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USE OF STIFFNESS FOR EVALUATING COMPACTNESS OF COHESIVE GEOMATERIALS

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Abstract

There has been a recent push towards adoption of the in-place soil stiffness as a means of assessing compactness of pavement geomaterials. The Humboldt GeoGauge™ is a relatively new and promising instrument that is portable, that provides instantaneous results and that does not require handling of radioactive materials. Unlike the nuclear gage, which yields the soil unit weight and water content, the GeoGauge™ yields soil stiffness corresponding to very low strains. Based on a series of low-strain soil stiffness measurements made under controlled laboratory conditions on compacted silts from Oahu, the variation of modulus with water content, dry unit weight, degree of saturation, volume change upon wetting, shear strength and soil plasticity is discussed. These results help advance the understanding of the role of stiffness in assessing compactness of cohesive soils. For compacted partly saturated soils, the dry unit weight can be related to stiffness and water content. This relationship is derived herein. Using this relationship, measured values of stiffness and water content can then be used to predict the dry unit weight in the field.

This work involved testing of tropical soils, which can undergo irreversible changes upon drying, resulting in permanent alterations in soil properties. Upon drying, the tropical cohesive soils tested became less plastic, coarser (downward shift in grain size and higher sand equivalent), and exhibits a higher maximum dry unit weight and lower optimum water content.

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Chapter 1 Introduction

The in situ dry unit weight, in relation to a laboratory-determined value of maximum dry unit weight, or relative compaction is traditionally used to evaluate the degree of compaction. There are several methods available for measuring the in situ dry unit weight that are widely accepted by engineers, designers and contractors. However, using the in situ dry unit weight for compaction control has been argued to be less logical than perhaps using soil stiffness, which has led to the sponsorship of a Federal Highway Administration (FHWA) GeoGauge™ pooled fund study entitled "Non-nuclear Testing of Soils and Granular Bases Using the GeoGauge™." In light of this, there has been a recent push towards adoption of the in-place soil modulus as a means of assessing compactness of geomaterials (Fiedler et al., 2000). A device for measuring the low strain soil stiffness was used on several cohesive soils on O'ahu to evaluate compactness under both field and controlled laboratory conditions. The variation of stiffness with water content, dry unit weight, degree of saturation, volume change upon wetting, shear strength and plasticity is discussed herein (Section 4.1). These test results help advance the understanding of the role of stiffness in assessing compactness of cohesive geomaterials.

There are also two secondary objectives as follows:

1) derive a relationship between low strain stiffness, dry unit weight and water content (Section 4.3) and
2) study the effects of drying on properties of some tropical soils (Section 5).

Chapter 2 Background and Literature Review

2.1 In Situ Methods for Compaction Control

Compaction control typically involves measurement of the in-place moist unit weight and moisture content. The in situ dry unit weight is then estimated, and compared to the maximum dry unit weight at the optimum moisture content based on a specified compaction effort. The unit weight and moisture content are commonly estimated using the rubber balloon calibrated vessel, sand cone device, nuclear density gauge and time domain reflectometry (TDR – Drnevich, 2000). The major disadvantage of the first two methods is that the results are not instantaneous since oven drying of the soil sample is needed to measure the moisture content. TDR is a relatively new procedure that requires extensive calibration for local soils. The nuclear density gauge provides relatively accurate and instantaneous results. However, its major disadvantage is the hassle of handling radioactive materials. It may currently be the method of choice for many agencies and engineers. Some may avoid using it either because of a lack of trust in the instrument or a disinterest in having to deal with the administrative aspects of storing, personnel training, transporting, exposure monitoring and general upkeep. It would therefore be ideal if compaction control could be performed using a device that does not involve radioactive materials and that yields instantaneous results.

2.2 Use of Low Strain Modulus for Compaction Control

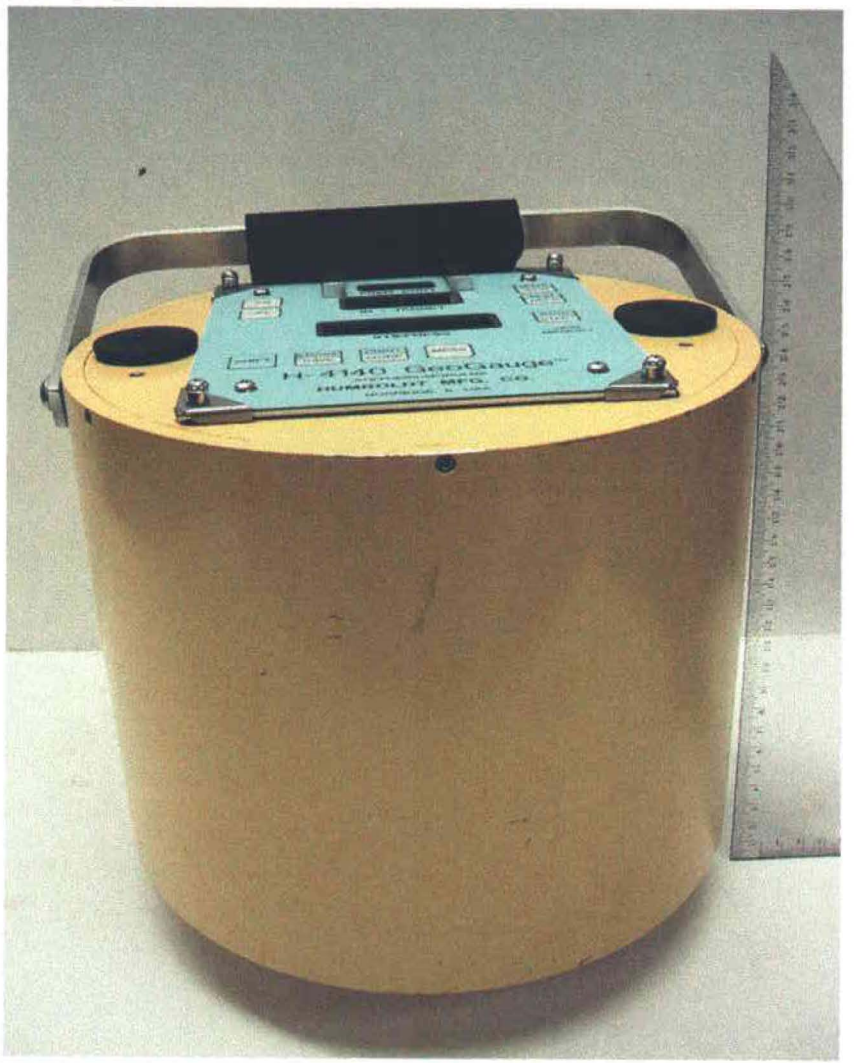
The low strain shear modulus (G_{max}) is a basic soil parameter used primarily in soil dynamic response analyses. Recently, the shear wave velocity ($V_s = \sqrt{G_{max}/\rho}$ where ρ is the soil mass density) has been proposed as a means of estimating the liquefaction resistance of soils (Youd et al., 2001). This is in part due to the fact that the shear wave velocity and the cyclic resistance ratio are both influenced by void ratio among other things. Since the dry unit weight is directly related to void ratio, it seems reasonable to postulate that low strain shear modulus can also be used for compaction control.

In the lab, G_{max} is measured using the resonant column test (Hardin and Drnevich, 1972; Kim and Novak, 1981) or using bender elements (Dyvik and Madshus, 1985). In situ methods of measuring low strain soil modulus include: (1) downhole and crosshole methods (Hoar and Stokoe, 1978); (2) seismic cone (Robertson et al., 1985); (3) suspension logging method (Kitsunezaki, 1980); (4) spectral analysis of surface wave (Nazarian and Stokoe, 1984; Stokoe et al., 1994; Menzies, 2001); (5) seismic refraction (Louie, 2001); (6) p-wave ultrasonic testing (Yesiller et al., 2000); (7) portable falling weight deflectometers (Carl Bro Pavement Consultants, 2002); and (8) a device called GeoGauge™ that measures stiffness (force/displacement) at very small displacements (Fiedler et al., 2000). Methods (4) to (8) are all non-destructive methods that require no drilling, can be used on the ground surface, and therefore have the potential for

use in compaction control. With these methods, the equipment is portable, does not involve use of radioactive materials and provides G_{max} relatively quickly. In this study, the GeoGauge™ was adopted for measuring soil stiffness. An attempt will be made to relate the low strain GeoGauge™ stiffness to dry unit weight and water content. If successful, it will offer a safe and expedient alternative for compaction control.

