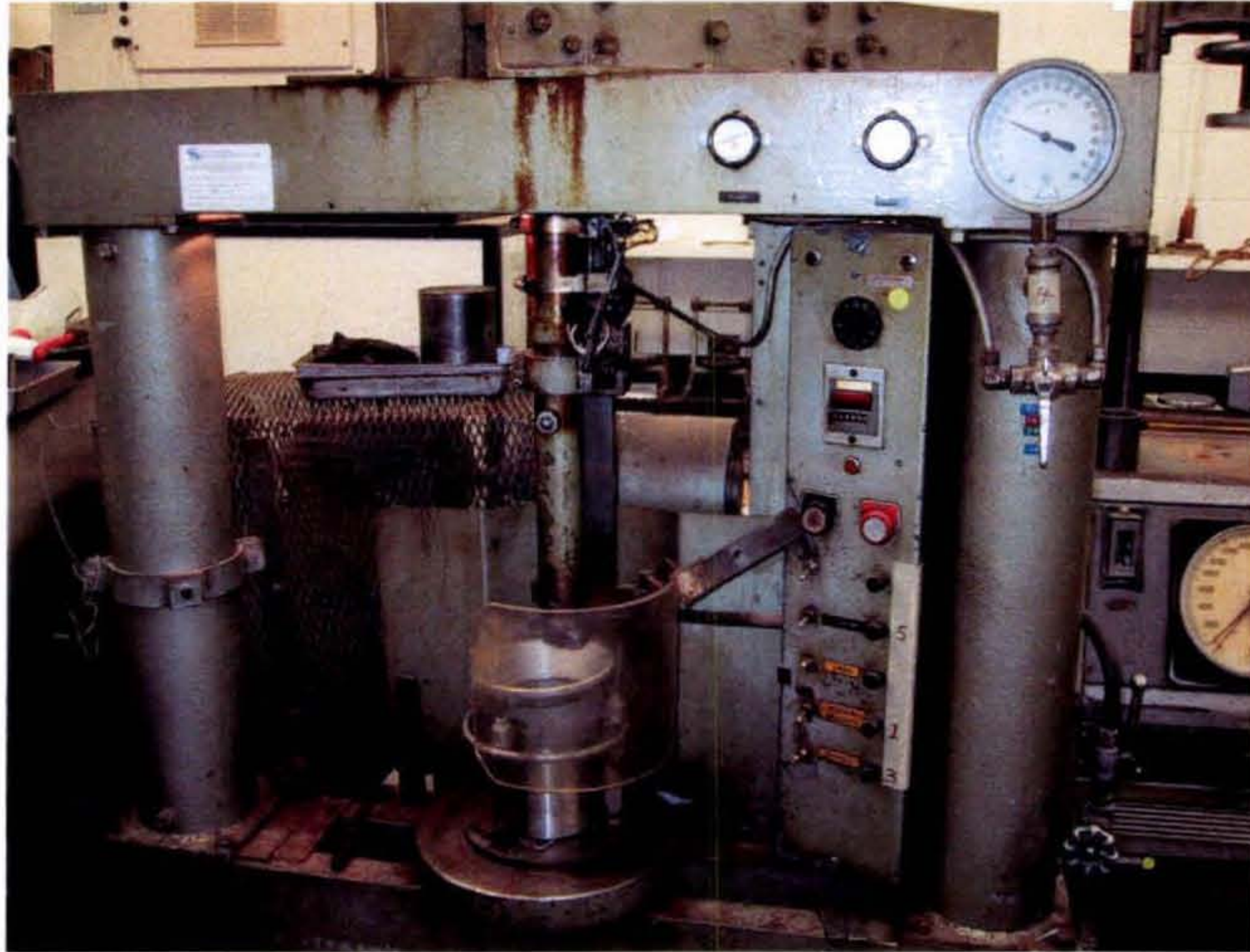


Figure 5.1 Kneading compactor for R-value testing

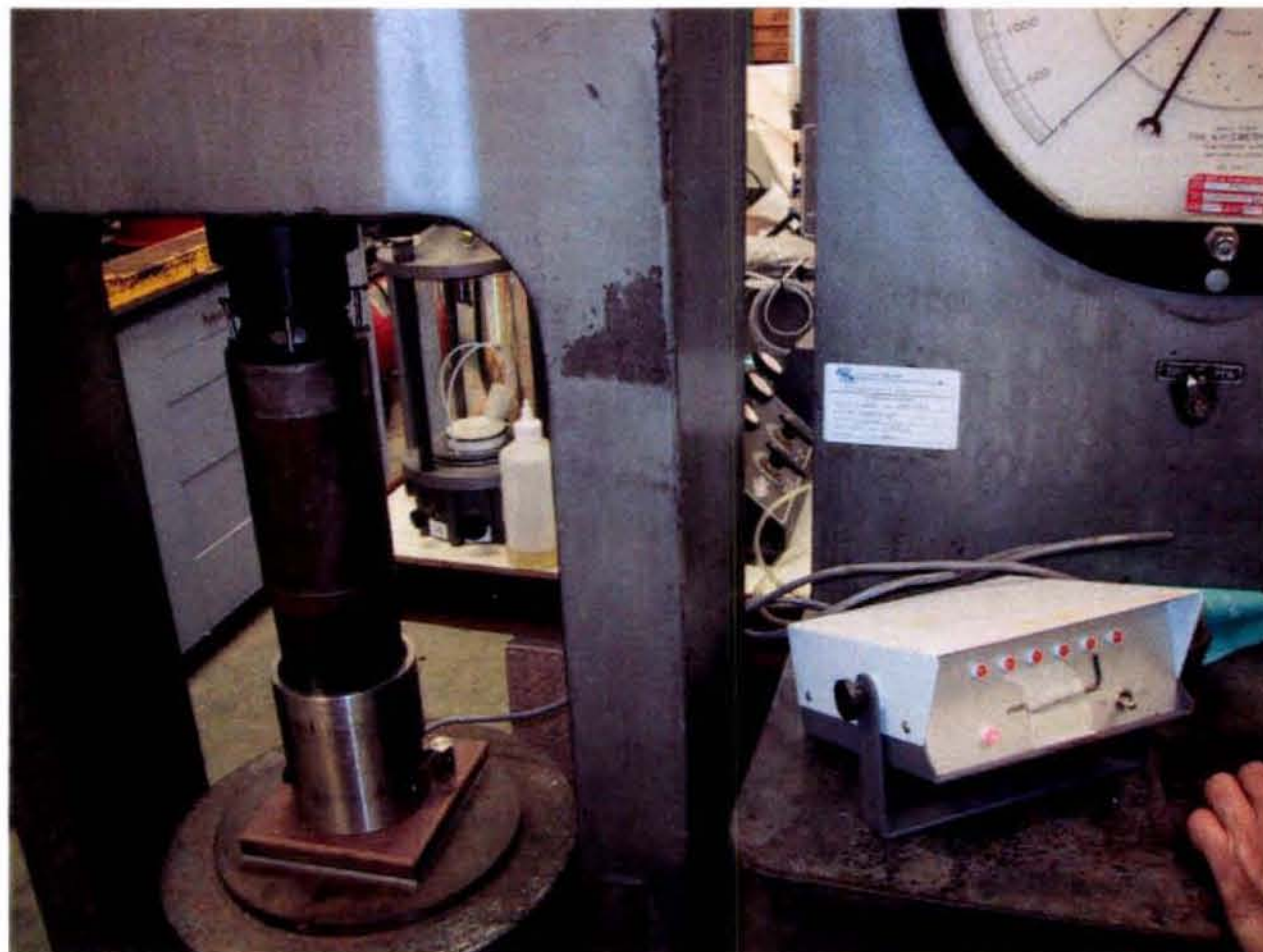


Figure 5.2 Exudation indicator device and loading frame with soil press for R-value testing

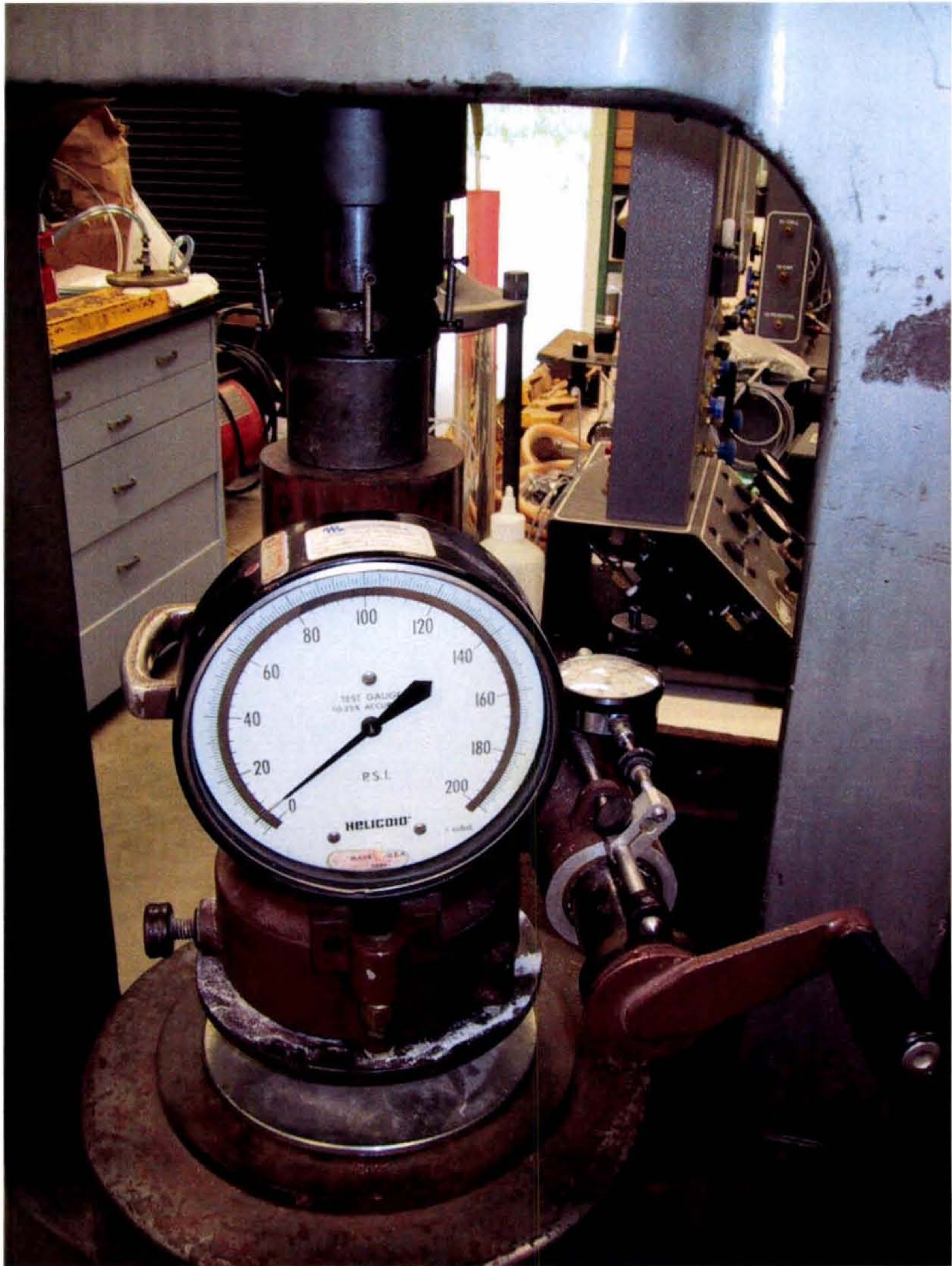


Figure 5.3 Hveem stabilometer device for R-value testing

An individual R-value test sample was prepared by weighing 1000 grams (2.205 lbs) of moist soil. An initial R-value test was performed to obtain a baseline reading of the exudation pressure corresponding to the in situ moisture content. Based on this result, subsequent samples were prepared by adjusting the moisture accordingly to achieve the desired range of exudation pressures (i.e., 100 psi (689 kPa) to 800 psi (5516 kPa)) required to provide R-values at 240 psi (1,655 kPa) and 300 psi (2,068 kPa) exudation pressures.

5.2.2 Compaction

The soil was compacted using a kneading compaction in accordance with ASTM Standard D2844-01. Moisture contents of the same batch of soil as the test specimen were determined. Following compaction, the weight and height of the sample was measured to enable estimation of the dry unit weight prior to exudation portion of the test. The physical state of the soils prior to exudation were observed to be mostly wet of optimum (Figure 5.4), which is consistent with the statement in the Asphalt Institute (1982) that "because of the exudation pressure requirements, specimens for R-value determinations are compacted wet of the line of optimum."

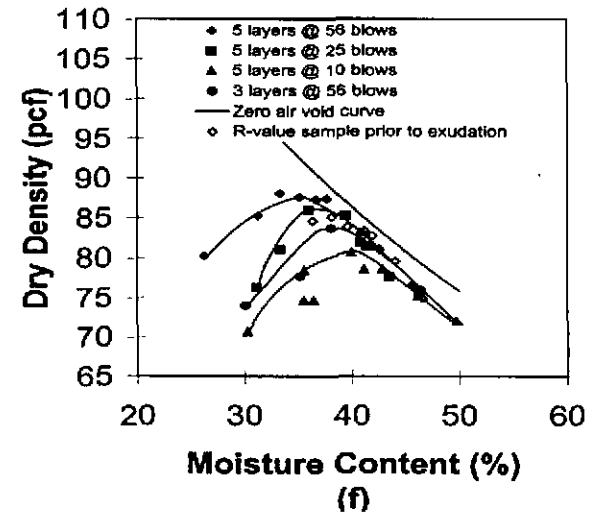
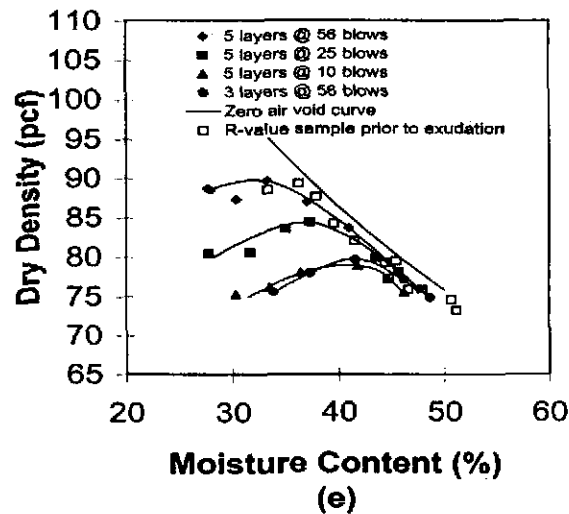
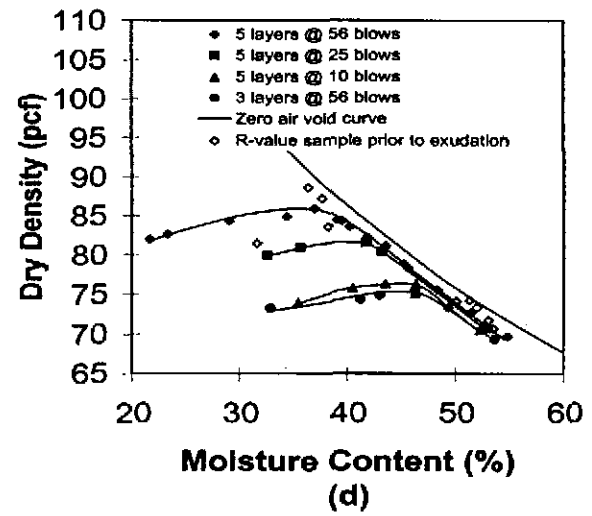
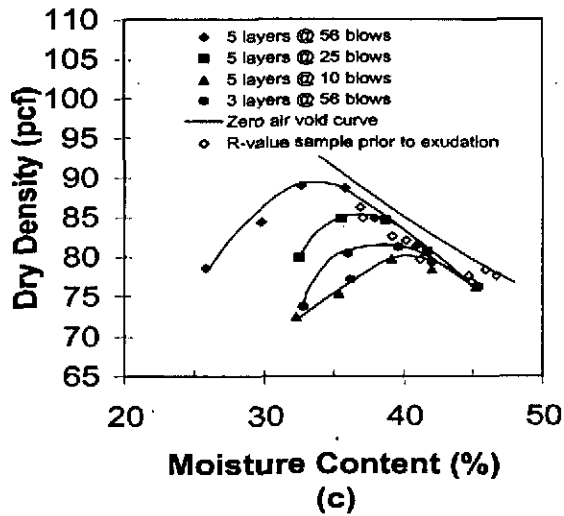
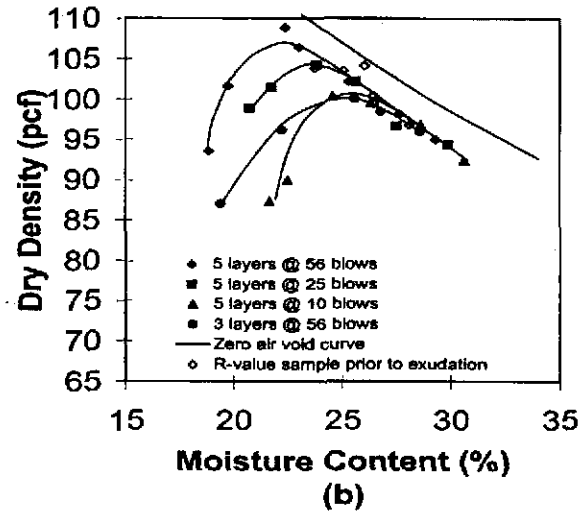
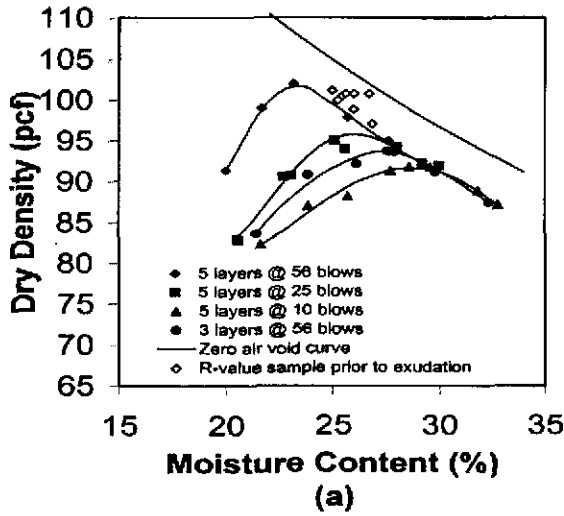


Figure 5.4 Water content and dry unit weight of R-value samples prior to exudation (a) Waipio; (b) Kapolei; (c) Mililani Mauka; (d) Wahiawa in situ; (e) Wahiawa intermediate and (f) Wahiawa oven-dry

5.2.3 Exudation Pressure

After measuring the weight and height of the soil specimen, a phosphor-bronze plate and filter paper were placed on top of the specimen. The mold was inverted and then placed on the exudation device. Prior to pushing the sample down and commencing the exudation pressure test, a light grade oil was placed on the inside of the mold to aid in the sample push down. The coating of oil was used for the higher plasticity samples because the specimen were found to stick to the steel mold, thus deforming the sample during exudation and rendering the sample useless for R-value testing.