2.3 GeoGauge™

Produced by Humboldt Manufacturing Company, the GeoGauge™ (Fig. 1) is purported to measure the stiffness of the top 100 to 150 mm of the surface soil. The gauge is a portable cylinder, 28 cm in diameter, 25.4 cm high and weighs approximately 10 kg. A 114-mm-O.D. and 88-mm-I.D. ring footing extends from the bottom of the instrument. Powered by six D-Cell batteries, an internal harmonic oscillator imparts very small vertical displacements ($< 1.27 \times 10^{-6}$ m) to the soil via the ring foot. According to the manufacturer (Humboldt Mfg. Co., 2000), a stress of 27.58 kPa is imparted on the soil, which is appropriate for pavement and foundation loads. Internal geophones are used to measure the force and velocity time histories at 25 distinct frequencies varying between 100 and 196 Hz, from which the values of force and the corresponding displacement are obtained. Stiffness is calculated as the average force per unit displacement over the various test frequencies.



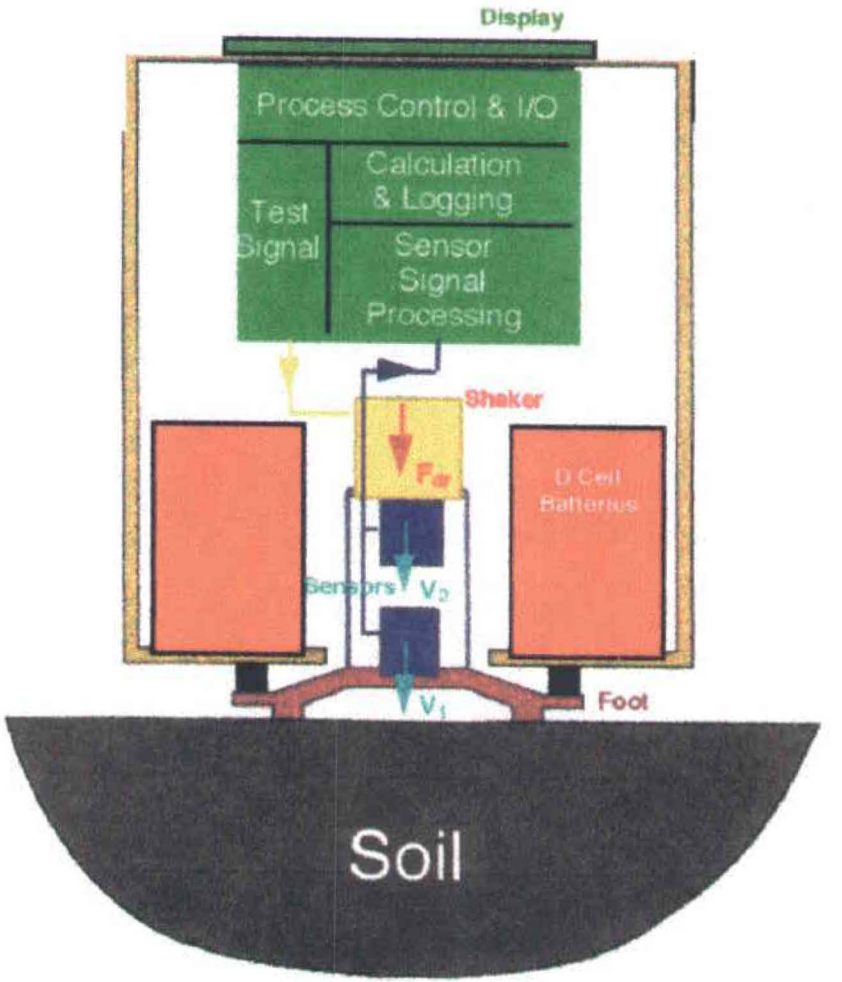


Figure 1. GeoGauge™ Device

Chapter 3 Laboratory Soil Testing

3.1 Sampling Locations

Soils from the following four locations on the island of O'ahu, Hawai'i, were sampled and tested (Fig. 2):

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

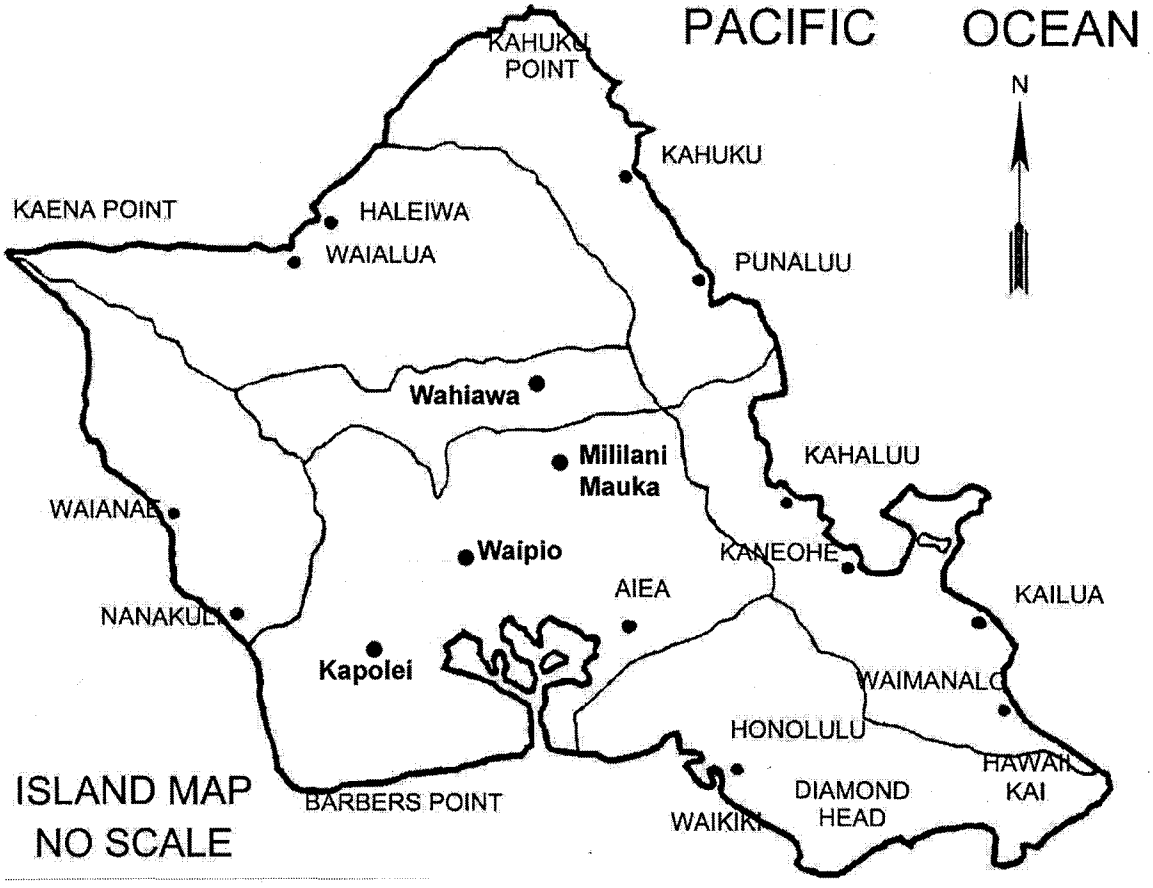


Figure 2. Soil Sampling and Testing Locations

A trench was dug at each location to expose the undesiccated soil for in situ testing and sampling. In situ tests performed include GeoGauge™ stiffness measurements, nuclear gauge and sand cone testing (See Section 4.2). To preserve the moisture content for laboratory testing, soil samples were:

1. placed in heat-sealed plastic bags;
2. each bag of soil was then placed in a 5-gallon plastic bucket, capped off with a lid containing an O-ring seal;
3. each bucket was stored in a 100%-humidity, concrete-curing room located in the structures lab in Holmes Hall at the Department of Civil and Environmental Engineering, University of Hawai'i.