Upon completion of the exudation pressure test, the specimen was placed on the expansion apparatus and the initial height of the sample was determined. 200mL of water was then placed in the mold for 24 hours. Expansion pressure measurements were not recorded for this research project because access to the facilities were not available after business hours.

5.2.4 Resistance-Value Testing

After 24 hours, the water was drained and the specimen was air-dried. It was observed that during extrusion of the higher plasticity specimens from the mold to the stabilometer, the specimen would stick to the sides of the neoprene rubber diaphragm causing it to bulge at the bottom. This was observed during testing of the Mililani Mauka and Wahiawa samples but not the Waipio and Kapolei samples. Therefore, for the Mililani Mauka and Wahiawa samples, the neoprene rubber diaphragm was coated with a light grade oil to aid in advancing the specimen into the Hveem stabilometer.

After the specimen was extruded, a vertical pressure was applied at a rate of 0.05 inches per minute (1.27 mm per min) until it reached 160 psi (1,103 kPa) or 2000 lb (907

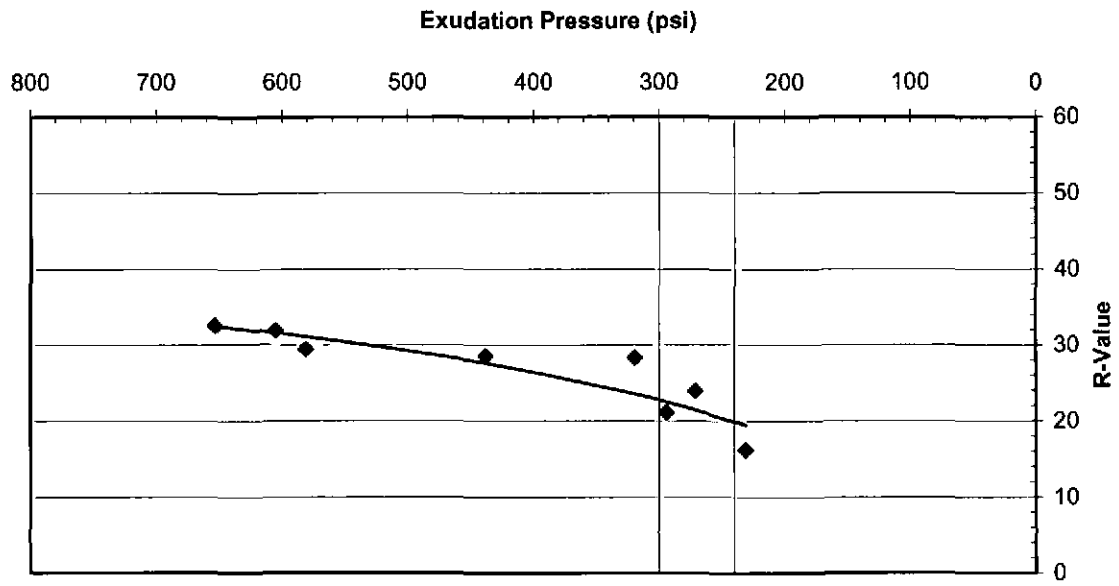
kg). At this stress, the horizontal pressure was recorded. Then the vertical load was reduced by half followed by a reduction in the horizontal pressure to 5 psi (35 kPa) using the displacement pump (see Figure 2.2 and Figure 5.3). The number of turns required to increase the horizontal pressure to 100 psi (689 kPa) was determined. According to Oglesby and Hicks (1982), the intent of this displacement procedure is to measure the penetration of the diaphragm into the interstices of the sample. Without this correction, any roughness of the surface of the specimen could result in an error on the R-value.

5.3 Analysis of Test Results

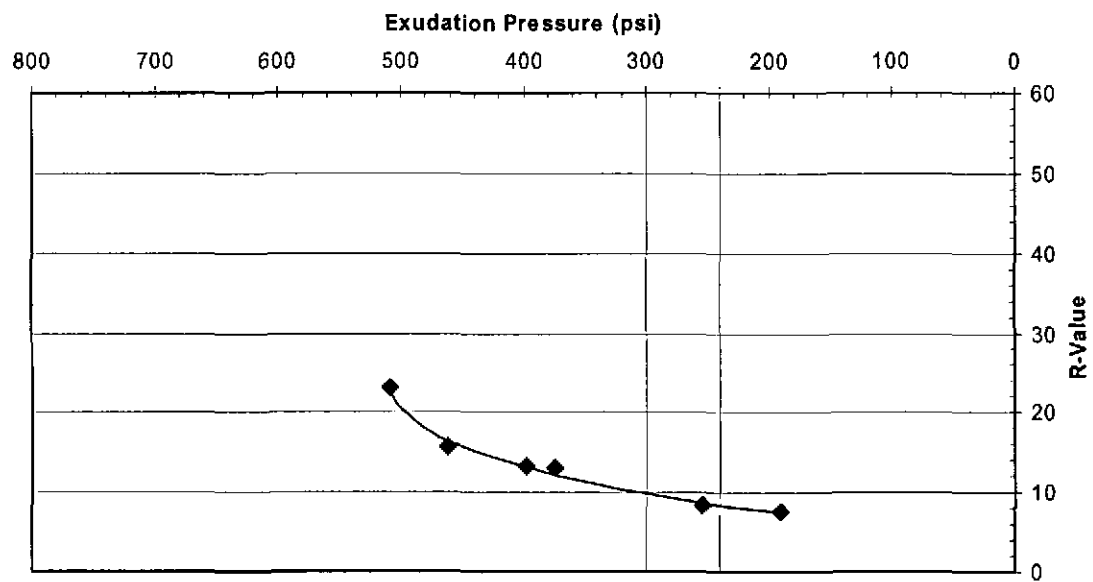
The R-value was calculated using the following equation:

$$R = 100 - \frac{100}{\frac{2.5}{D} \left(\frac{P_v}{P_h} - 1 \right) + 1} \quad (5.1)$$

where R = resistance or R-value, P_v = vertical pressure (160 psi or 1,103 kPa), P_h (psi) = horizontal pressure at P_v = 160 psi (1,103 kPa), and D = turns displacement reading. If the specimen height is not between 2.45 (62.23 mm) and 2.55 (64.77 mm) inches, a correction for the R-value is required. This chart can be found in ASTM Standard D2844-01. The R-value was plotted versus exudation pressure so that values at exudation pressures of 240 psi (1,655 kPa) and 300 psi (2,068 kPa) can be interpolated (Figure 5.5). Based on these tests, the R-value samples at an exudation pressure of 300 psi (2,068 kPa) were prepared at relative compaction values of between 87% and 99%, and moisture contents of +4% to +13% above optimum. One interesting observation on the R-values for the Wahiawa soil can be made. As the sample is dried out, the R-values tend to increase (see Figs. 5.5). For example at an exudation pressure of 300 psi (2,068 kPa), the R-values varied from 8.3 for the in situ to 10 for intermediate to 20.6 for the oven-dry.

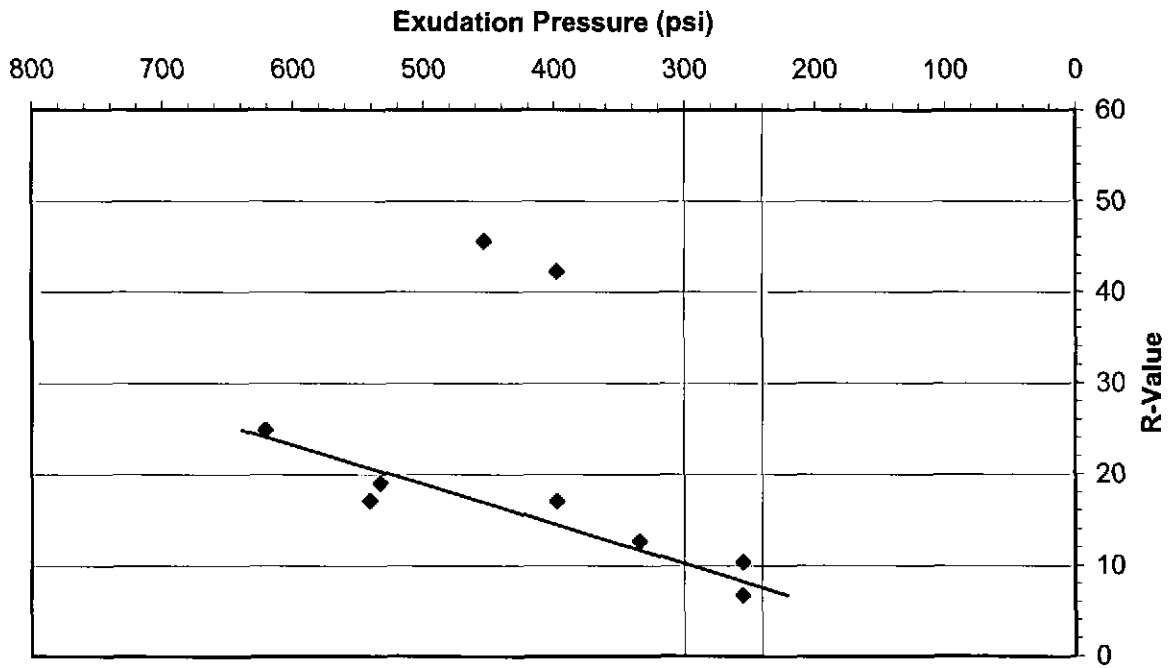


(a)

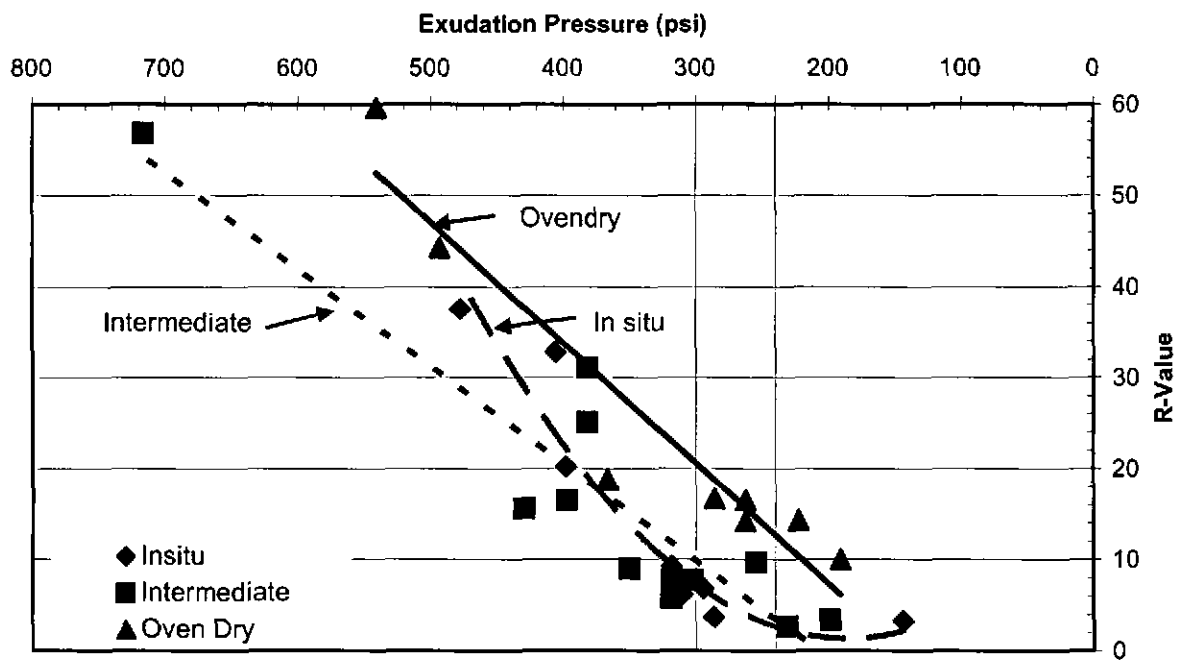


(b)

Figure 5.5 R-value versus exudation pressure for soils from (a) Waipio; (b) Kapolei; (c) Mililani Mauka; (d) Wahiawa



(c)



(d)

Figure 5.5 R-value versus exudation pressure for soils from (a) Waipio; (b) Kapolei; (c) Mililani Mauka; (d) Wahiawa (continued)

CHAPTER 6 CORRELATION ANALYSIS

6.1 Correlations between R-value and CBR

Based on test results on Waipio, Kapolei, Mililani Mauka, Wahiawa in situ and Wahiawa intermediate, four methods of correlating R-value and CBR are presented. A fifth method is also proposed that relates R-value to index properties alone. The Wahiawa oven-dry samples were excluded from the correlation analyses because the samples were dried to temperature extremes that regular soils do not experience, and therefore are judged to be inappropriate for inclusion in this work. Nevertheless, the data provided useful insight into the effects of drying on the measured properties.

6.1.1 Method 1

Linear relationships between R-value and CBR are plotted in Figs. 6.1 through 6.26. Charts were developed for 3 relative compactions (95% dry-of-optimum, 100% or optimum, and 95% wet-of-optimum), 4 compaction efforts (5 layers at 56 blows, 5 layers at 25 blows, 5 layers at 10 blows and 3 layers at 56 blows) and 2 exudation pressures (240 psi (1654 kPa) and 300 psi (2068 kPa) giving a total of 24 figures. The remaining 2 of the 26 figures are for correlations between the Kentucky CBR and R-value at 2 exudation pressures. There was insufficient data to generate charts for 90% relative compaction as the limited quantity of soil available precluded the extension of the compaction curves.

As a result of omitting the Wahiawa oven-dry points, only five data points were used in each regression analysis. The slopes and intercepts are summarized in Table 6.1 along with the coefficients of determination or R^2 .

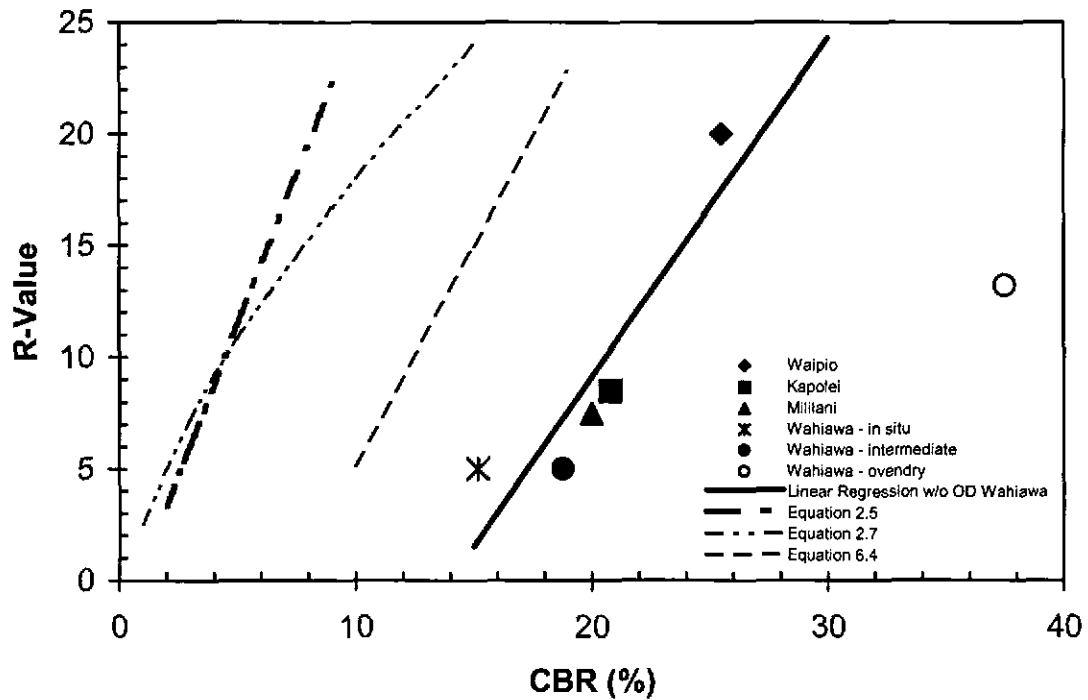


Figure 6.1 CBR vs. R-Value ($EP^1 = 240$ psi, 5 Layers @ 56 Blows, $RC^1 = 100\%$)

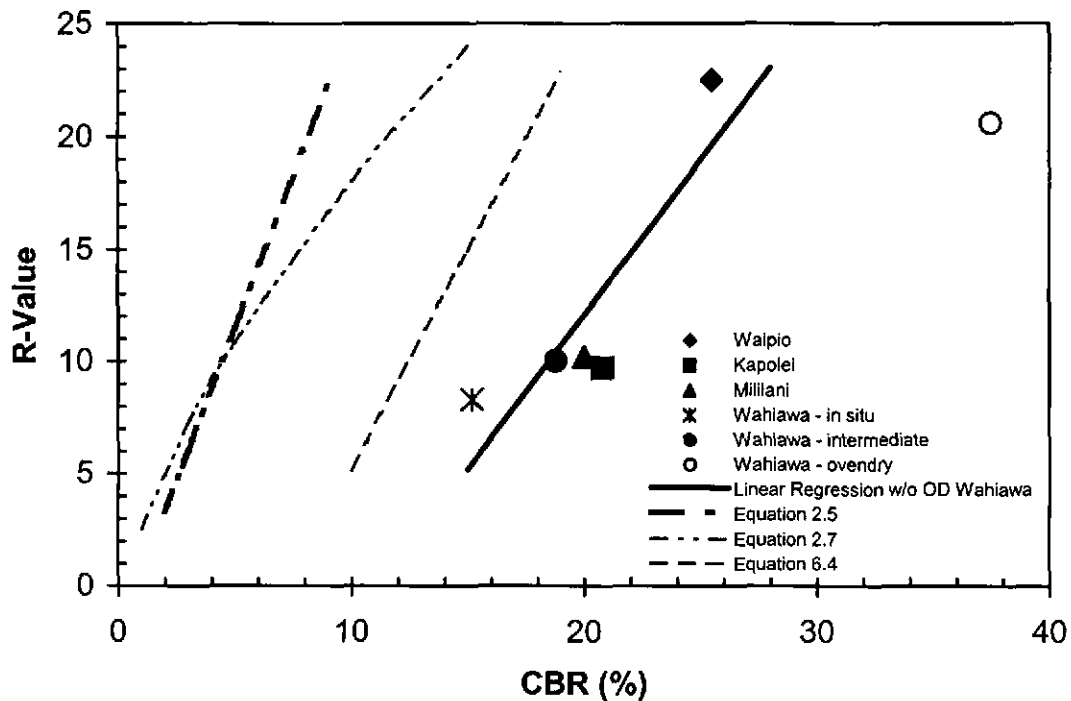


Figure 6.2 CBR vs. R-Value ($EP = 300$ psi, 5 Layers @ 56 Blows, $RC = 100\%$)

Note 1. EP = exudation pressure and RC = relative compaction.