These steps are necessary to prevent drying of the soil samples which can lead to irreversible changes in properties of tropical soils (See Section 5).

3.2 Index Tests and Results

Laboratory tests were performed to determine the following:

1. Atterberg limits
2. Grain size distribution
3. Specific gravity
4. Sand equivalent
5. Compaction curves

3.2.1 Atterberg Limits

Liquid and plastic limits were determined for each soil in accordance with ASTM Standard D4318-98. Test results are summarized in Table 1.

Table 1. Summary of Atterberg Limits for the In Situ Soil

Location	Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Waipio	1	45	25	20
	2	43	27	17
	3	48	37	10
	4	46	29	17
	Average	46	30	16
Kapolei	4	42	26	16
	4B	40	28	12
	23	42	26	15
	26	41	28	13
	27	41	28	13
	Average	41	27	14
Mililani Mauka	C	95	42	53
	1	102	47	55
	2	88	44	44
	7	95	39	57
	10	100	47	53
	Average	96	44	51
Wahiawa	25	94	44	50
	35A	97	48	49
	35B	97	49	47
	55	109	49	60
	56	97	45	52
	Average	99	47	52

The test results are also summarized in Figure 3. Based on the Unified Soil Classification System (USCS), soils from Waipio and Kapolei are classified predominantly as ML while soils from Mililani Mauka and Wahiawa are classified predominantly as MH.

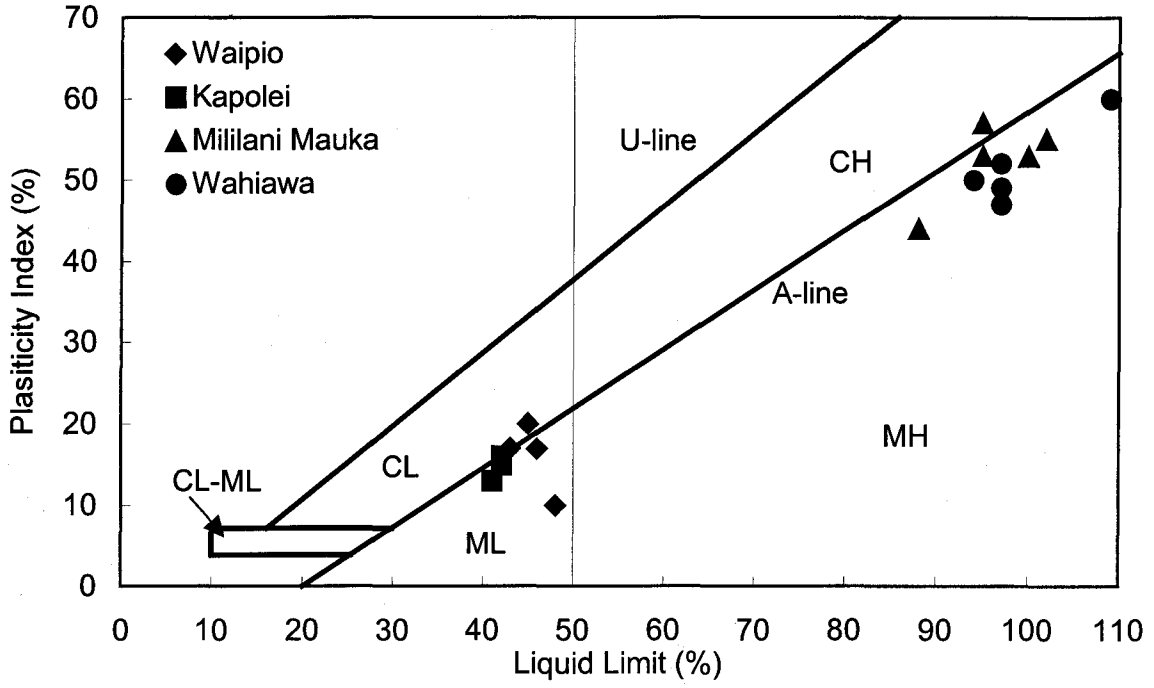


Figure 3. Atterberg Limits and Plasticity Chart

3.2.2 Grain Size Distribution

Grain size distributions were obtained by performing hydrometer testing and sieve analysis in accordance with ASTM Standard D422-63. Three methods were used for determining the grain size distribution for the Kapolei soil:

Method 1.

1. Soil from the same bucket were divided into two 100g portions.
2. Several moisture contents of the soil were then measured. Using the moist weight from (1) and the moisture content from (2), the total dry weight can then be estimated.

3. One portion was wet sieved through a stack of sieves. The material retained on the sieves was oven-dried to determine the dry weights.
4. The portion passing the No. 200 sieve was not saved 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 (3) from the total dry weight from (2).
5. The second portion of the soil from (1) was wet sieved through the No. 200 sieve and the fines and water were collected.
6. The collected soil/water mix from (5) was then dried to a moisture content that is near the in situ value.
7. After determining the moisture content, a portion of the moist fines equivalent to a dry weight of 50g was subjected to hydrometer testing.
8. The results from (3) and (7) were then combined to yield the complete grain size distribution.

Method 2.

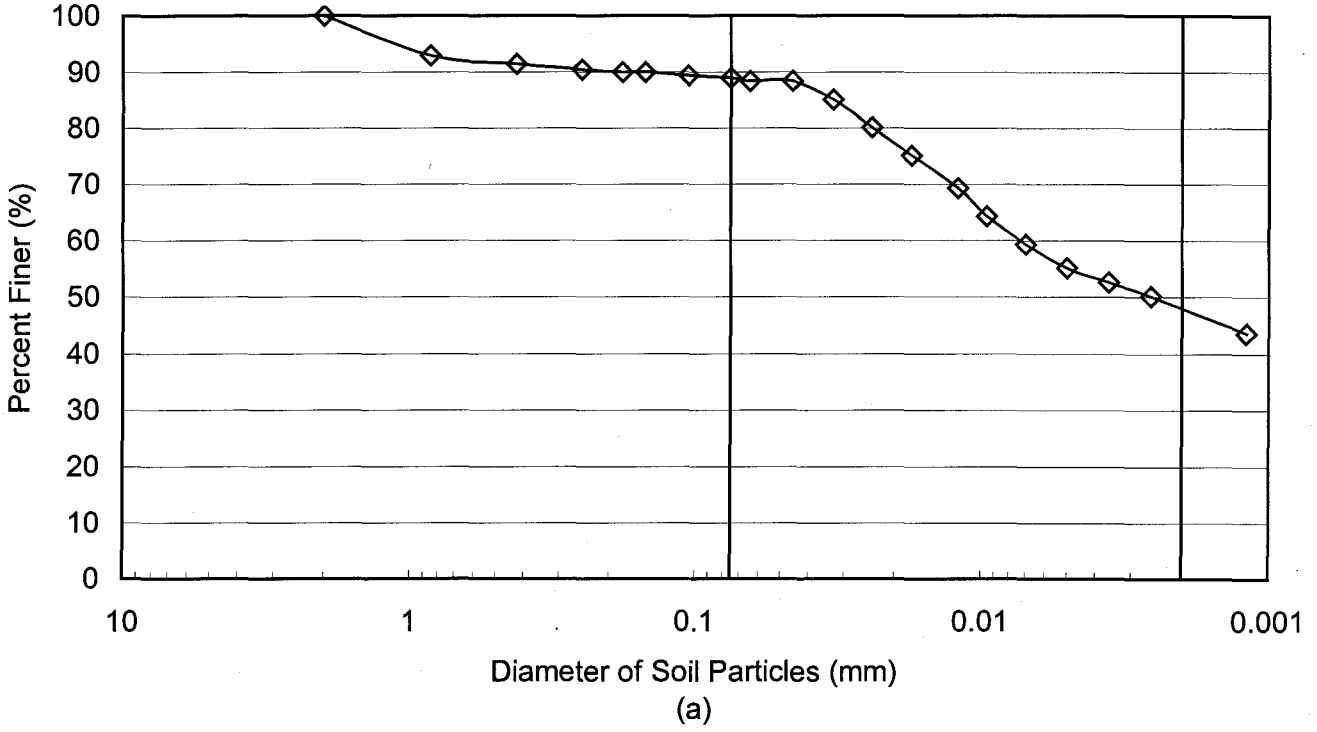
This method is the same as method 1 except for steps 1 and 4. In step 1, only one portion of sample was used for wet sieving. In step 4, all the fines passing the No. 200 sieve were collected for hydrometer testing.