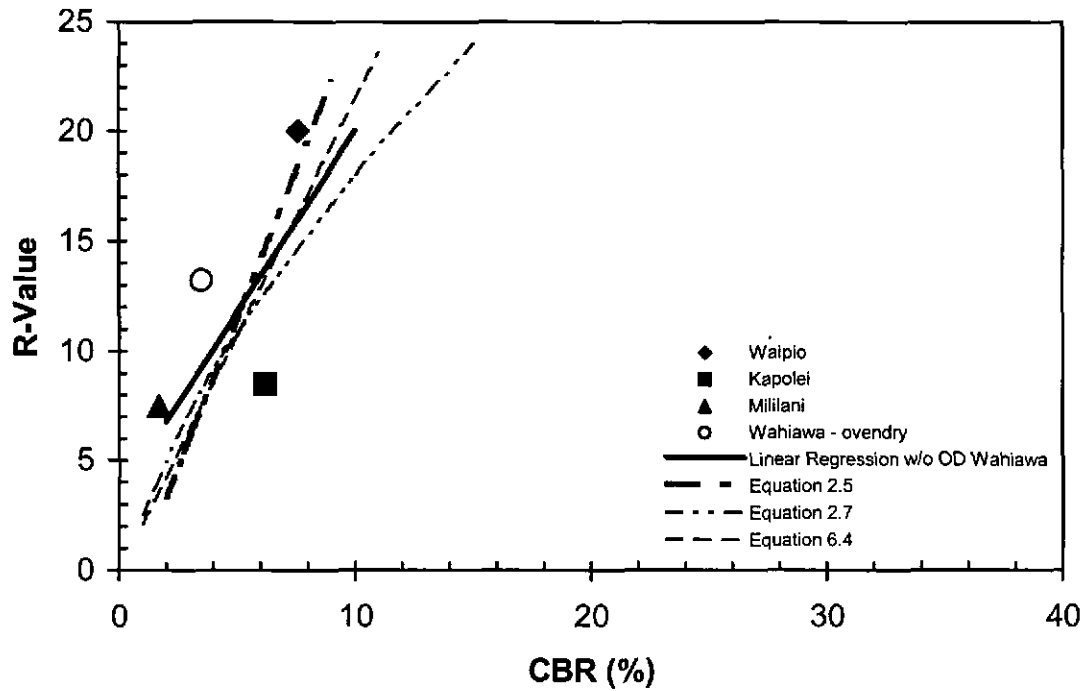


Figure 6.3 CBR vs. R-value (EP = 240 psi, 5 Layers @ 56 Blows, RC = 95% Dry)

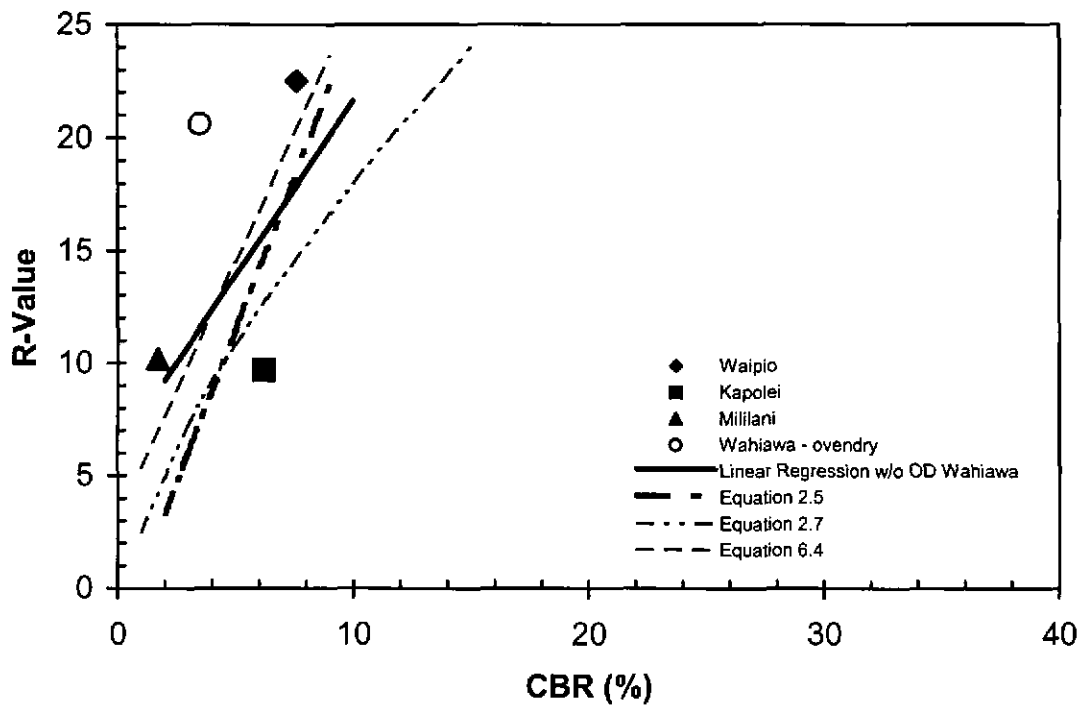


Figure 6.4 CBR vs. R-value (EP = 300 psi, 5 Layers @ 56 Blows, RC = 95% Dry)

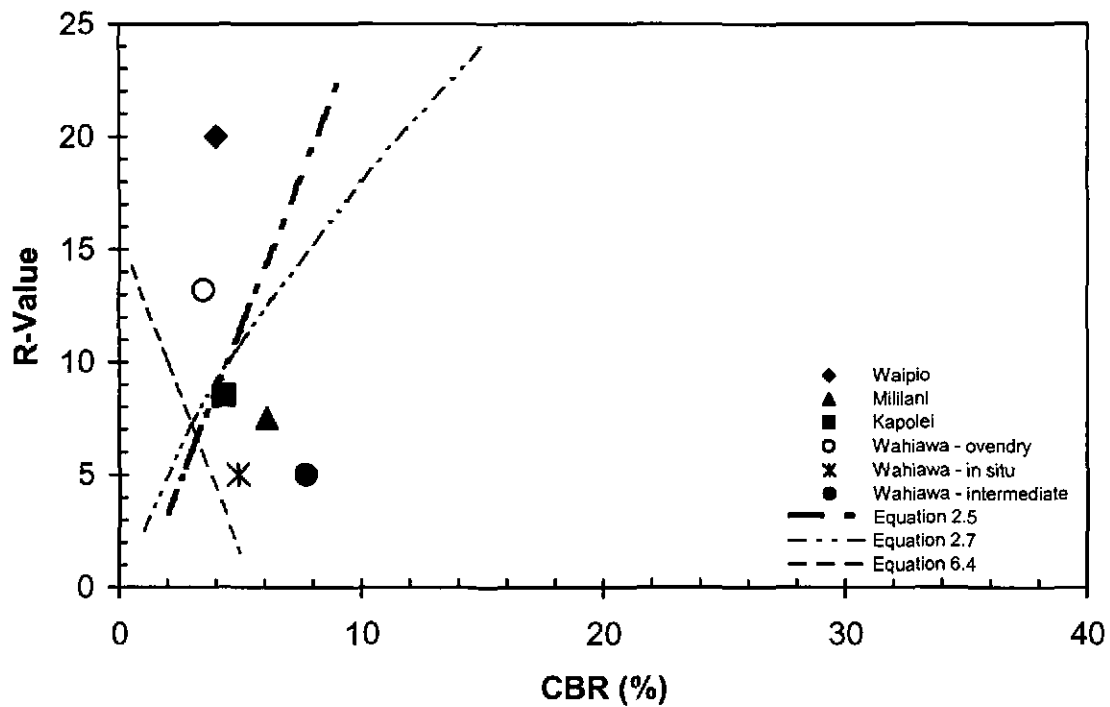


Figure 6.5 CBR vs. R-value (EP = 240 psi, 5 Layers @ 56 Blows, RC = 95% Wet)

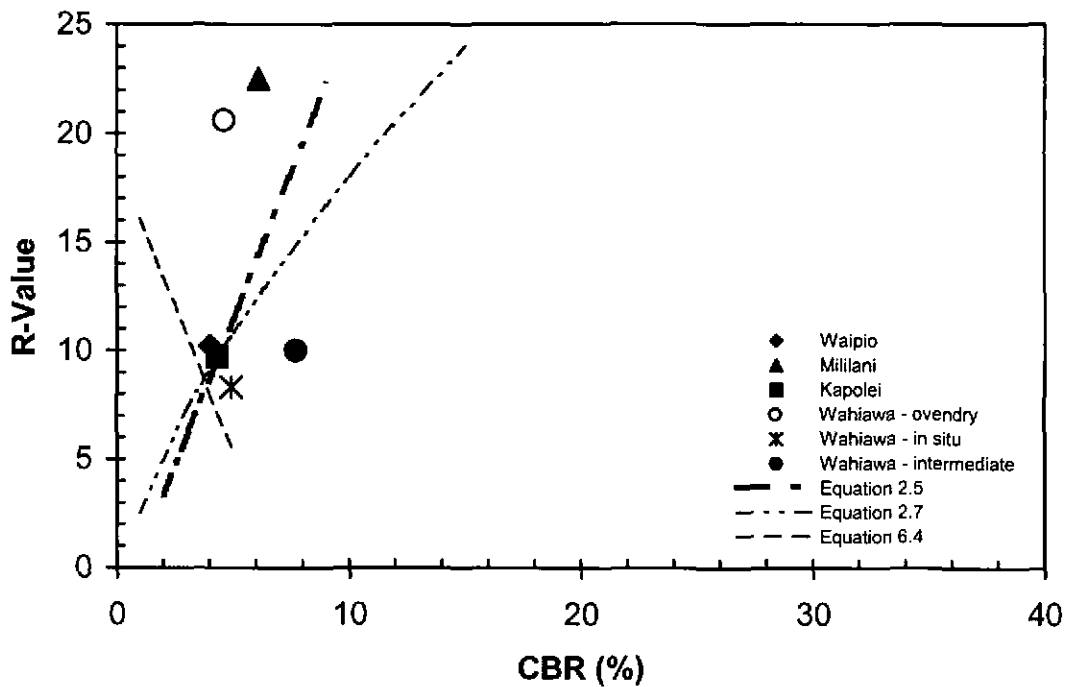


Figure 6.6 CBR vs. R-value (EP = 300 psi, 5 Layers @ 56 Blows, RC = 95% Wet)

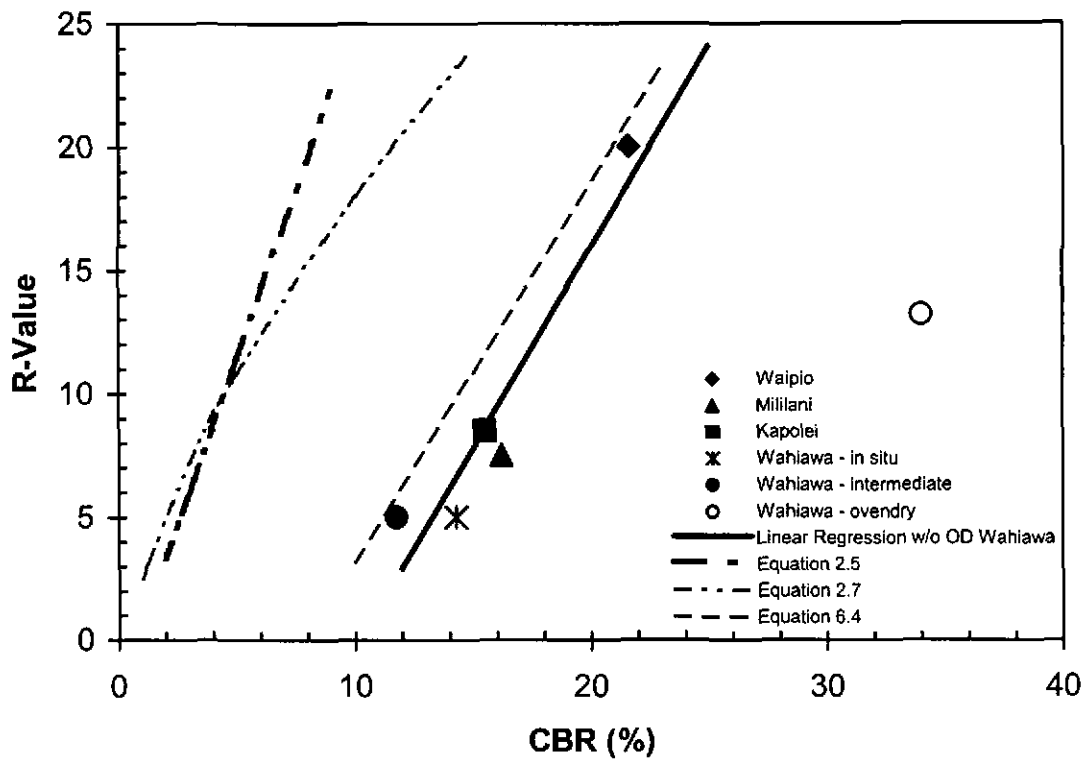


Figure 6.7 CBR vs. R-value (EP = 240 psi, 5 Layers @ 25 Blows, RC = 100%)

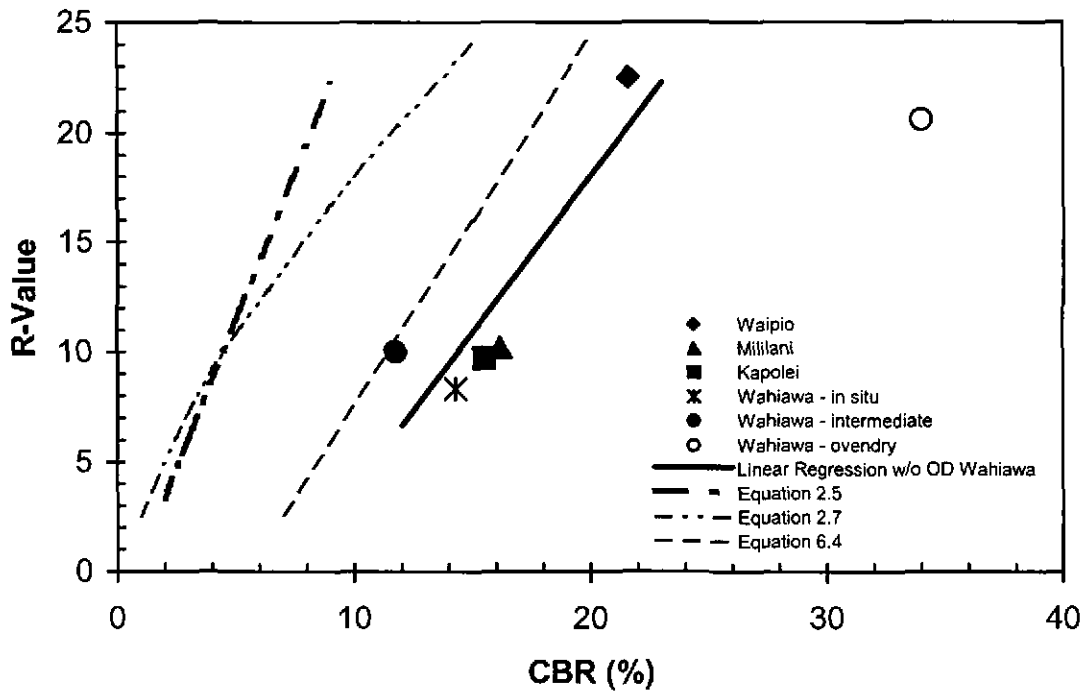


Figure 6.8 CBR vs. R-value (EP = 300 psi, 5 Layers @ 25 Blows, RC = 100%)

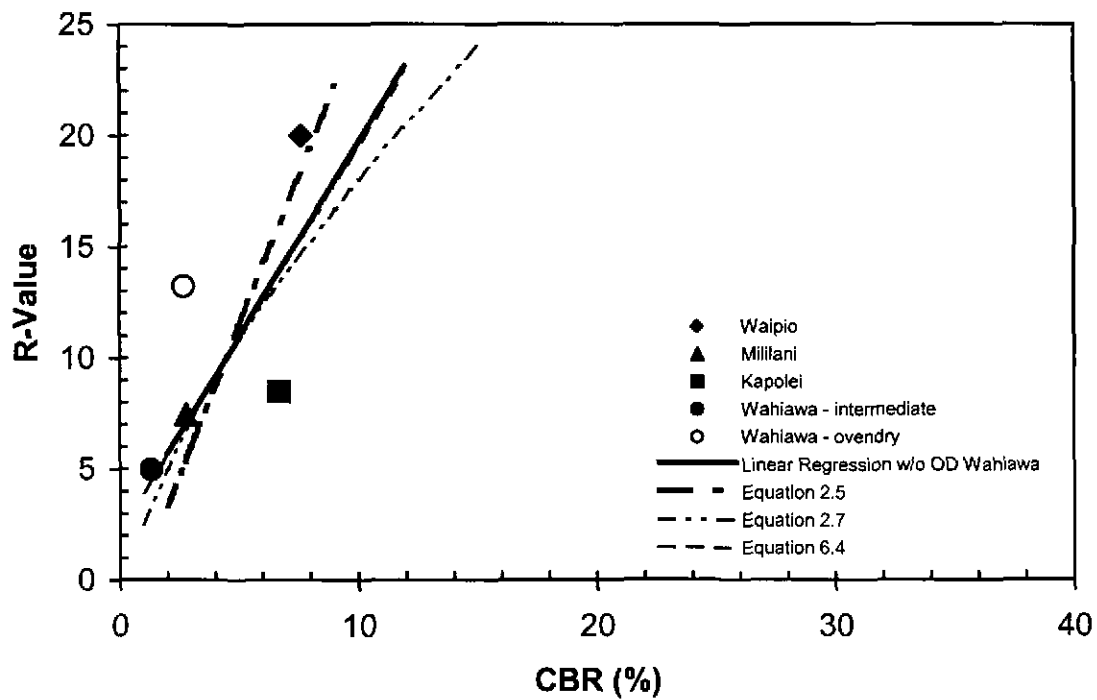


Figure 6.9 CBR vs. R-value (EP = 240 psi, 5 Layers @ 25 Blows, RC = 95% Dry)

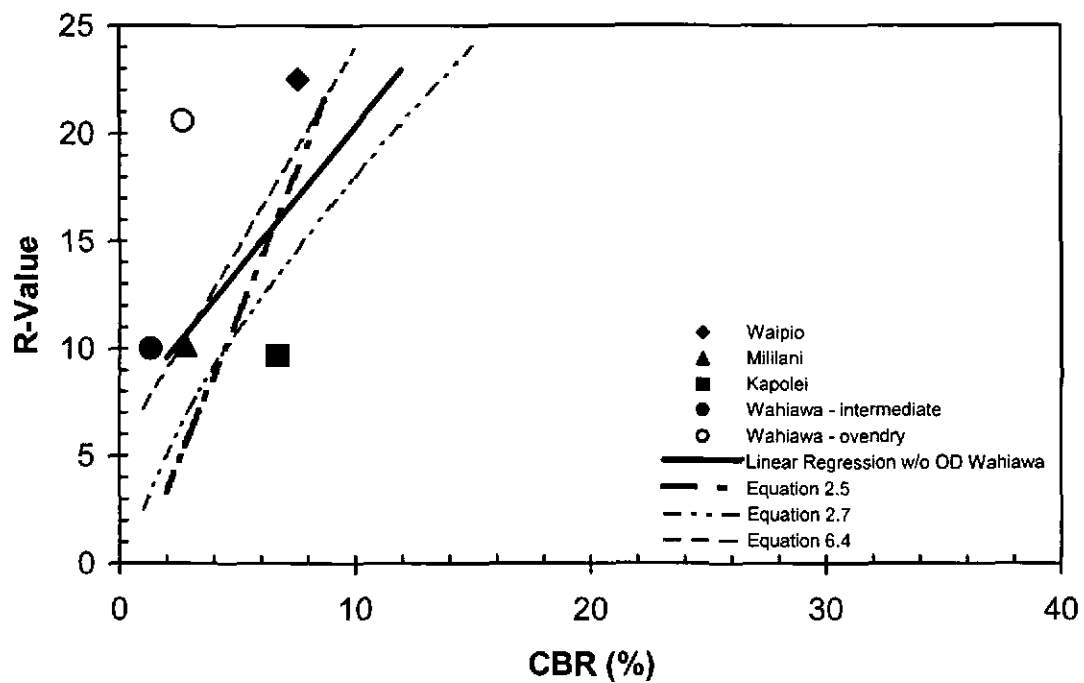


Figure 6.10 CBR vs. R-value (EP = 300 psi, 5 Layers @ 25 Blows, RC = 95% Dry)

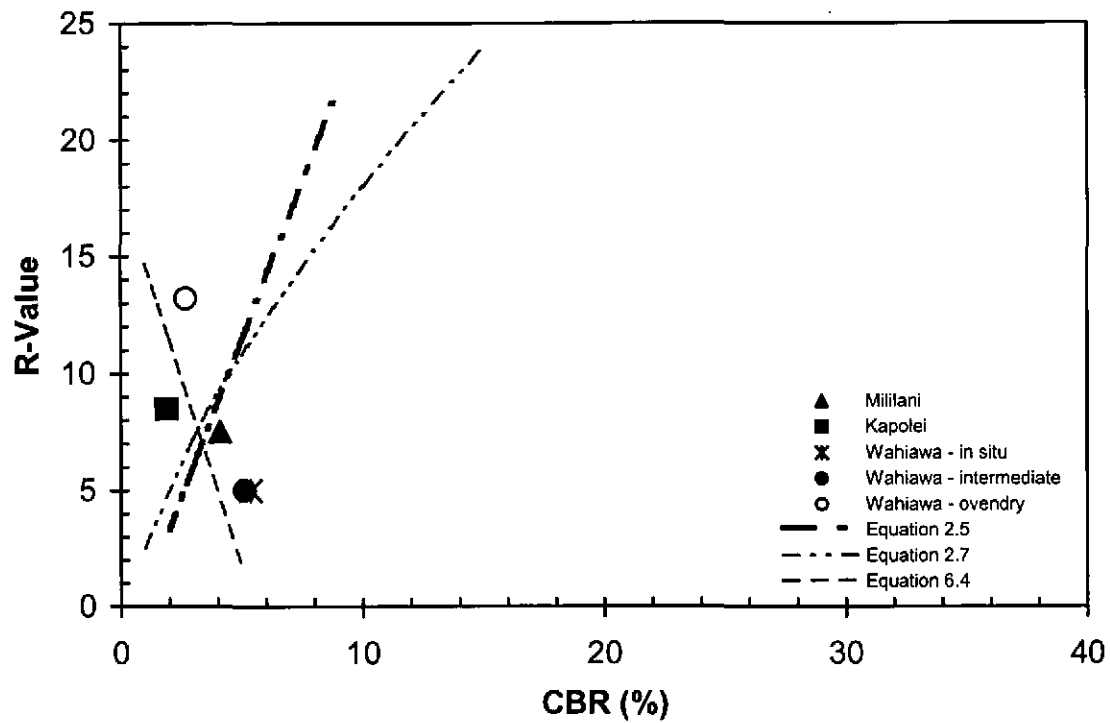


Figure 6.11 CBR vs. R-value (EP = 240 psi, 5 Layers @ 25 Blows, RC = 95% Wet)

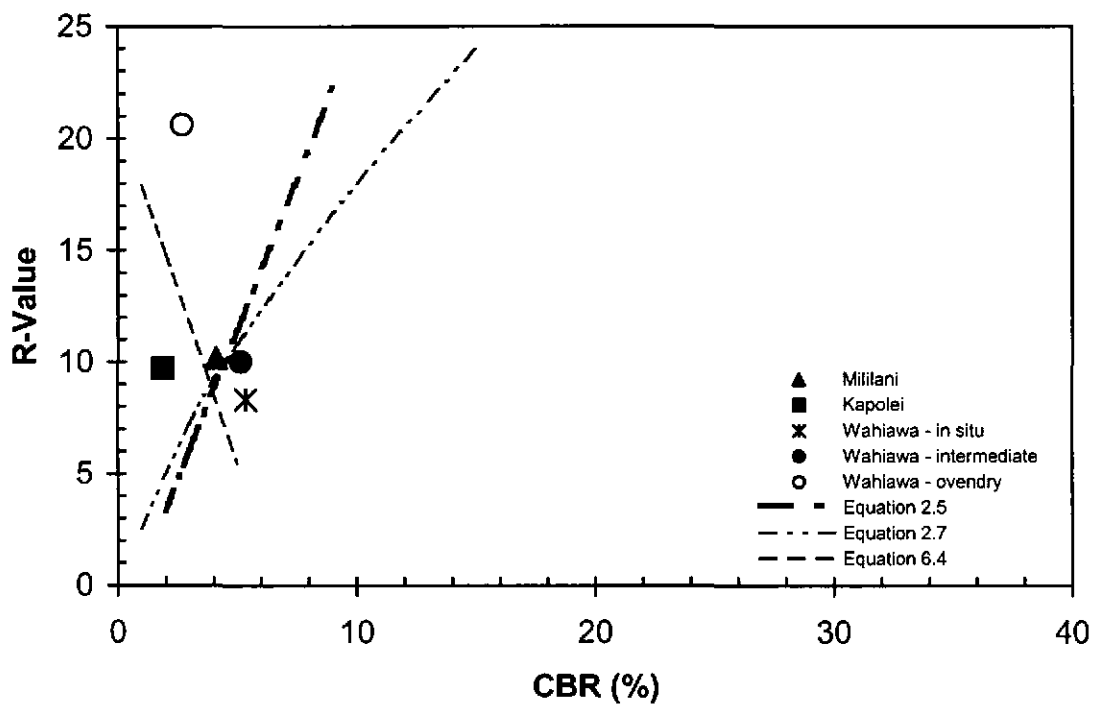


Figure 6.12 CBR vs. R-value (EP = 300 psi, 5 Layers @ 25 Blows, RC = 95% Wet)

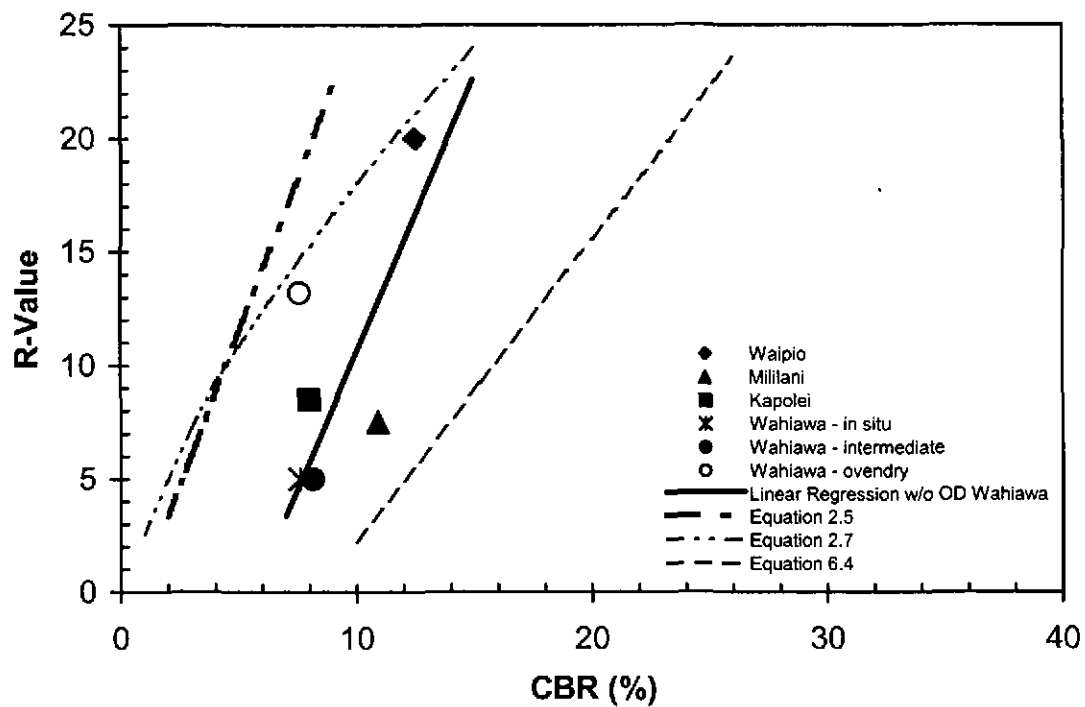


Figure 6.13 CBR vs. R-value (EP = 240 psi, 5 Layers @ 10 Blows, RC = 100%)

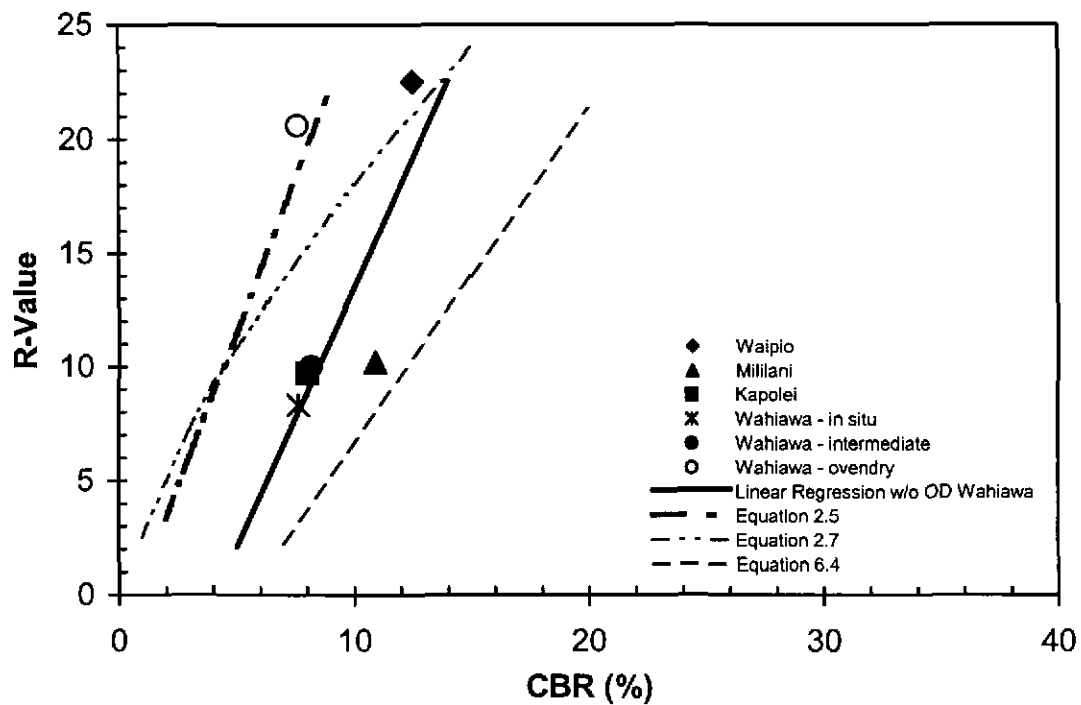


Figure 6.14 CBR vs. R-value (EP = 300 psi, 5 Layers @ 10 Blows, RC = 100%)

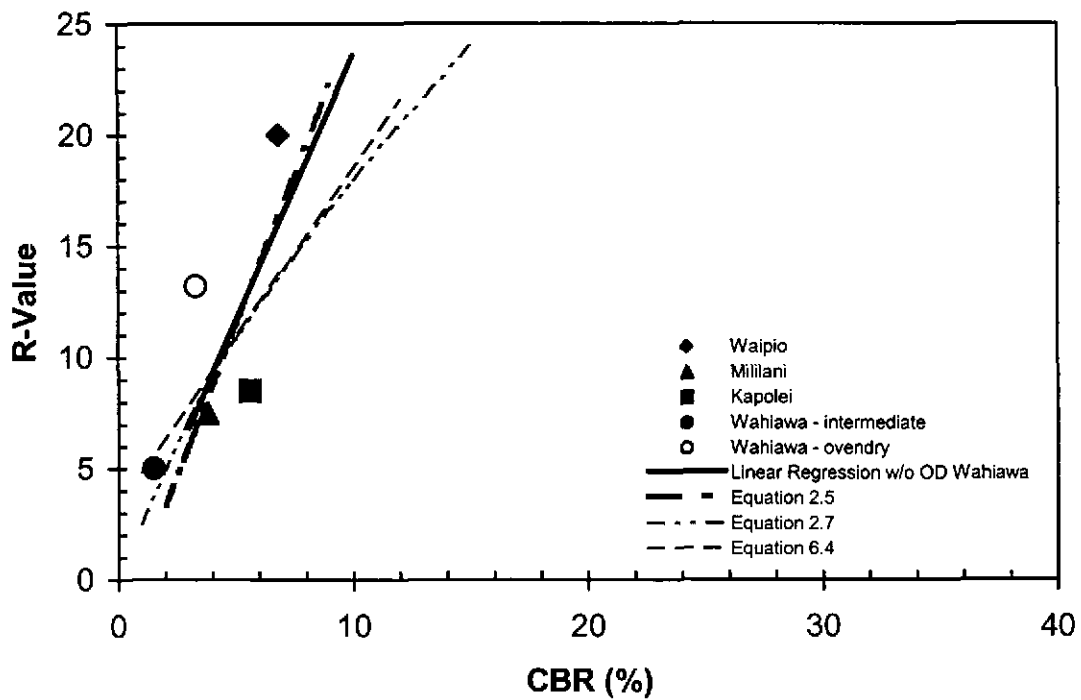


Figure 6.15 CBR vs. R-value (EP = 240 psi, 5 Layers @ 10 Blows, RC = 95% Dry)

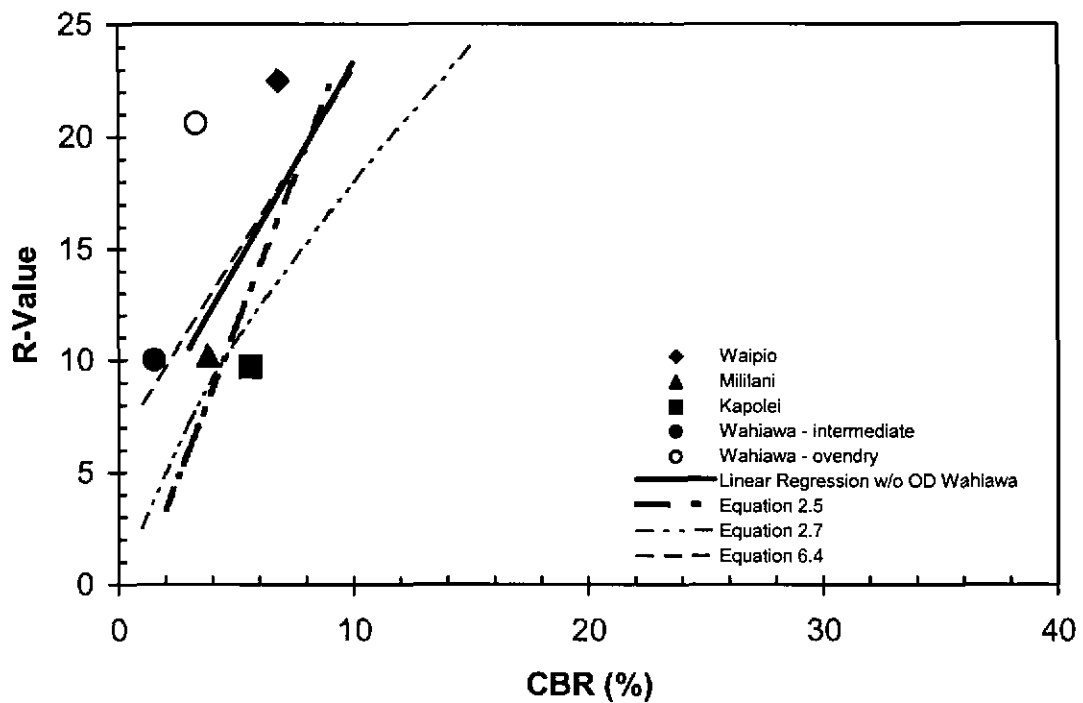


Figure 6.16 CBR vs. R-value (EP = 300 psi, 5 Layers @ 10 Blows, RC = 95% Dry)

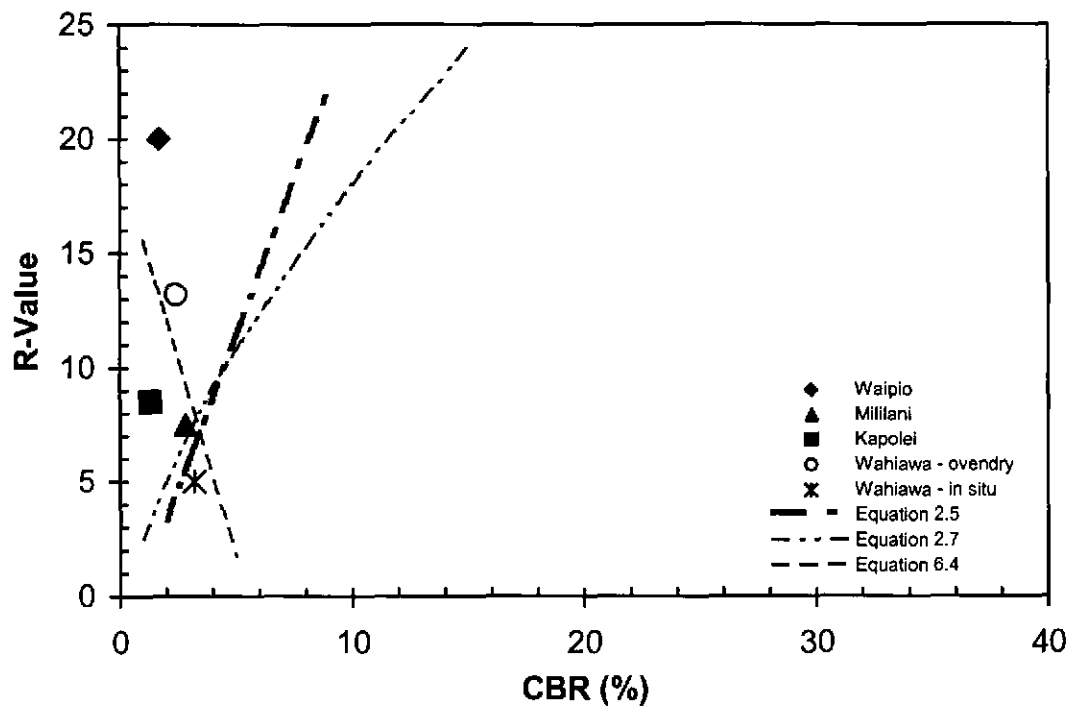


Figure 6.17 CBR vs. R-value (EP = 240 psi, 5 Layers @ 10 Blows, RC = 95% Wet)