Method 3.

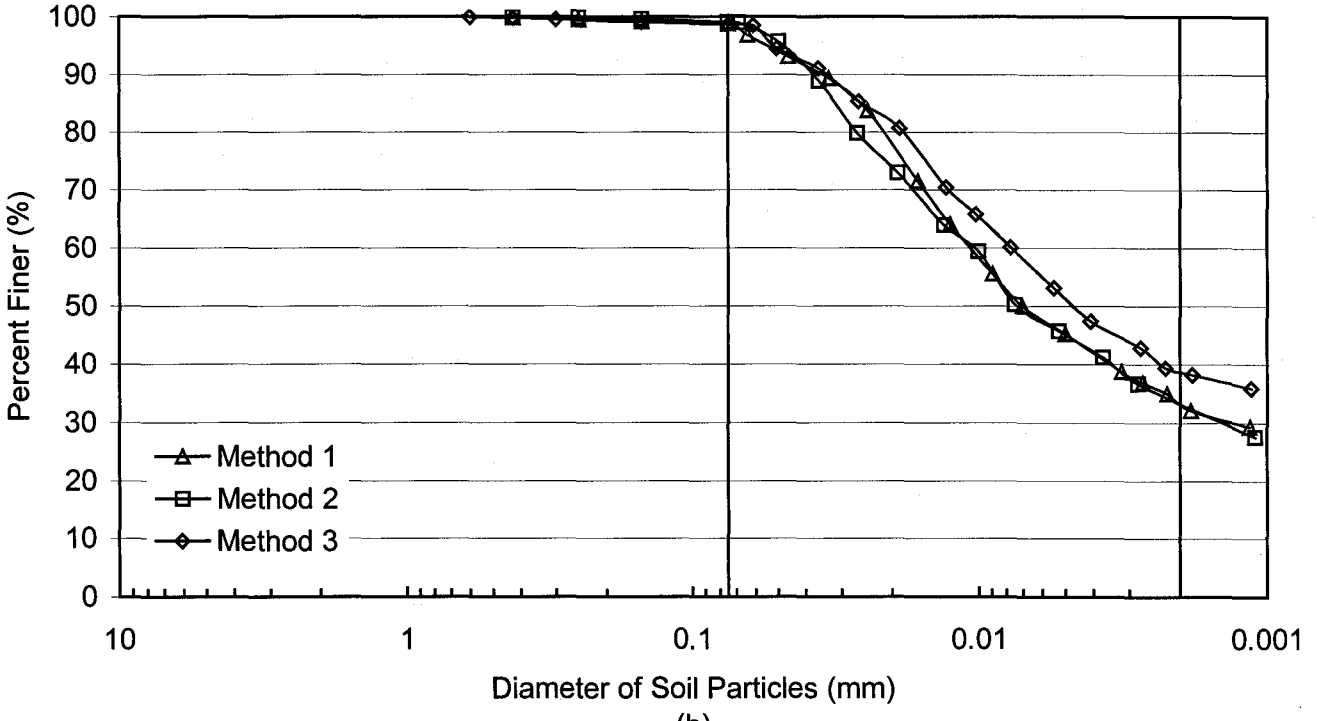
This method is the same as method 2 except that the soil retained on the No.60, 100 and 200 sieves was mixed with 100ml of standard sodium hexametaphosphate solution for several hours and stirred in a mechanical shaker. The deflocculated material was wet sieved through the stack of the three finest sieves again. The material retained on the sieves was

oven-dried to determine the dry weights while the fraction passing through 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 was subjected to hydrometer testing.

The results from all three methods are plotted in Fig. 4b. Methods 2 and 3 are the most reliable methods but they are the most tedious to perform because a significant amount of water has to be dried down to perform the hydrometer test. When the test results from the methods were compared, they all yielded similar results, although method 3 yielded a slightly finer grain size distribution because of the use of the deflocculant prior to wet sieving through the three smallest sieves. Because the differences are relatively insignificant, and for the sake of convenience, the grain size distributions of the soil from the other three locations were obtained using the simpler method 1.

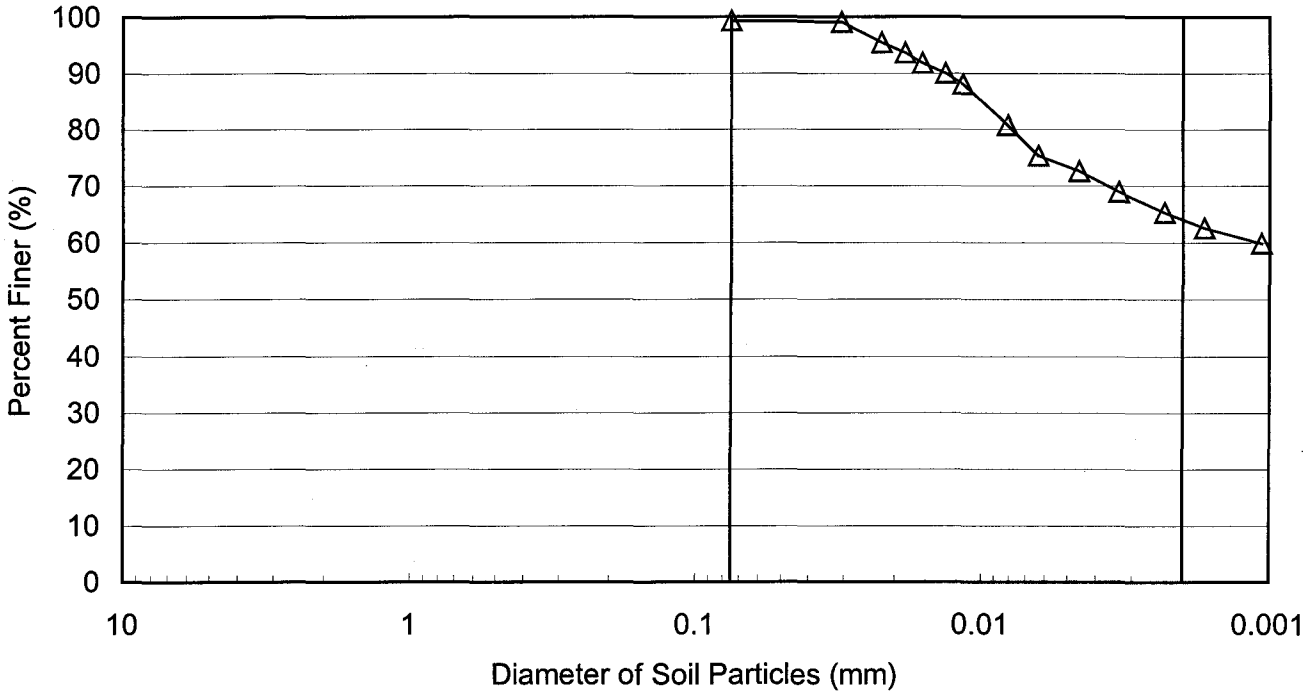


(a)