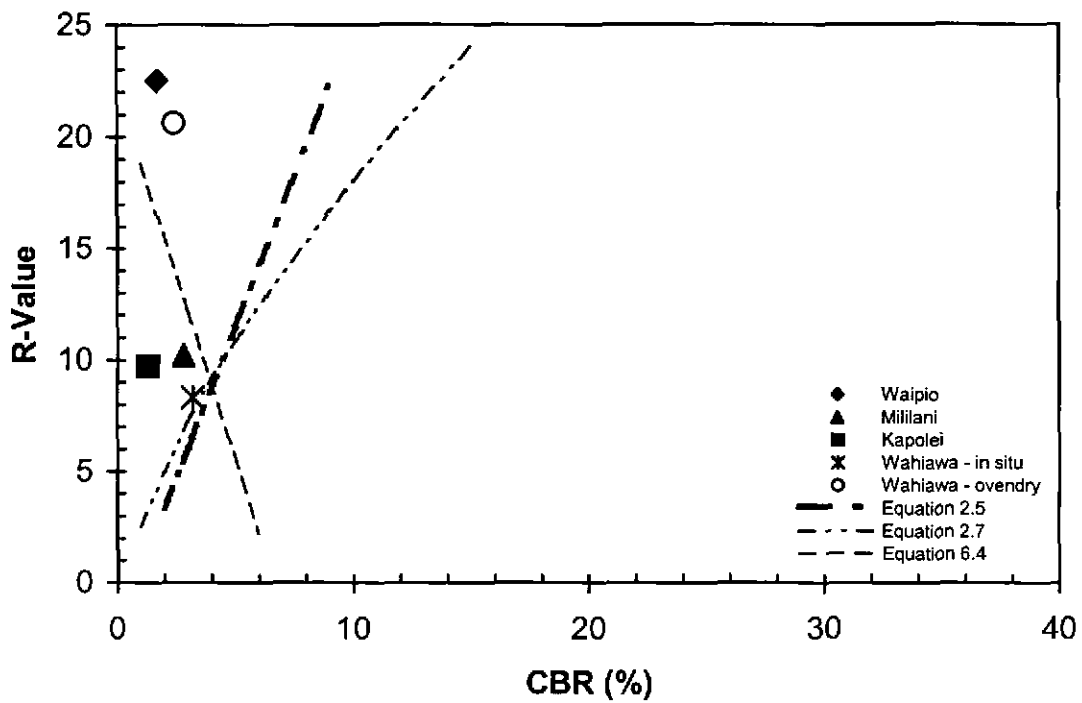


Figure 6.18 CBR vs. R-value (EP = 300 psi, 5 Layers @ 10 Blows, RC = 95% Wet)

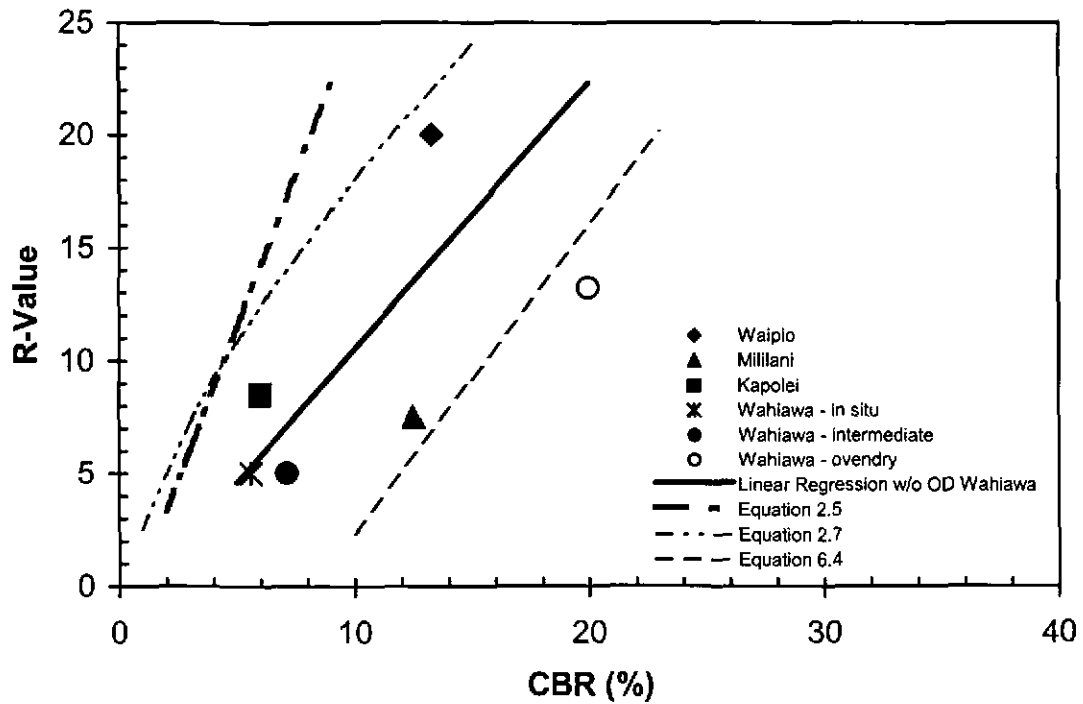


Figure 6.19 CBR vs. R-value (EP = 240 psi, 3 Layers @ 56 Blows, RC = 100%)

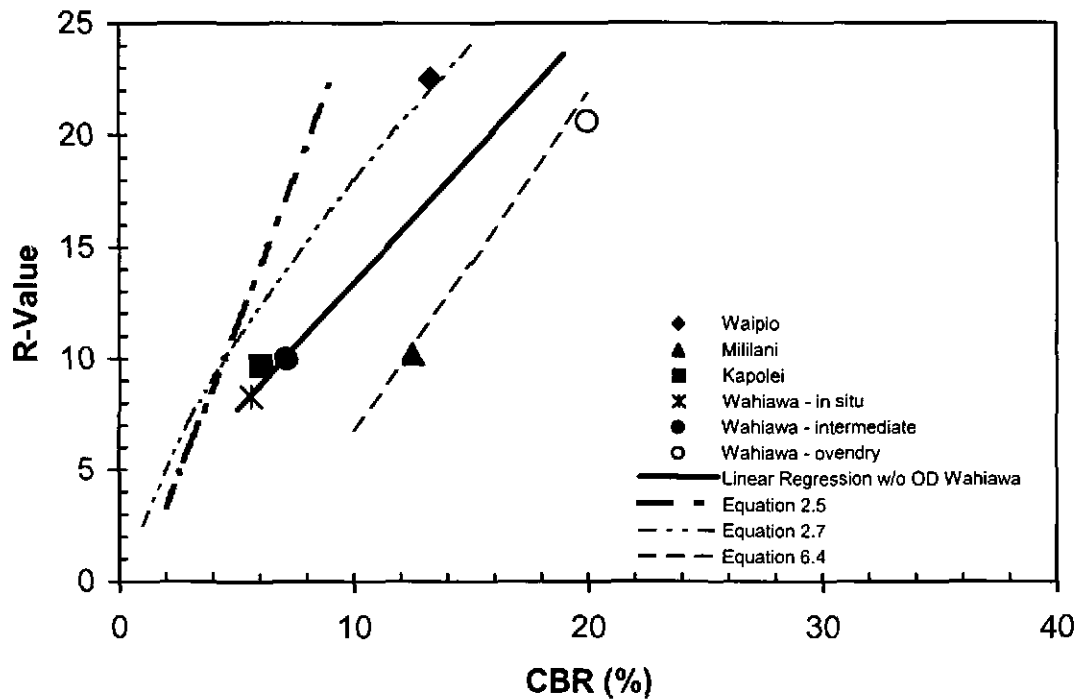


Figure 6.20 CBR vs. R-value (EP = 300 psi, 3 Layers @ 56 Blows, RC = 100%)

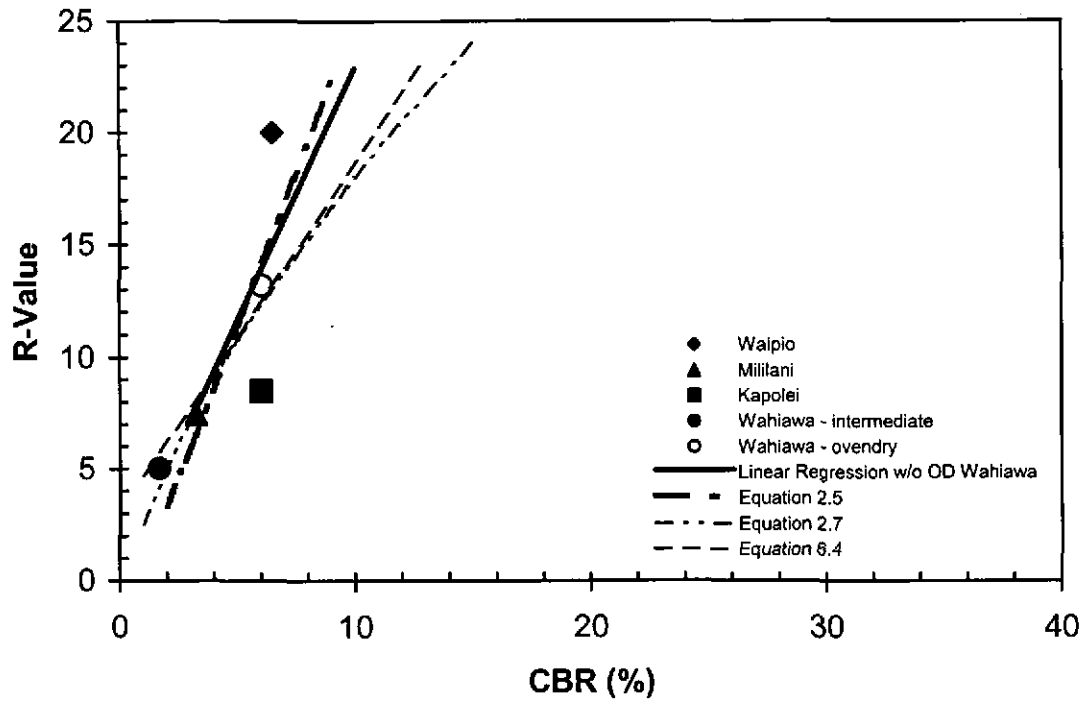


Figure 6.21 CBR vs. R-value (EP = 240 psi, 3 Layers @ 56 Blows, RC = 95% Dry)

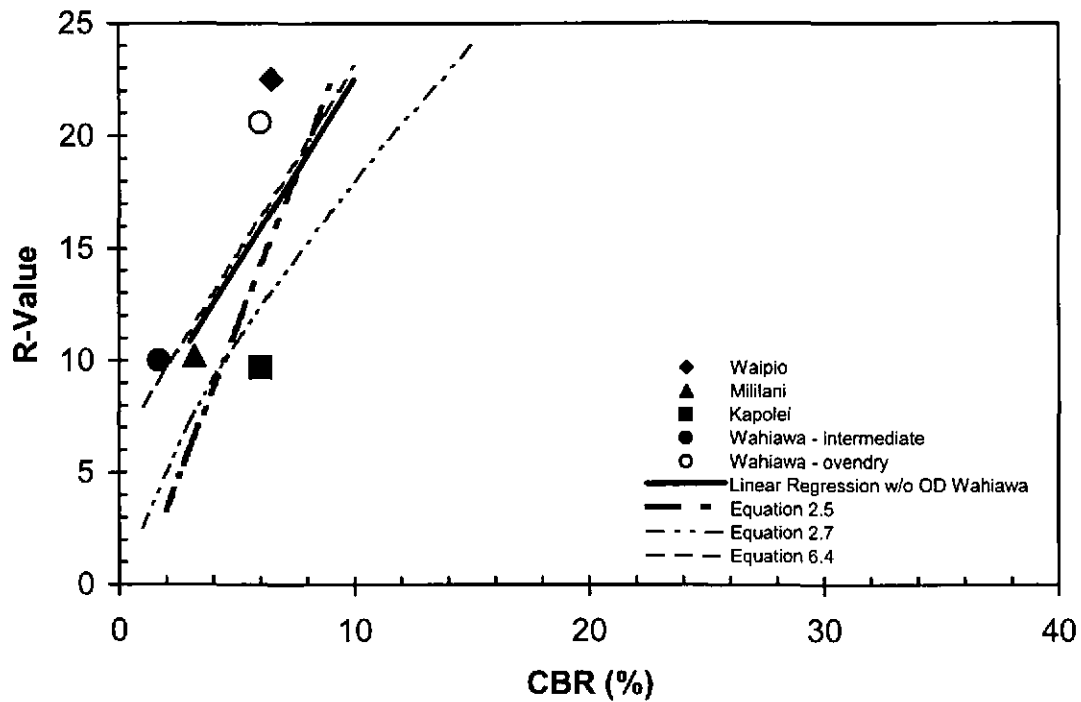


Figure 6.22 CBR vs. R-value (EP = 300 psi, 3 Layers @ 56 Blows, RC = 95% Dry)

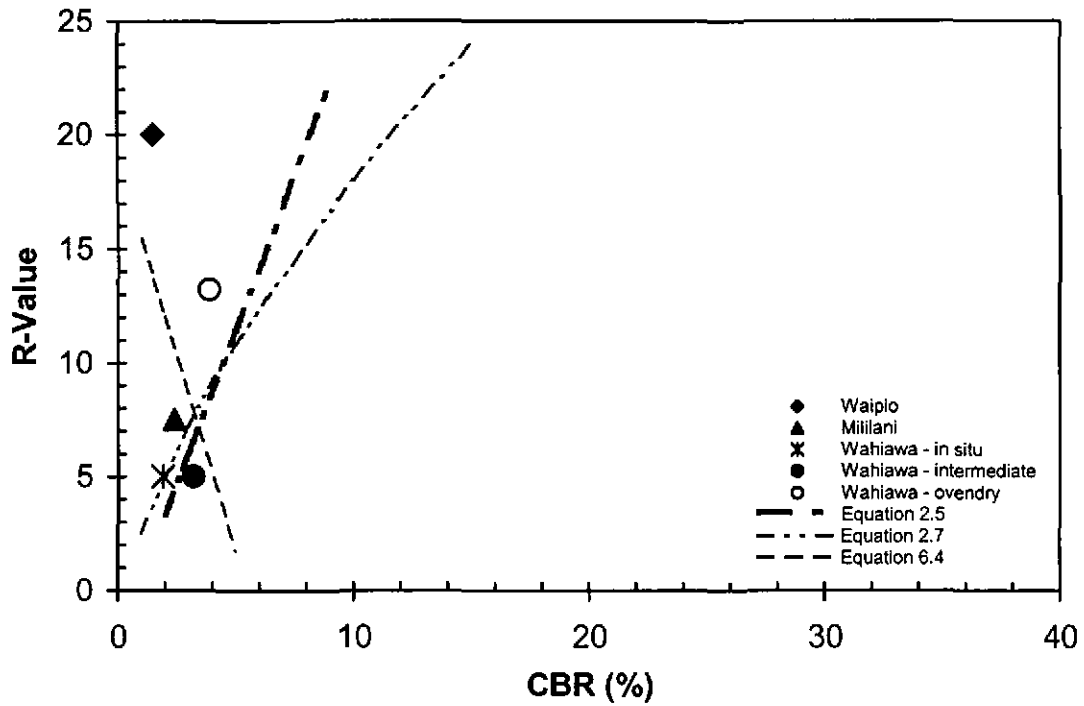


Figure 6.23 CBR vs. R-value (EP = 240 psi, 3 Layers @ 56 Blows, RC = 95% Wet)

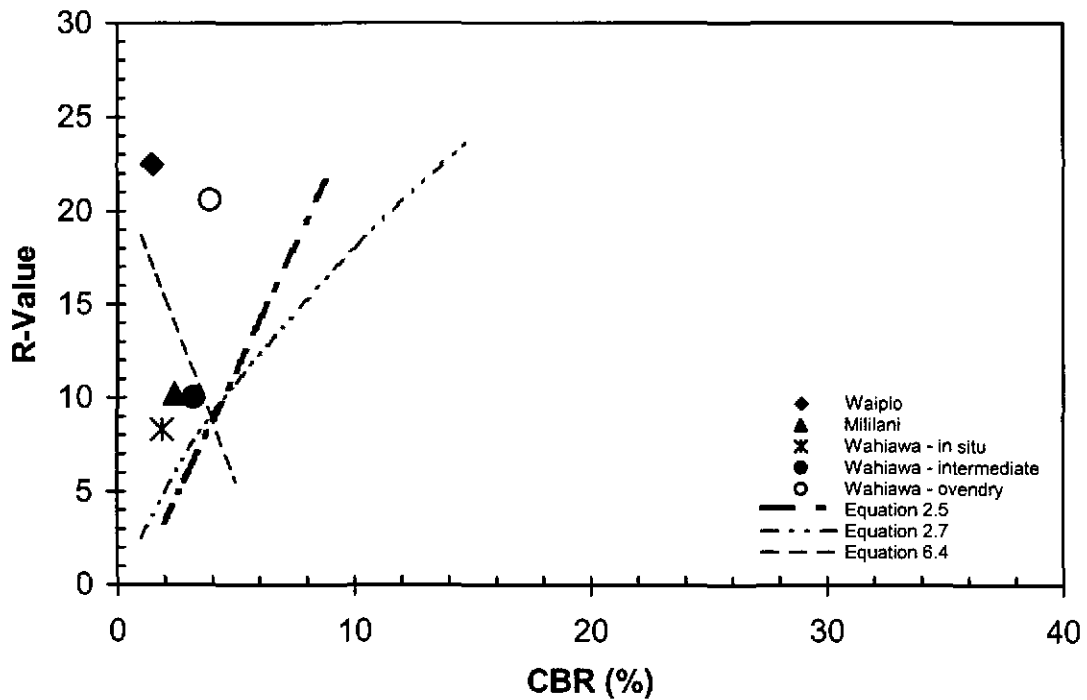


Figure 6.24 CBR vs. R-value (EP = 300 psi, 3 Layers @ 56 Blows, RC = 95% Wet)

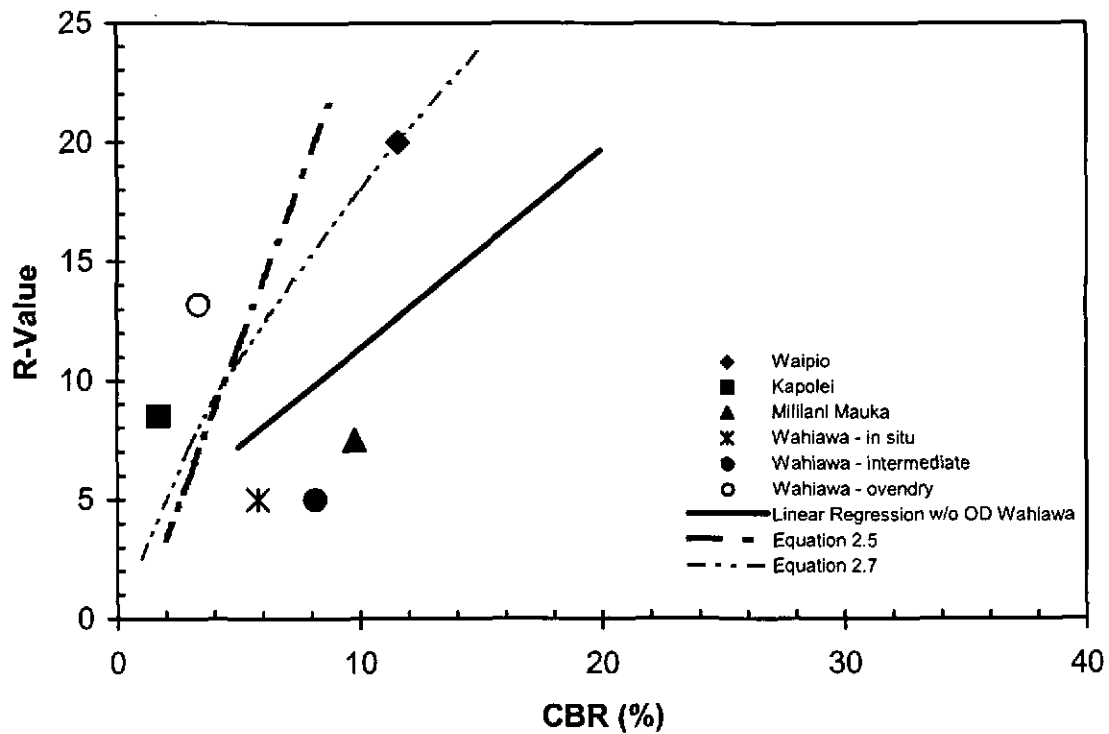


Figure 6.25 CBR vs. R-value (EP = 240 psi, Kentucky CBR)

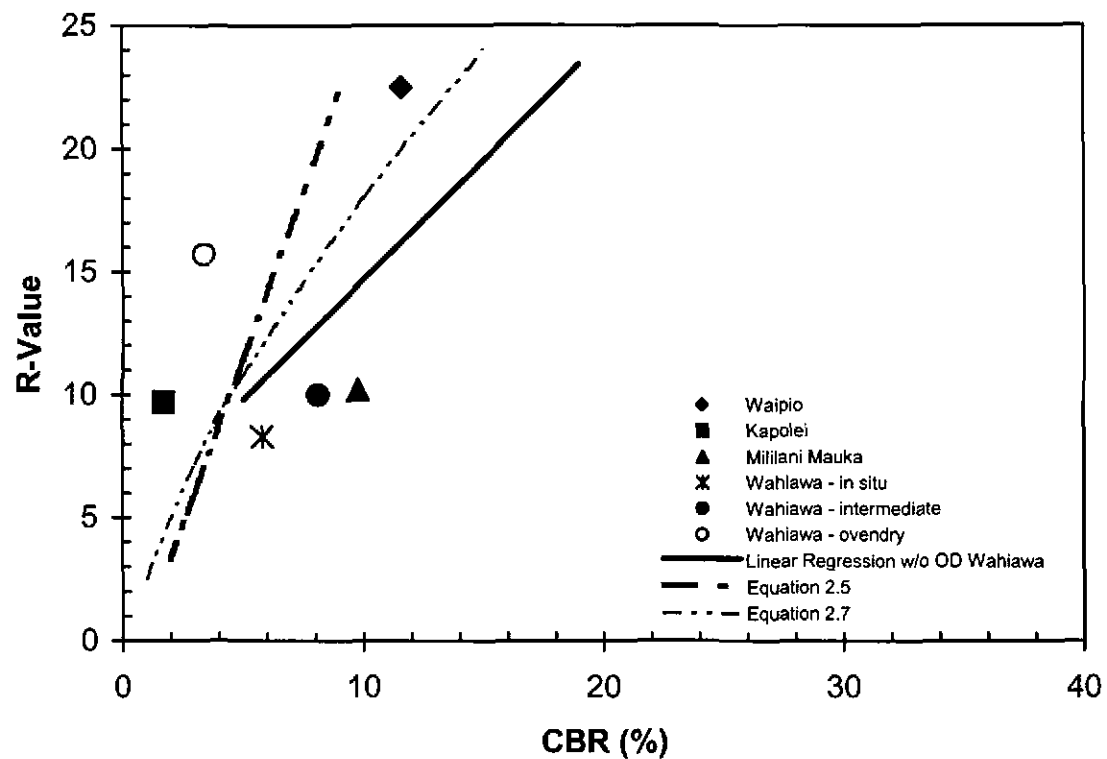


Figure 6.26 CBR vs. R-value (EP = 300 psi, Kentucky CBR)

Table 6.1 Slope and intercept from linear regression of R-value versus CBR without Wahiawa oven dry

Compaction Effort	Physical State	Exudation Pressure (psi)	Slope m	Intercept c	R ² ^a
5 layers, 56 blows	95% RC ^b dry of optimum	240	1.65	3.45	0.539
5 layers, 56 blows	95% RC dry of optimum	300	1.55	6.14	0.433
5 layers, 56 blows	100% RC	240	1.52	-21.3	0.822
5 layers, 56 blows	100% RC	300	1.37	-15.4	0.766
5 layers, 56 blows	95% RC wet of optimum	240	-2.53	22.8	0.374
5 layers, 56 blows	95% RC wet of optimum	300	-1.77	21.7	0.210
5 layers, 25 blows	95% RC dry of optimum	240	1.74	2.23	0.624
5 layers, 25 blows	95% RC dry of optimum	300	1.33	6.98	0.411
5 layers, 25 blows	100% RC	240	1.63	-16.7	0.896
5 layers, 25 blows	100% RC	300	1.42	-10.4	0.442
5 layers, 25 blows	95% RC wet of optimum	240	-1.05	10.8	0.862
5 layers, 25 blows	95% RC wet of optimum	300	-0.194	10.3	0.127
5 layers, 10 blows	95% RC dry of optimum	240	2.40	-0.329	0.685
5 layers, 10 blows	95% RC dry of optimum	300	1.83	5.01	0.453
5 layers, 10 blows	100% RC	240	2.40	-13.5	0.691
5 layers, 10 blows	100% RC	300	2.27	-9.26	0.702
5 layers, 10 blows	95% RC wet of optimum	240	-4.23	19.8	0.324
5 layers, 10 blows	95% RC wet of optimum	300	-3.36	20.2	0.208
3 layers, 56 blows	95% RC dry of optimum	240	2.23	0.553	0.590
3 layers, 56 blows	95% RC dry of optimum	300	1.66	5.89	0.369
3 layers, 56 blows	100% RC	240	1.18	-1.33	0.493
3 layers, 56 blows	100% RC	300	1.13	2.04	0.517
3 layers, 56 blows	95% RC wet of optimum	240	-6.75	24.6	0.475
3 layers, 56 blows	95% RC wet of optimum	300	-5.43	25.0	0.369

Note a. R² = coefficient of determination

b. RC = relative compaction

Note that the trendlines for the wet-of-optimum plots have a negative slope indicating that as CBR increases, R-value decreases. This seems counterintuitive and is perhaps an indication that the CBR for wet-of-optimum samples is less reliable for use in correlating with R-value.

To determine the R-value at a particular exudation pressure, the CBR corresponding to one of the above four compactive efforts and one of the above three

relative compactions is needed. The R-value is obtained using a linear equation ($R = m \text{ CBR} + c$) where m = slope and c = intercept from Table 6.1 or by reading the value from the appropriate graph. The linear regression line was also plotted in Figure 6.1 through 6.26. These lines were omitted for the wet-of-optimum charts because they have a negative slope. Also shown for comparison are curves for equations 2.5 and 2.7. In general, for the range of R-values that was measured (i.e., < 25), equations 2.5 and 2.7 are reasonable for dry-of-optimum soils, overpredict the R-value for soils at optimum, and underpredict the R-value for wet-of-optimum soils.

The R-values determined using Van Til et al's (1972) correlation with the Kentucky CBR is shown in Figure 6.27. Superimposed on this plot are the data obtained from this testing program. While the Van Til et al. correlation predicted the R-value reasonably well for the Kapolei soil, the R-value was overestimated for the remaining 4 soils.

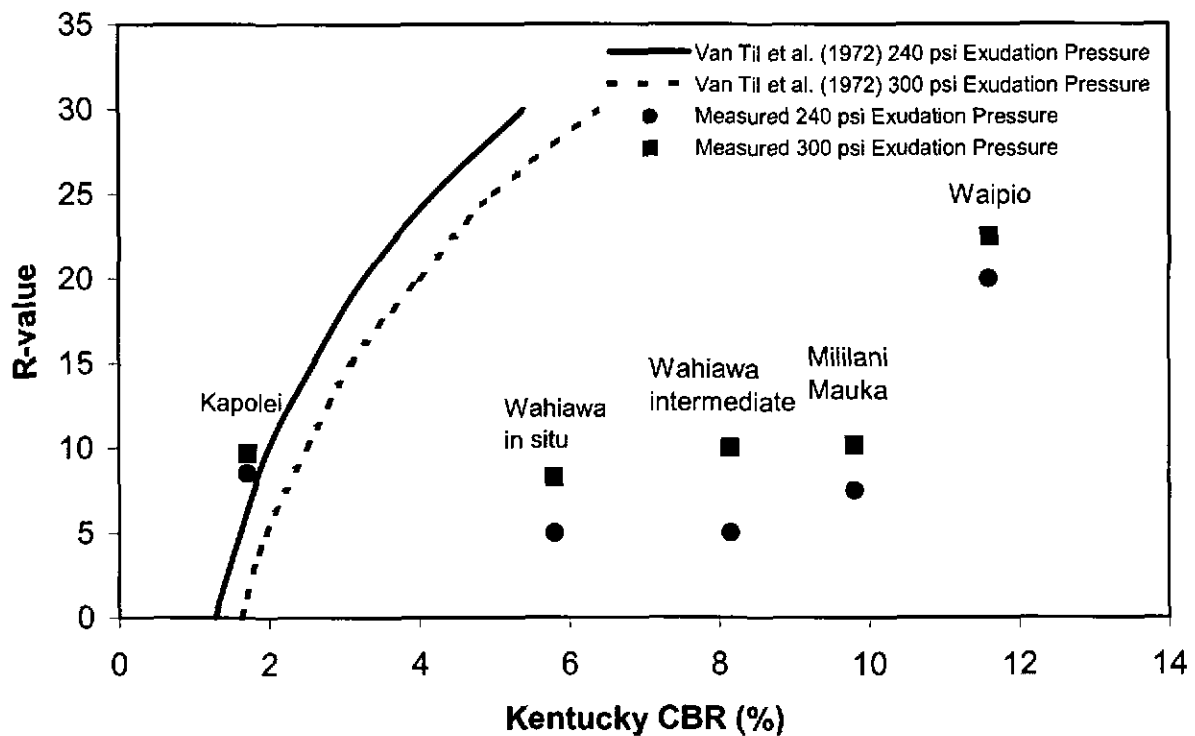


Figure 6.27 Comparison of measured R-value versus predicted using Van Til et al. (1972)

6.1.2 Method 2

In this method, a linear regression was performed on the slopes (column 4 of Table 6.1) and intercepts (column 5 of Table 6.1) obtained from Method 1. The slope and intercept were each related to the following dependent variables: energy ratio, relative compaction, moisture content relative to optimum, and exudation pressure. The energy ratio is defined as the compaction energy per unit volume used for the actual test normalized by the compaction energy per unit volume for the Standard Proctor test according to Procedure A of ASTM D698-00. The compaction energy per unit volume in ft-lbs/ft³ is defined as:

$$ER = \frac{\text{No. of blows per layer} \times \text{No. of layers} \times \text{Wt. of Hammer} \times \text{Hammer Drop Height}}{\text{Mold Vol.}} \quad (6.1)$$

The energy ratios are 4.53, 2.02, 0.808 and 0.996 for 5 layers at 56 blows, 5 layers at 25 blows, 5 layers at 10 blows and 3 layers at 56 blows, respectively. A wetness factor was established as follows: 0 for optimum, -1 for wet-of-optimum and +1 for dry-of-optimum.

The slopes (m) and intercepts (c) are related to the energy ratio (ER), relative compaction (RC), wetness factor (WF) and the exudation pressure (EP in psi) as follows:

$$m = 0.1693ER + 2.482WF + 0.4599RC + 0.00224EP - 45.34 \quad (6.2)$$

$$c = -0.9051ER - 7.834WF - 4.458RC + 0.0515EP + 423.1 \quad (6.3)$$

The coefficients of determination for equations 6.2 (Figure 6.28) and 6.3 (Figure 6.29) are 0.782 and 0.857, respectively. These two equations were then combined and used to predict the R-value for each of the CBR test as follows:

$$R = (0.1693ER + 2.482WF + 0.4599RC + 0.00224EP - 45.34)CBR - 0.9051ER - 7.834WF - 4.458RC + 0.0515EP + 423.1 \quad (6.4)$$

The resulting calculated R-values are plotted versus the measured values in Figure 6.30.

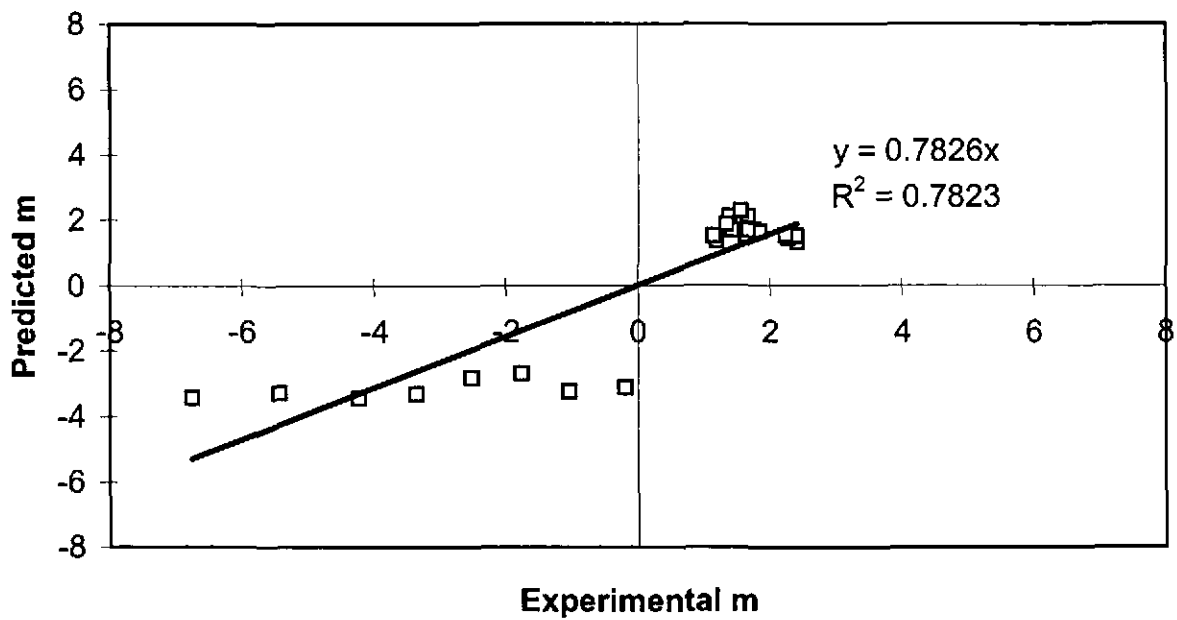


Figure 6.28 Predicted versus experimental slopes of the R-value versus CBR curves

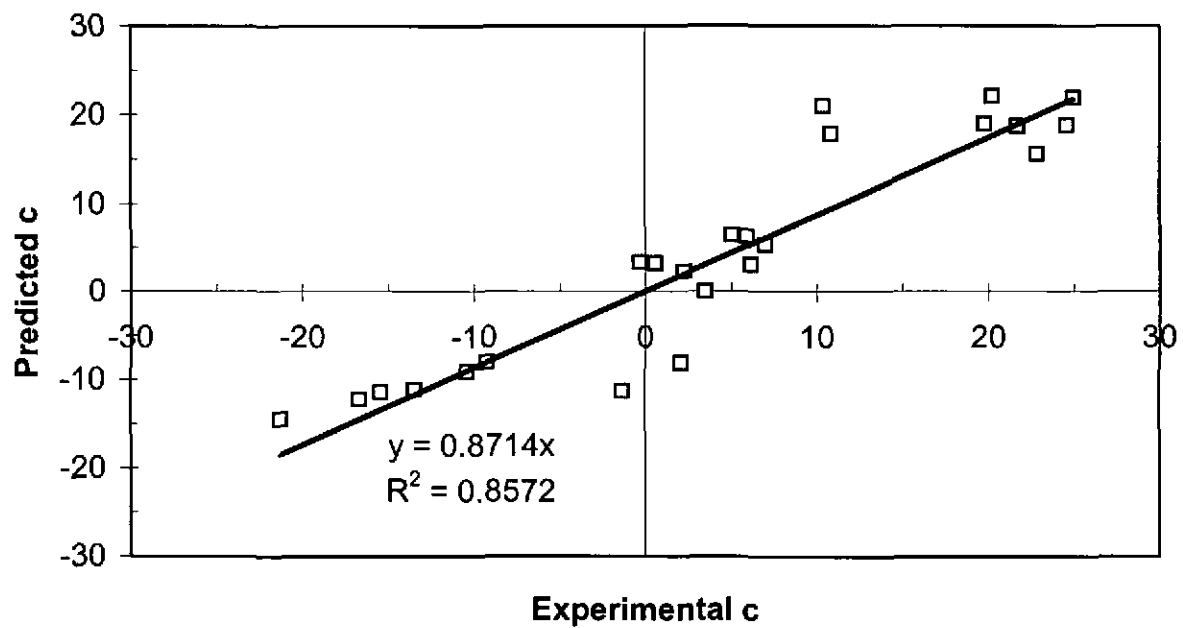


Figure 6.29 Predicted versus experimental intercepts of the R-value versus CBR curves

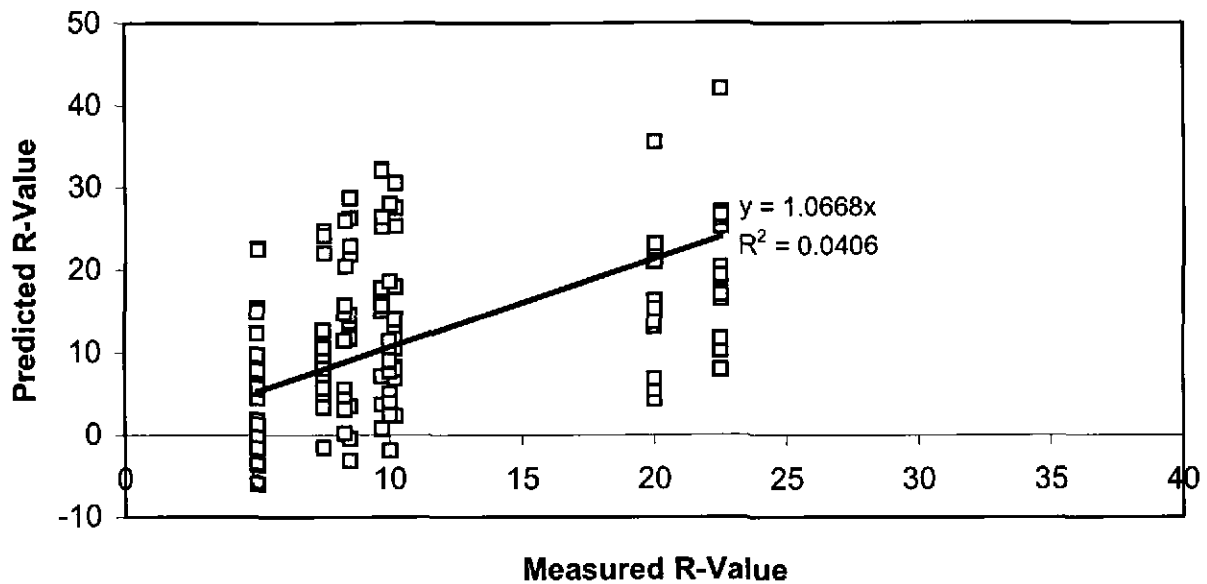


Figure 6.30 Comparison of predicted and measured R-values using Method 2

It can be seen that there is considerable variability in the predicted versus measured R-values. This is because all the CBR data points were used in this comparison irrespective of the relative compaction and the moisture content with respect to optimum while the correlation was derived using only CBRs at 100% and 95% relative compaction, the latter at both dry- and wet- of optimum. The suitability of this equation can be further evaluated by comparing how this equation plots relative to the data in Figures. 6.1 through 6.24. In general, it can be concluded that this equation is more suited to “dry-of-optimum” soils and less suitable for “optimum” and “wet-of-optimum” soils.