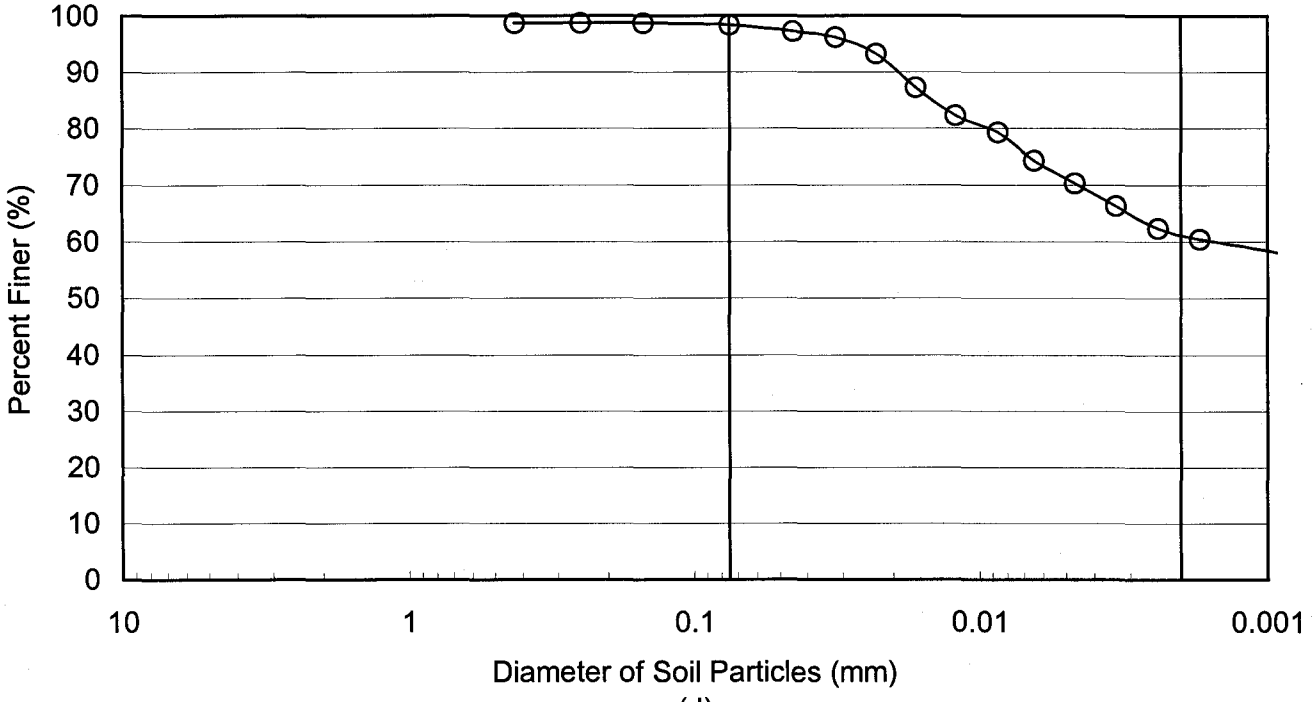


(b)

Figure 4. Grain Size Distributions for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa



(c)



(d)

Figure 4. (continued) Grain Size Distributions for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa

From Fig. 4, the sand, silt and clay fractions for each soil and the USCS group names and symbols are summarized in Table 2.

Table 2. Breakdown of Soil Type and Soil Classification

Location	Waipio	Kapolei	Mililani Mauka	Wahiawa
Sand Fraction (%)	12	1	1	1
Silt Fraction (%)	41	64	34	39
Clay Fraction (%)	47	35	65	60
Group Symbol	ML	ML	MH	MH
USCS Group Name	Silt	Silt	Elastic silt	Elastic silt

3.2.3 Specific Gravity

The specific gravity of the in situ soil solids was measured in accordance with ASTM Standard D854-98, the results of which are summarized in Table 3.

3.2.4 Sand Equivalent

The sand equivalent tests were performed in accordance with AASHTO T176-97 on the in situ soil from all four locations, the results of which are summarized in Table 4.

Table 3. Specific Gravity Test Results

Location	Sample No.	Specific Gravity
Waipio	1	2.82
	2	2.99
	3	2.90
	4	2.90
	Average	2.90
Kapolei	4	2.91
	23	2.97
	26	3.09
	27	3.04
	Average	3.00
Mililani Mauka	1	2.96
	2	2.94
	7	3.01
	10	3.01
	Average	2.98
Wahiawa	25	2.99
	25B	2.94
	35	3.06
	55	3.26
	56	3.22
	Average	3.09

Table 4. Sand Equivalent Test Results

Location	Sample No.	Sand Equivalent (%)
Waipio	1	8
	5	11
	18	13
	Average	10
Kapolei	4	8
	23	8
	26	7
	27	10
	Average	8
Mililani Mauka	1	11
	2	9
	7	10
	10	16
	Average	11
Wahiawa	56	14
	35	14
	55	13
	56B	18
	Average	14

3.2.5 Compaction

Stiffness and CBR tests were performed on 116-mm-high specimens compacted in a 150-mm-diameter mold. During compaction, a 61-mm-high spacer was placed at the bottom of the mold. Soil was compacted in five lifts using a 4.54 kg hammer in accordance with ASTM Standard D1883-94. A family of three compaction curves was obtained by varying the number of blows per lift from 56 to 25 to 10. The first compaction effort represents the modified Proctor compaction effort in accordance with Procedure C in ASTM Standard D1557-91. Three compaction efforts were performed since these specimens were later used for CBR testing to obtain a family of curves. Additionally, a fourth compaction curve was obtained by compacting the soils in three lifts using a 2.45 kg hammer at 56 blows per lift. This represents the standard Proctor compaction effort in accordance with Procedure C in ASTM Standard D698-91. The compaction energies are summarized in Table 5.

Table 5. Compaction Energy

Hammer Weight (kg)	Hammer Drop Height (mm)	Mold Diameter (mm)	No. of Lifts	No. of Blows	Compaction Energy (kN-m/m^3)
4.54	381	150	5	56	2680
4.54	381	150	5	25	1200
4.54	381	150	5	10	479
2.45	305	150	3	56	590

The compaction curves are plotted in Fig 5 for all four soils. In general, the ML soils have a higher maximum dry unit weight and a lower optimum water content than the MH soils.