6.1.3 Method 3

A simple relationship between R-value and CBR was developed involving the exudation pressure and the activity of the fine-grained soil by trial and error. Use of

other parameters were explored but the resulting correlation coefficient was highest with the following relationship:

$$R = K_1 \text{CBR}^{K_2} A^{K_3} \text{EP}^{K_4} \quad (6.5)$$

where A = activity (expressed as numeric and not in %), EP = exudation pressure (in psi), and K_1 through K_4 are constants obtained by using a solver to find the values that gave a minimum objective function. The objective function was defined as the sum of the square of the differences between the measured and predicted R-values. A coefficient of determination of 0.6196 was obtained based on the following values of K_1 through K_4 when forcing the regression line through the origin: $K_1 = 0.00652$, $K_2 = 0.04708$, $K_3 = -1.675$ and $K_4 = 1.096$ (Figure 6.31). This correlation was derived using CBR values interpreted at 95% and 100% relative compactions.

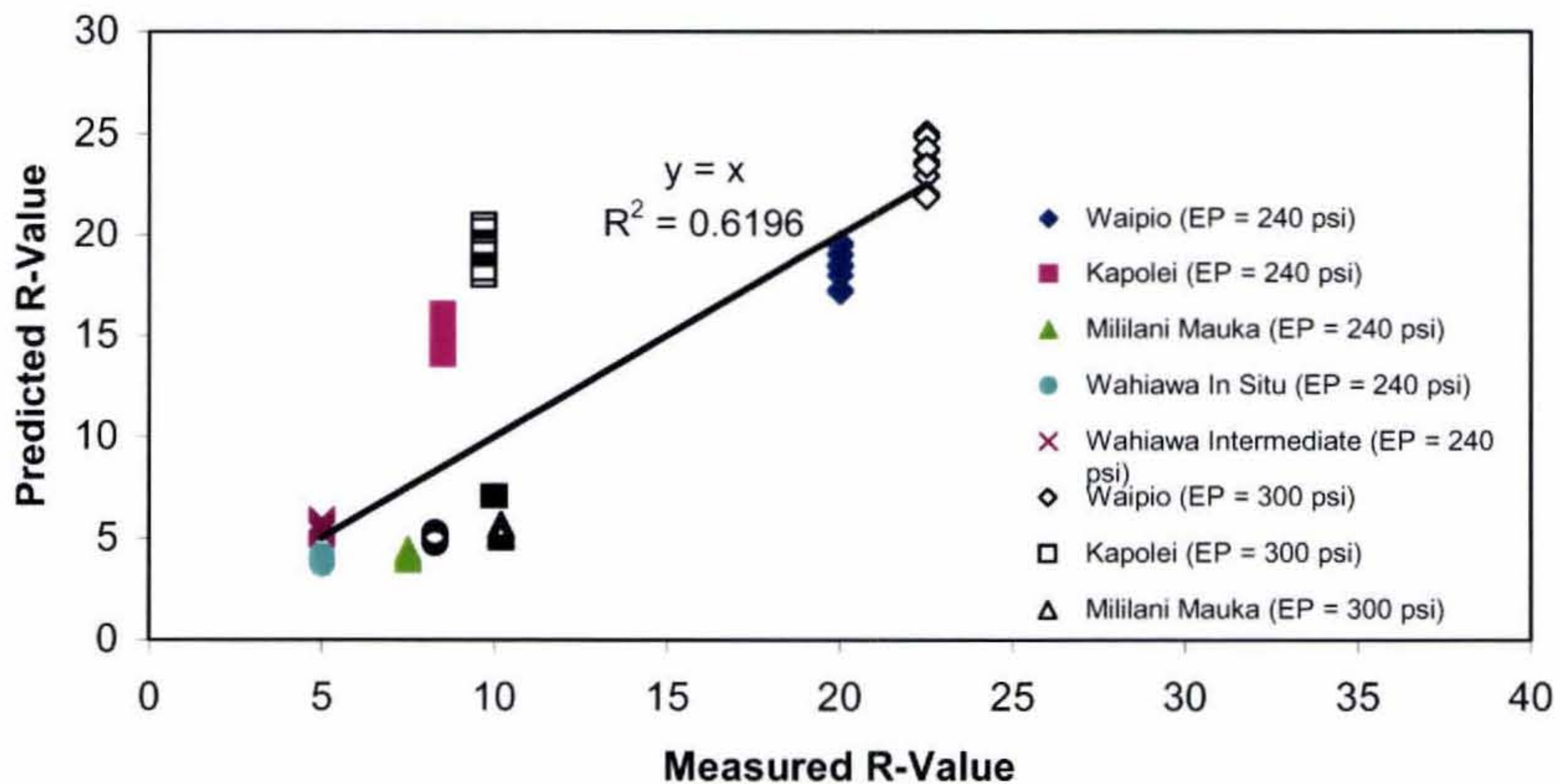


Figure 6.31 Comparison of predicted versus measured R-values using Method 3

6.1.4 Method 4

This procedure is based on the method of Li and Selig (1994) to estimate the resilient modulus at a given physical state (combination of dry unit weight and water content) of a soil. The eventual objective of this method is to relate the CBR at optimum corresponding to the Modified Proctor compaction effort (5 layers with 56 blows) to the R-value because of the high coefficient of determinations between CBR and R-values (0.822 and 0.766 for 240 and 300 psi exudation pressures, respectively) at this relative compaction. If the CBR at another energy ratio is available rather than the modified Proctor CBR at optimum, then several steps are needed to correlate CBR with R-value. These steps require two relationships: one between the optimum CBR at any compaction effort and the equivalent Modified Proctor CBR at constant dry unit weight and the second relating the CBR at any physical state on a compaction curve to the CBR at optimum along the same compaction curve. The equation for paths of constant energy ratio or compactive effort is as follows:

$$\frac{\text{CBR}}{\text{CBR}_{\text{opt}}} = \text{sech} \left[2.623ER^{0.2037} \text{PI}^{0.536} (w - w_{\text{opt}}) \right] \quad (6.6)$$

Note that sech refers to the hyperbolic secant of the term in the parentheses and $\text{sech } x = 2/(e^x + e^{-x})$. This regression equation yields a coefficient of determination of 0.721 when comparing the predicted and measured normalized CBRs. It is plotted in Figure 6.32 using a PI of 50%. The PI and energy ratios are included in the regression equation because they affect the width of the base of the CBR versus water content plots. As can be seen in Figures 4.6 through 4.11, the width of the base increases with increasing plasticity index (MH soils have broader bases than ML soils) and decreasing

compactive effort. Also shown on the plot is the data obtained from this testing program.

The equation for paths of constant dry unit weight is given by:

$$\frac{CBR_{mod}}{CBR_{opt}} = \text{sech}[0.2899(w - w_{opt}) - 0.4837] + 0.1065 \quad (6.7)$$

This equation relates the CBR at optimum corresponding to a given compactive effort with the CBR at Modified Proctor along lines of constant dry unit weight. Therefore, if the CBR corresponding to 100% relative compaction based on Standard Proctor is known, then the CBR corresponding to Modified Proctor at the same dry unit weight can be obtained using the above equation. This equation is plotted in Figure 6.33 along with the data generated in this study.

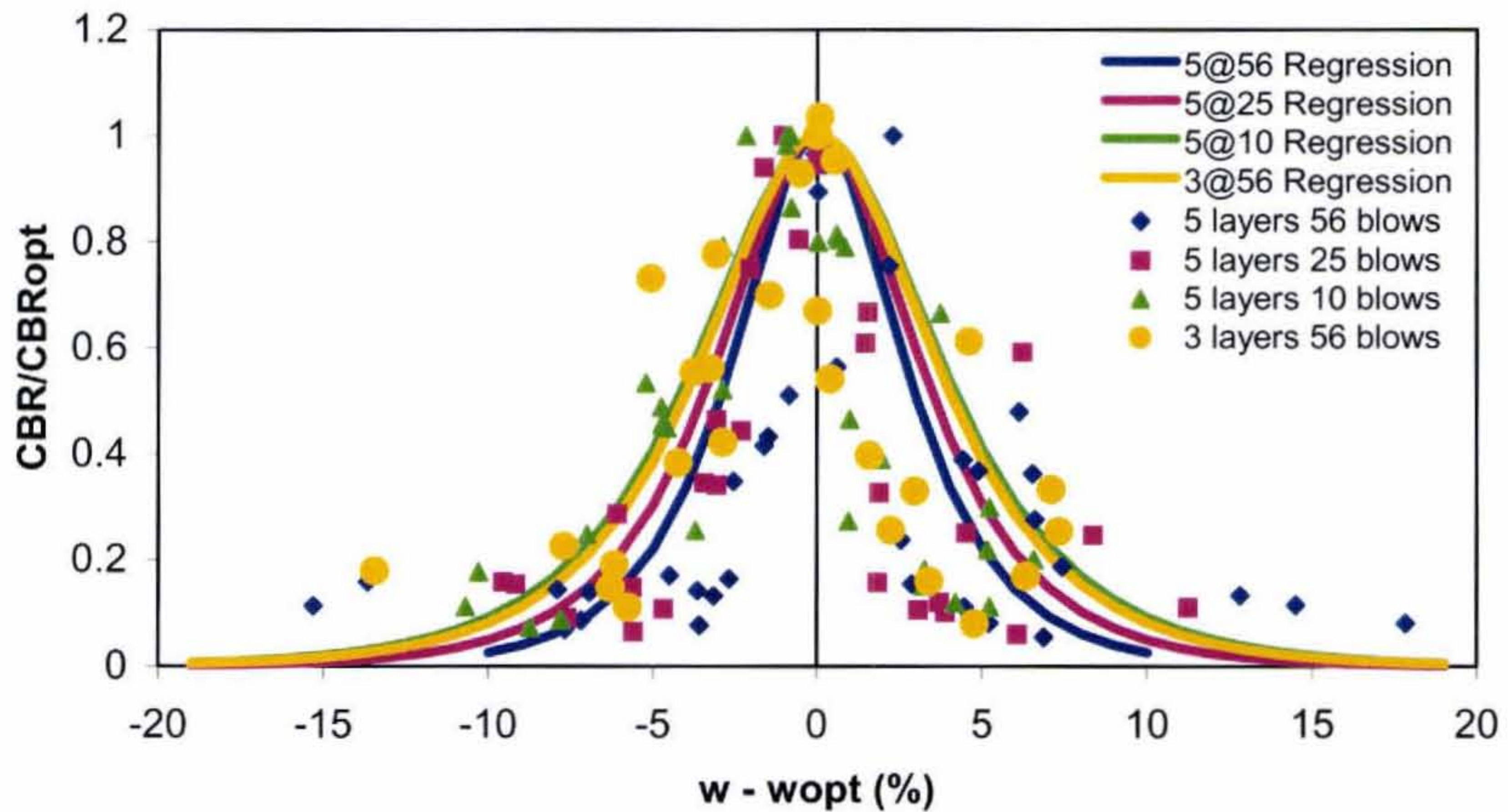


Figure 6.32 Normalized CBR versus water content for constant compactive effort

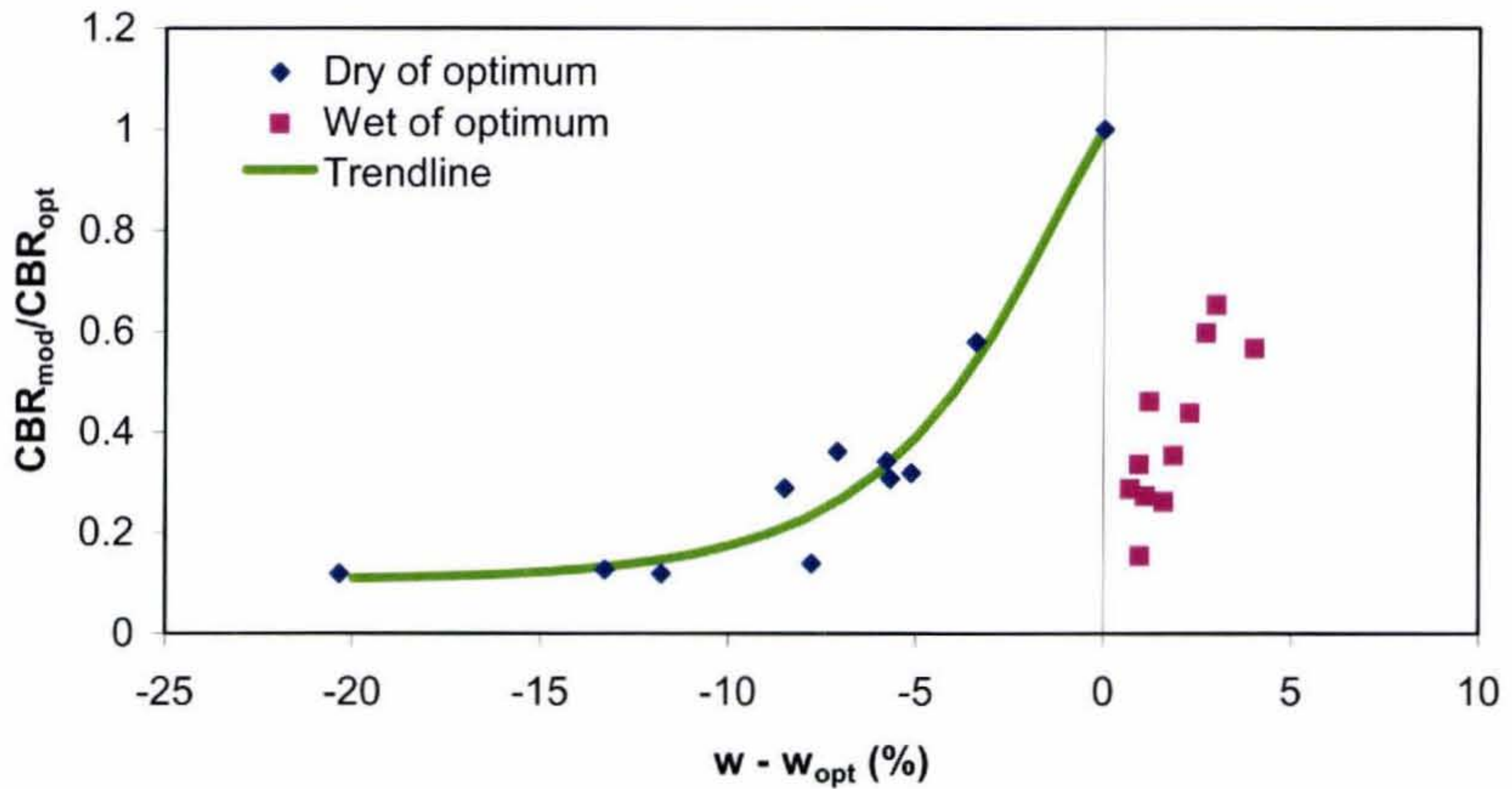


Figure 6.33 Normalized CBR versus water content for constant dry unit weight

Note that equation 6.7 is applicable only to dry-of-optimum samples. With the wet-of-optimum data points in Figure 6.33, the normalized CBR increases with increasing water content. At the optimum water content, the normalized CBR must revert back to unity; i.e., the plot must curve back to (0, 1.0), which means that there exists two possible values of the normalized CBR for a given value of $w - w_{opt}$. This further reinforces the fact that the R-value should not be correlated to wet-of-optimum CBRs.

If a CBR is available for a specimen prepared at a relative compaction other than Modified Proctor, then the following procedure can be used to estimate the CBR at optimum Modified Proctor:

1. For a given compactive effort, measure the CBR corresponding to a physical state.
2. Use equation 6.6 to estimate the CBR at optimum for the same compactive effort.

3. Use equation 6.7 to estimate the CBR corresponding to Modified Proctor at constant dry unit weight.
4. Use equation 6.6 again to estimate the Modified Proctor CBR at optimum.

The following figure and two scenarios are described to better illustrate this procedure.

1. If the CBR at Point Q in Figure 6.34 is required and the CBR is known at Point O, then Path OQ = Path OA + Path AQ. Estimate the CBR at Point A using equation 6.7. Using the value of CBR at Point A, estimate CBR at Point Q using equation 6.6.
2. If the CBR at Point Q is required and the CBR is known at Point C, then Path CQ = Path CO + Path OA + Path AQ. First, using the CBR at Point C, estimate the CBR at Point O using equation 6.6. The CBR at Point Q can now be estimated using Step 1.

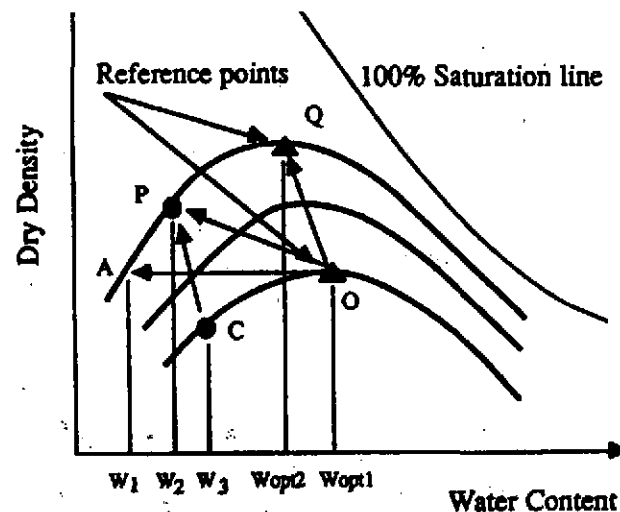


Figure 6.34 Path to obtain CBR based on Modified Proctor when the CBR at other compaction effort is known (Li and Selig, 1994)

This procedure was used to estimate the R-value by first estimating the Modified Proctor CBR at optimum for all the CBR tests performed at or dry of optimum. Then the following regression equations between R-value and CBR were used to estimate the R-value.

$$\text{For exudation pressure} = 240 \text{ psi, } R = 1.52\text{CBR} - 21.3 \quad (6.8)$$

$$\text{For exudation pressure} = 300 \text{ psi, } R = 1.37\text{CBR} - 15.4 \quad (6.9)$$

Even though this model appears rational, the R-values predicted using this method was very widely divergent (Figure 6.35). This is because the spread in the original data itself is quite variable and only a limited number of R-values are available for correlation.