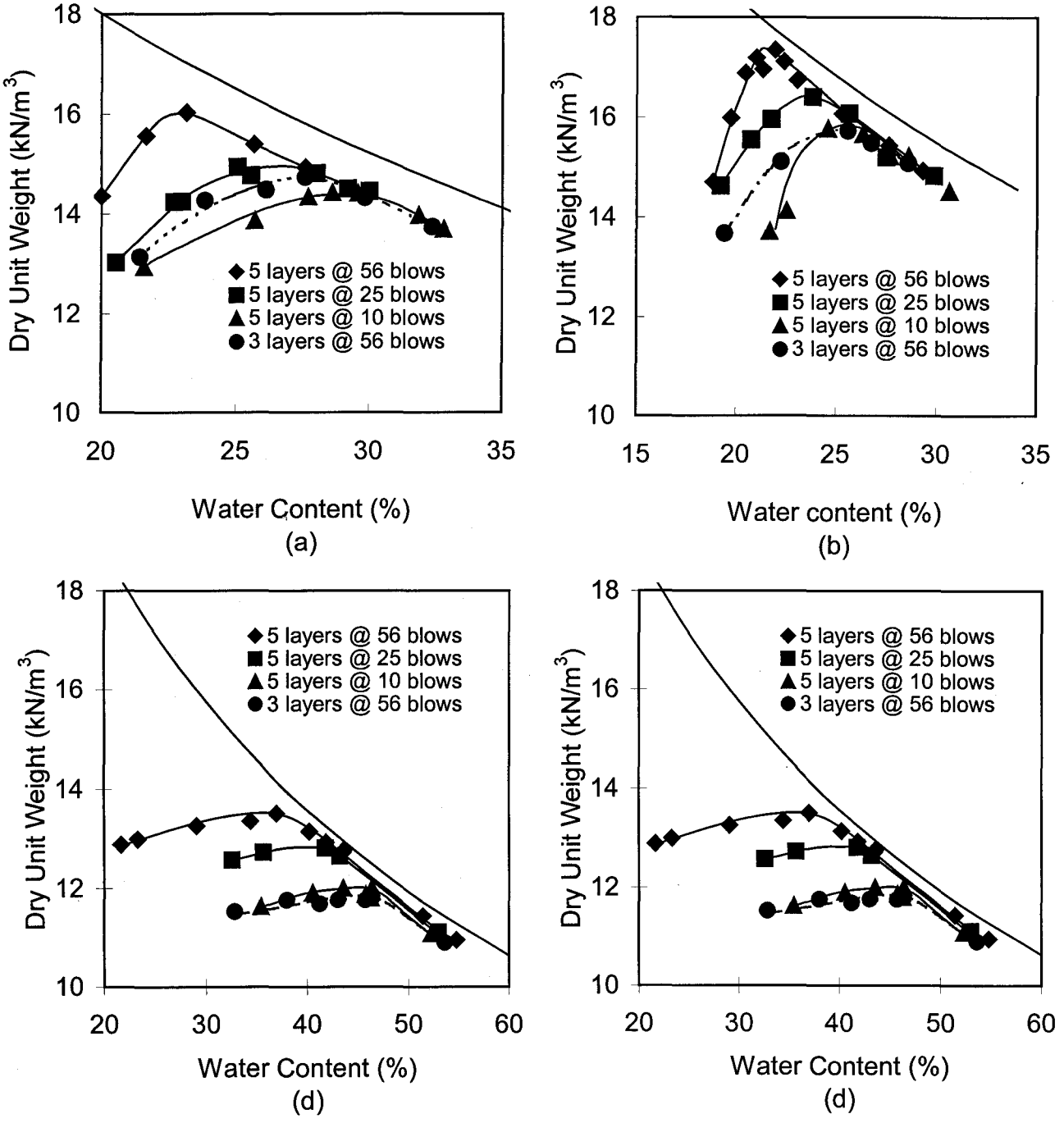


Figure 5. Results of Compaction Tests for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa

Chapter 4 GeoGauge™ Stiffness Test Results

After compaction, the spacer was removed, the mold was inverted so that the bottom of the soil specimen was flush with the base of the mold, which was bolted to the floor. GeoGauge™ stiffness measurements were made on the soil specimens in the mold immediately after compaction. Upon removal of the spacer, the top of the soil inside the mold was deeper than the length of the GeoGauge™ ring foot. To access the soil, a 64-mm-long aluminum extension to the ring foot, having the same I.D. and O.D., was used in the tests (Fig. 6). At least three determinations were made on each sample. The readings were very repeatable and an average was calculated. After stiffness testing, the samples were soaked for 4 days with an imposed surcharge of 6.82 kg. After soaking, the surcharge weights were removed and the stiffness remeasured. The surcharge load was then reapplied and CBR testing was performed.

Use of the ring foot extension on the GeoGauge™ stiffness is non-standard and the effects of its use on the stiffness were studied by performing duplicate GeoGauge™ tests in the free field - one with and one without the extension footing. Based on ninety-three comparative stiffness measurements at several sites in Hawai'i with cohesive soils, use of the extension results in a stiffness reduction of about 7% on average (coefficient of determination = 0.93) - see Figure 7. The effects of testing the soil in the mold as opposed to in the free field are compared in Section 4.3.

Figure 6. Ring Foot Extension for GeoGauge™

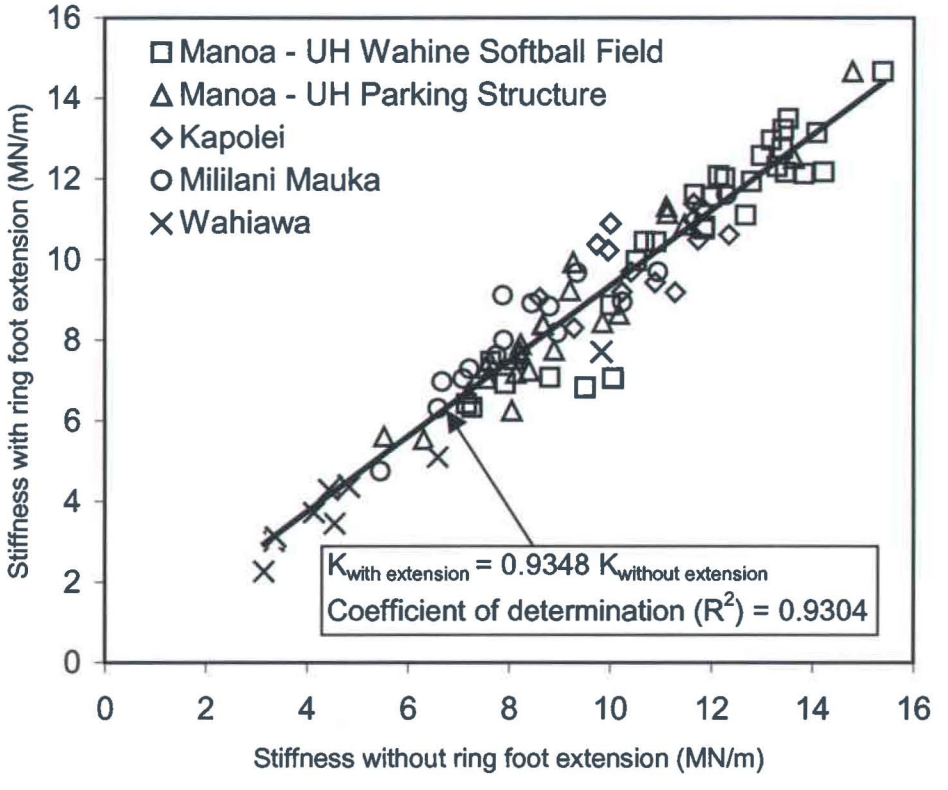


Figure 7. Comparison of GeoGauge™ Stiffness with and without the Ring Foot Extension

4.1 Laboratory Stiffness Tests

4.1.1 Immediately After Compaction

The results of GeoGauge™ stiffness tests for the four soils are summarized in Fig. 8 through Fig. 11. Compaction curves for Waipio, Kapolei, Mililani Mauka and Wahiawa are shown in Figs. 8a, 9a, 10a and 11a, respectively. Plots b, c and d in Fig. 8 through 11 are the post-compaction stiffness varying with dry unit weight, water content, and degree of saturation, respectively.

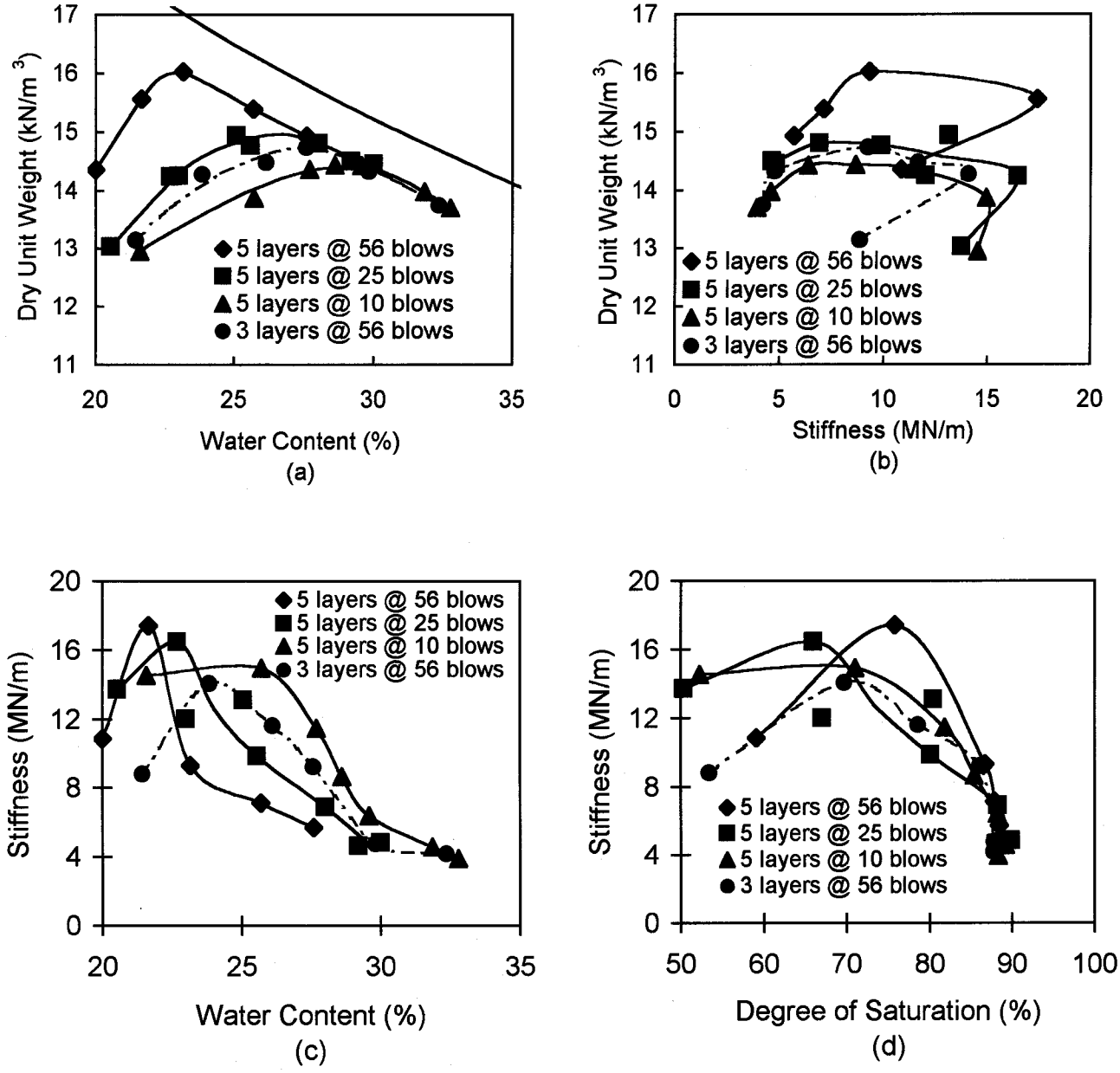


Figure 8. Results of GeoGauge™ Stiffness Testing for Waipio Soil (a) Compaction curves (b) Dry unit weight versus stiffness (c) Stiffness versus water content and (d) Stiffness versus degree of saturation

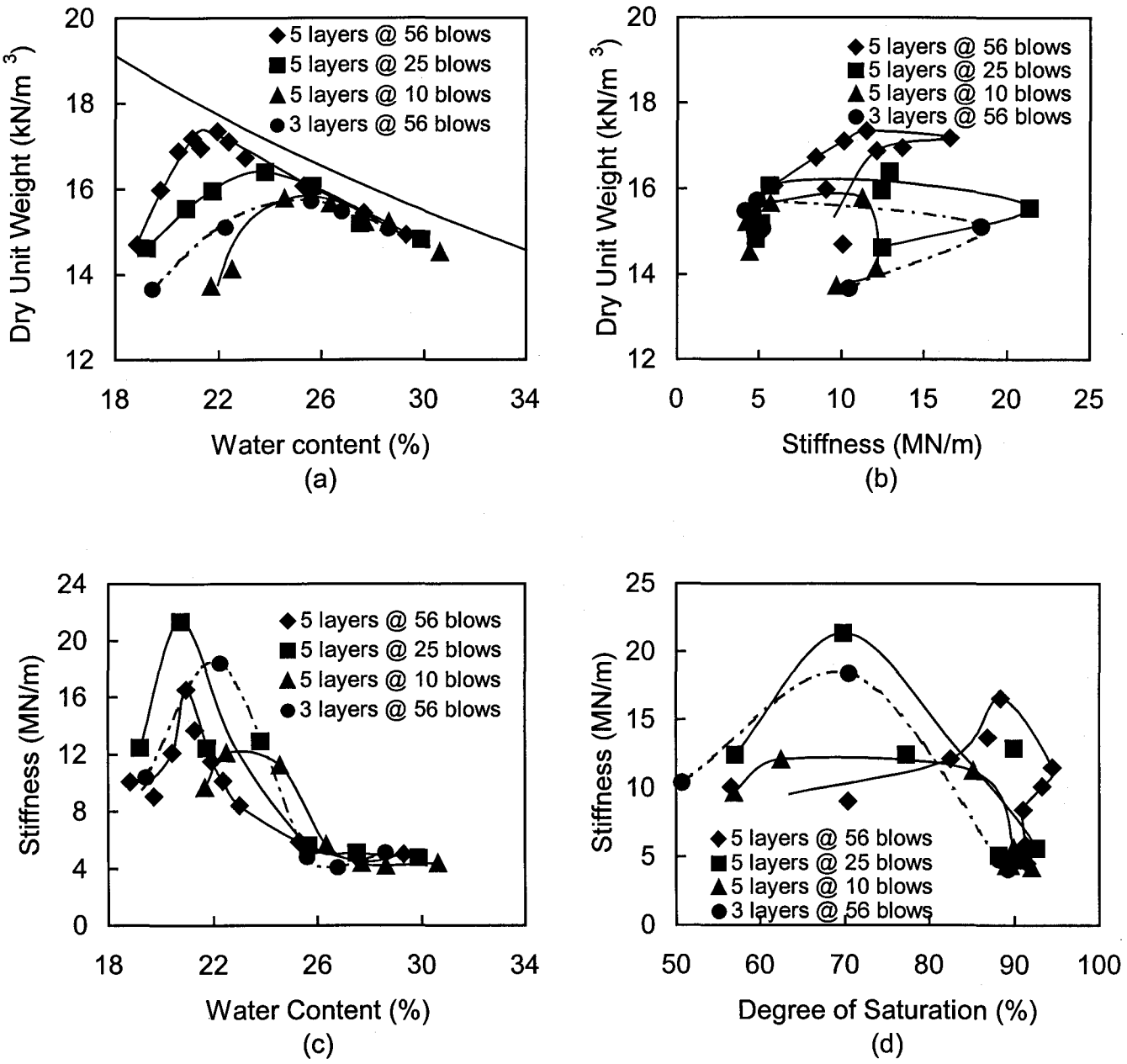


Figure 9. Results of GeoGauge™ Stiffness Testing for Kapolei Soil (a) Compaction curves (b) Dry unit weight versus stiffness (c) Stiffness versus water content and (d) Stiffness versus degree of saturation