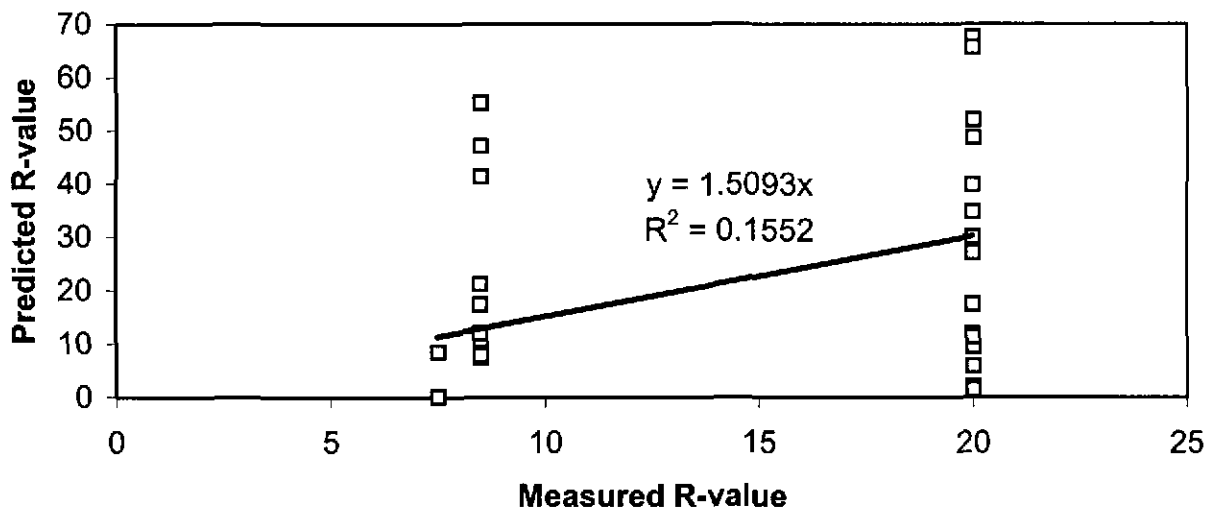
6.1.5 Method 5

In this procedure, the R-value is correlated to index properties (specifically the activity) and the exudation pressure. This method was developed in light of the Arizona DOT procedure, which did not provide reliable R-values for the tropical soils tested (Table 6.2).

Table 6.2 Comparison of measured R-value with those predicted using the Arizona DOT chart at 300 psi exudation pressure

Soil	PI (%)	% Passing #200 Sieve	R-value from ADOT Chart (Table 2.2)	R-value measured
Waipio	15.9	89	15	22.5
Kapolei	14.1	99	15	9.7
Mililani Mauka	51.0	99	3	10.2
Wahiawa in situ	51.4	98	3	8.3
Wahiawa intermediate	45.1	100	4	10
Wahiawa ovoidry	19.4	100	12	20.6

Exudation Pressure = 240 psi (1655 kPa)



Exudation Pressure = 300 psi (2068 kPa)

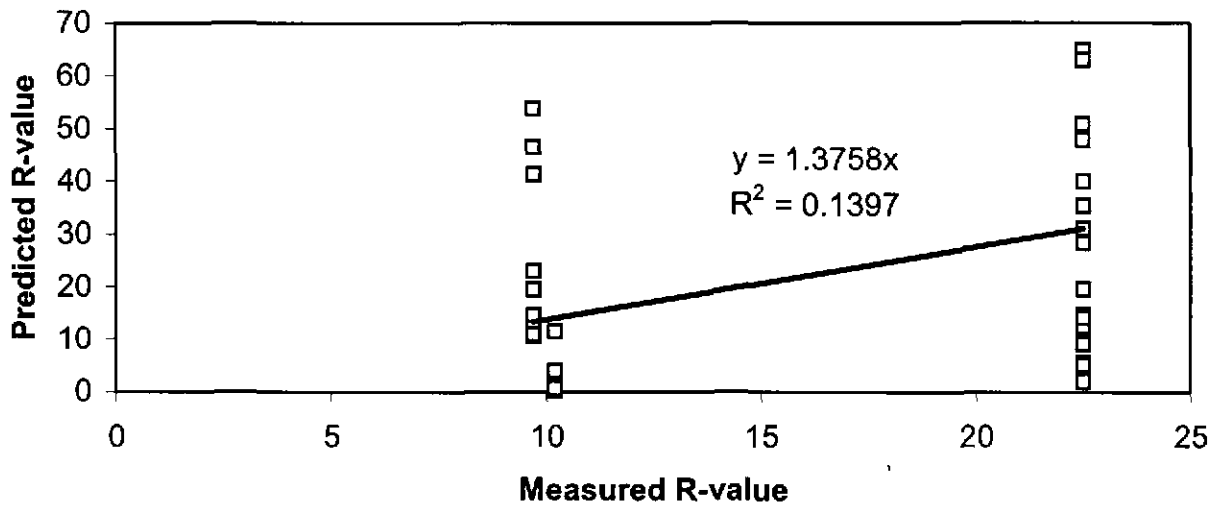


Figure 6.35 Comparison of predicted and measured R-values using Method 4

Use of other parameters were explored but the resulting correlation coefficient was highest with the following relationship that relates R-value with activity (A) and exudation pressure (EP):

$$R = C_1 A^{C_2} EP^{C_3} \quad (6.10)$$

where constants $C_1 = 0.005616$, $C_2 = -1.71$ and $C_3 = 1.131$. Equation 6.10 has a similar form as equation 6.5 except that the CBR term is eliminated. A comparison of the predicted and measured R-values is shown in Figure 6.36. The coefficient of determination obtained was 0.6114, approximately the same as that obtained with equation 6.5. From this exercise, it appears that the R-value is more dependent on the soil characteristics and the exudation pressure and less dependent on the value of CBR.

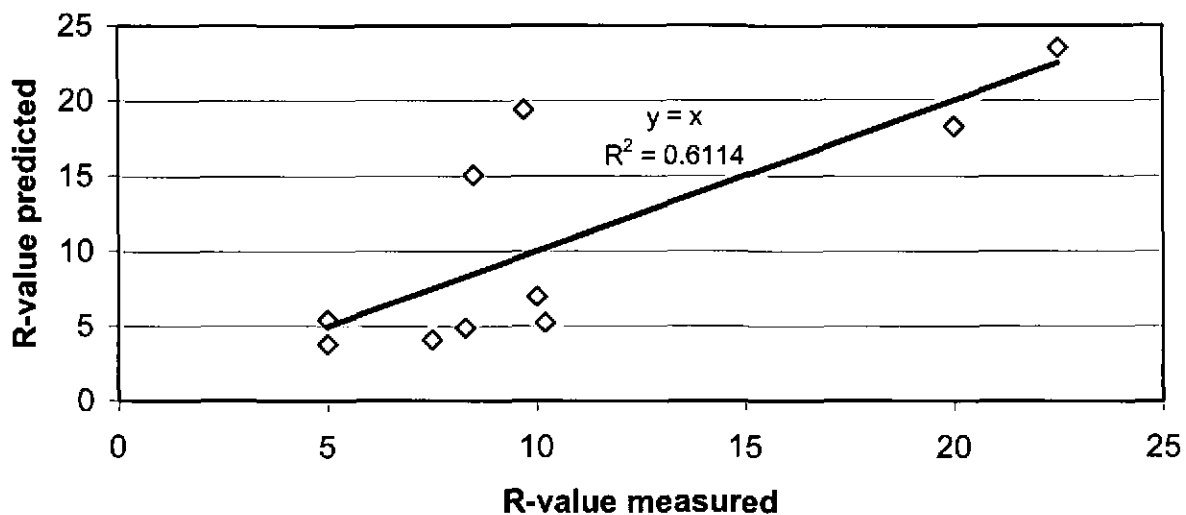


Figure 6.36 Comparison of predicted versus measured R-values using equation 6.10

6.2 Choice of Correlation Method

The choice of method to use to correlate CBR with R-value depends on the physical state at which the CBR is measured. Methods 1, 2 and 3 were developed based on interpreted CBRs at 95% and 100% relative compaction. Method 4 was developed based on all the actual test data rather than values of interpreted CBR at 95% and 100% relative compaction.

Method 1 is the recommended procedure for estimating R-value if the CBR is available at any of the following relative compaction, physical state and compactive effort: 95% relative compaction dry-of-optimum, 100% relative compaction or at optimum, and 95% relative compaction wet-of-optimum, and compactive efforts of 5 layers at 56 blows (Modified Proctor), 5 layers at 25 blows, 5 layers at 10 blows and 3 layers at 56 blows (Standard Proctor). If the CBR is not available at any of the above relative compaction, physical states and compactive efforts, then the other methods should be used.

Methods 2 and 4 are appropriate only for dry-of-optimum soils. Method 3 is more versatile but the exponent for CBR is 0.04708 implying that the R-value is not very sensitive to CBR. However, among methods 2, 3 and 4, method 3 produces results that have least variability.

Method 5 relates R-value to the exudation pressure and activity only and is useful if CBR values are not available.

Limitations and general comments on each method are summarized in Table 6.3.

Table 6.3 Limitations of the methods to estimate R-value

Method	Limitations	Comments
1	<ol style="list-style-type: none"> Valid only for CBR measured on samples compacted using 4 specific energy ratios and 2 relative compactions (95% and 100%). Not valid for CBR measured on wet-of-optimum samples. 	<ol style="list-style-type: none"> This is the recommended procedure for use in design of flexible pavements. Simple to use correlations provided in charts as well as in the form of linear equations. Correlations established based on CBR values interpreted at 95% and 100% relative compaction.
2	<ol style="list-style-type: none"> Valid for CBR measured on dry-of-optimum samples only. Very low coefficient of determination. 	<ol style="list-style-type: none"> Correlation in the form of one equation. Correlations established based on CBR values interpreted at 95% and 100% relative compaction. Can be used on compactive efforts and relative compactions other than the ones used to derive this correlation but extrapolation required.
3	<ol style="list-style-type: none"> Valid only for CBR measured on samples compacted using 4 specific energy ratios and 2 relative compactions (95% and 100%). 	<ol style="list-style-type: none"> Correlation in the form of one equation. Correlations established based on CBR values interpreted at 95% and 100% relative compaction. Exponent for CBR is very low indicating R-value is more correlated to activity and exudation pressure and less correlated to CBR. Valid for CBR measured on dry-of-, at- and wet-of-optimum samples.
4	<ol style="list-style-type: none"> Valid for CBR measured on dry-of-optimum samples only. Very low coefficient of determination. 	<ol style="list-style-type: none"> More general correlation in the form of several equations that can be used to estimate the R-value from a CBR obtained on samples prepared at any relative compaction and energy ratio. Correlations established based on all CBR test data.
5		<ol style="list-style-type: none"> Simple correlation in the form of one equation. Correlation independent of CBR.

CHAPTER 7 SUMMARY AND CONCLUSIONS

7.1 Summary

CBR, R-value and laboratory index tests were conducted on samples collected from four different locations on the island of Oahu: Waipio, Kapolei, Mililani Mauka and Wahiawa. The Waipio and Kapolei soils were classified as ML (AASHTO A7-6) while the Mililani Mauka and Wahiawa soils were classified as MH (AASHTO A7-5). The Wahiawa soil was significantly wetter than the other three with water contents in excess of 50%. The liquidity index for the Wahiawa soil was 0.11 while the other three soils had negative liquidity indices, an indication that they are desiccated.

Due to the higher water contents, the Wahiawa soil was tested at three different stages of drying: first at its natural or in situ state, second after oven drying the soil; and third after drying the soil to approximately half its natural water content (termed Wahiawa intermediate). Therefore, the Wahiawa soil can be regarded as three different soils corresponding to three different stages of drying.

CBR tests were performed at several compactive efforts. At each compactive effort, the CBR was measured over a range of molding water contents, thereby enabling a family of curves to be plotted. Swell was also measured after soaking the soil for 4 days. Maximum volumetric expansion of about 2% and 7% were observed in the ML and MH soils, respectively.

The CBR family of tests involves preparing samples over a range of moisture contents and dry unit weights. Unlike the CBR, the R-value test data do not directly permit selection of field compaction conditions. The R-value test is measured over a

range of exudation pressures by varying the water content, and the design R-value is selected based on a value of exudation pressure that best represents the worst condition likely to be reached in place in the subgrade several years after construction (Howe, 1961). As a result of this difference between the CBR and R-value, it is important to know not only the correlation between the two parameters but also under what conditions are the correlations applicable.

Existing correlations between CBR and R-value have limited applicability. With the exception of the Van Til (1972) correlation, the physical state at which the CBR is measured and correlated to R-value is not defined. The performance of “indirect” correlations such as Equations 2.5 and 2.7 was found to be reasonable for dry-of-optimum soils, overpredict the R-value for soils at optimum, and underpredict the R-value for wet-of-optimum soils. The Van Til et al. procedure results in an overprediction of the R-value in 4 of the 5 soils tested. The Arizona DOT correlation between R-value and index properties also showed poor agreement with measured data.

New correlations to estimate R-values were developed as part of this study. In deriving these correlations, the Wahiawa oven-dry data was excluded because these soils were dried to temperature extremes that regular soils do not experience, and therefore, are judged to be inappropriate for use. Nevertheless, the data provided useful insight into the effects of drying on the measured properties. For the Wahiawa soil, the CBR and R-value increased with increasing degree of drying indicating that the soil underwent irreversible changes upon drying.

7.2 Conclusions and Recommendations

A total of 5 correlations are included in this report. The first is a simple linear regression between R-value and CBR. Separate correlations were developed for various relative compactions, compactive effort and exudation pressures. These correlations (Figs. 6.1 through 6.26) are direct and are recommended for use by HDOT in the design of flexible pavements. However, these correlations are more suitable for CBR samples prepared at- or dry-of-optimum. Wet-of-optimum samples yielded negative slopes, implying that R-value decreases with increasing CBR. Seemingly counterintuitive, it is therefore less desirable to correlate wet-of-optimum CBR with R-value.

Other correlations were developed but they are less direct and should **only** be used if the CBR is available at a physical state different than the ones used to develop Figs. 6.1 through 6.26. A second correlation resulted in an equation (6.4) that relates the R-value at a desired exudation pressure to the CBR measured at a given relative compaction and compactive effort. This equation was developed by performing linear regression on the slope and intercept from method 1, where they were made functions of the relative compaction, compactive effort, exudation pressure and a wetness factor. This correlation appears to be valid only for dry-of-optimum samples.

A third correlation relates R-value to CBR, activity and exudation pressure with appropriate exponents. The exponent for CBR is 0.04708 implying that the R-value is not very sensitive to CBR. The coefficient of determination was 0.62 when comparing the measured and predicted R-values.

The first three correlations were established based on CBR values interpreted at 95% and 100% relative compaction. A fourth method was developed based on all the

CBR data rather than interpreted CBR values. This procedure appears rational, has significant scatter in the results, and again is not applicable for wet-of-optimum CBRs.

The fifth method relates R-value to the activity of the soil and exudation pressure. It is useful for estimating R-value when CBR data is not available. The coefficient of determination was 0.61.

7.3 Suggestions for Future Work

The scope of work included testing of a limited number of soil types (ML and MH), based on which the correlations were developed. Additional tests (e.g., on CL soils) should be performed so that the correlations can be updated if necessary to include a wider range of soil types.

Van Til's (Figure 2.4) correlation results in unconservative R-values (generally too high) for a given Kentucky CBR. Equations 2.5 and 2.7 are reasonable for dry-of optimum soils, overpredict the R-value for soils at optimum, and underpredict the R-value for wet-of-optimum soils. Additional research may be useful in assessing these consequences on past flexible pavement designs.

For future flexible pavement designs, it is recommended that the HDOT specify that CBRs be measured at a given physical state (say 100% relative compaction using 5 layers at 56 blows). Companion R-values should be determined at say 300 psi exudation pressure by HDOT on the same soil, and the correlation in Figure 6.2 assessed and updated on a regular basis (say once every five years).

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APPENDIX

R-Value Database

Table A1 Interpreted R-values and soil properties

Soil	Specific Gravity	Natural Water Content (%)			% Fines	% Clay	Plasticity Index (%)	Liquid Limit (%)	USCS Symbol	Interpreted R-Value	
		Low	High	Mean						Exudation Pressure 240 psi	Exudation Pressure 300 psi
Waipio	2.90	26	29	28	89	48	16	46	ML	20	22.5
Kapolei	3.00	19	21	20	99	38	14	41	ML	8.5	9.7
Mililani	2.98	28	33	31	99	64	51	95	MH	7.5	10.2
Mauka											
Wahiawa in situ	3.08	51	57	53	99	62	51	99	MH	5	8.3
Wahiawa intermediate	3.08	-	-	26	99	67	45	87	MH	5	10
Wahiawa ovendry	3.11	-	-	0	99	53	19	64	MH	13.2	20.6

Table A2 Measured R-values

Soil	Exudation Pressure (psi)	R-Value	w Prior to Exudation (%)	γ_d Prior to Exudation (pcf)
Waipio	231	16.2	26.9	97.1
	271	23.9	26.0	98.9
	294	21.1	26.7	100.8
	319	28.3	25.4	100.5
	438	28.5	26.0	100.8
	581	29.4	25.6	100.8
	605	31.9	25.2	100.0
	653	32.5	25.0	101.2
Kapolei	191	7.6	28.1	96.8
	245	8.5	26.5	99.5
	374	13.0	26.4	100.5
	398	13.2	26.0	104.1
	462	15.7	25.1	103.6
	509	23.0	23.8	103.9
Mililani Mauka	255	6.7	44.6	77.7
	255	10.4	44.8	76.9
	334	12.7	46.7	77.7
	398	17.0	41.1	79.6
	398	42.2	36.8	86.3
	454	45.6	37.0	84.9
	533	19.0	45.9	78.4
	541	17.1	40.1	82.1
	621	24.9	39.1	82.6
Wahiawa In Situ	143	3.3	53.5	70.7
	286	3.7	49.4	73.3
	294	6.9	48.4	75.6
	310	6.2	45.3	78.9
	318	9.3	45.7	78.4
	398	20.2	39.5	84.4

406	32.8	37.7	87.3
477	37.6	36.4	88.7

Table A2 Measured R-values (cont'd)

Soil	Exudation Pressure (psi)	R-Value	w Prior to Exudation (%)	γ_d Prior to Exudation (pcf)
Wahiawa Intermediate	199	3.4	51.2	73.2
	231	2.7	50.7	74.6
	255	9.7	45.4	79.5
	302	7.8	46.6	75.9
	318	5.7	47.9	75.9
	318	7.7	44.4	79.3
	350	9.0	44.7	77.3
	382	25.1	37.9	87.6
	382	31.0	36.2	89.3
	398	16.6	41.5	82.0
	430	15.6	39.6	84.2
	716	56.8	33.3	88.5
Wahiawa Ovendry	191	10.0	46.6	74.9
	223	14.3	41.9	82.7
	263	14.2	44.0	79.6
	263	16.5	42.6	81.1
	286	16.7	41.2	83.3
	366	18.8	39.6	83.9
	493	44.3	38.2	85.1
	541	59.6	36.4	84.6