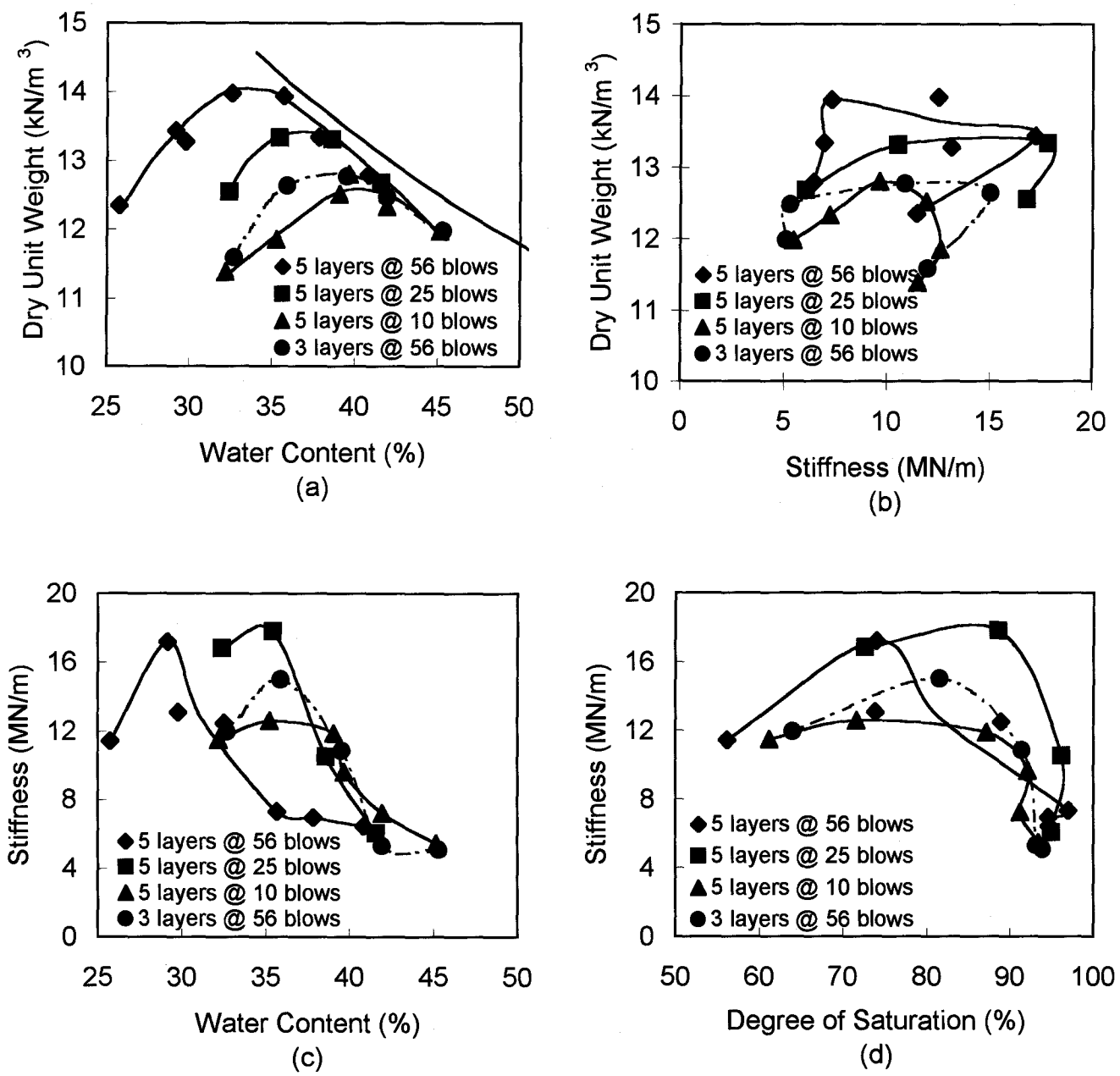


Figure 10. Results of GeoGauge™ Stiffness Testing for Mililani Mauka Soil (a) Compaction curves (b) Dry unit weight versus stiffness (c) Stiffness versus water content and (d) Stiffness versus degree of saturation

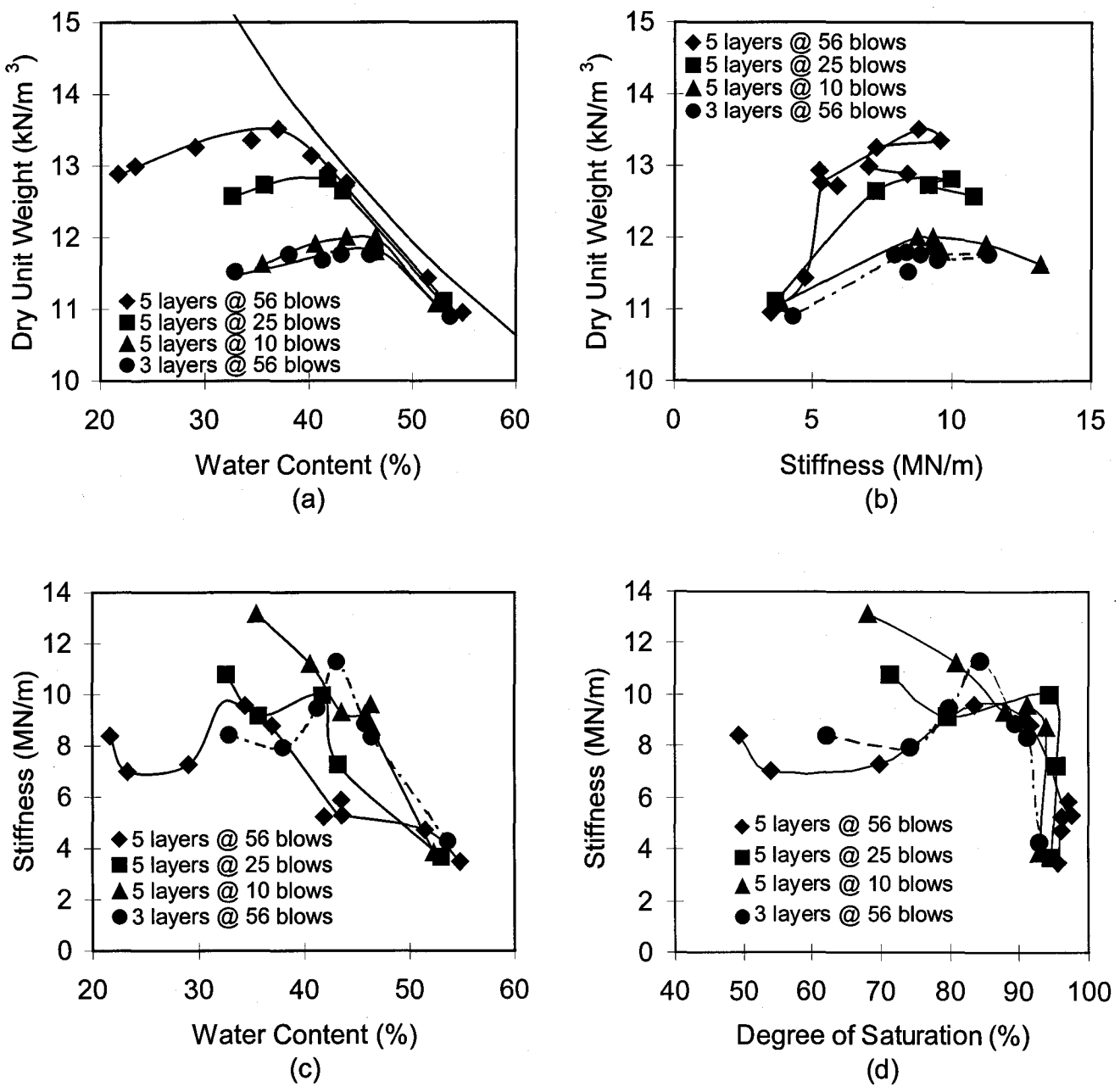


Figure 11. Results of GeoGauge™ Stiffness Testing for Wahiawa Soil (a) Compaction curves (b) Dry unit weight versus stiffness (c) Stiffness versus water content and (d) Stiffness versus degree of saturation

In general, the following observations can be:

1. The GeoGauge™ stiffness before soaking peaks dry of optimum. The water content at peak stiffness and at peak density do not coincide due to the following reasons:

   - Modulus generally increases with (i) increasing effective stress and (ii) decreasing void ratio. The vertical effective stress of partially saturated soils can be estimated using the following expression by Bishop et al. (1950):

     $$\sigma_v' = \sigma_v - u_a + \chi(u_a - u_w) \quad (1)$$

     where σ_v = total stress, u_a = pore air pressure, u_w = pore water pressure, $(u_a - u_w) = \psi$ = matric suction and χ = effective stress parameter. χ is zero for dry soils and unity for saturated soils. Soil matric suction can be extremely high at low water contents and decreases to zero at 100% degree of saturation. Effective stress is governed by the product $\chi\psi$. Thus, there must exist some value of degree of saturation at which $\chi\psi$ is greatest, and hence, effective stress and the GeoGauge™ stiffness are maximum. As the water content decreases from the optimum along the compaction curve, the void ratio increases but yet, the stiffness peaks. Therefore, when the water content decreases from the optimum, the influence of increasing effective stress (due to increased suction), which has a tendency to increase stiffness, far outweighs the effects of increasing void ratio, which has a tendency to reduce stiffness, resulting in the peak.

- Cohesive soils dry of optimum tend to be flocculated. As the molding water content increases, the compacted soil structure tends towards a dispersed state (Seed et al., 1960). In a flocculated structure, the soil particles orient themselves in an edge-to-face configuration because the edges are positively charged and the faces are negatively charged. These attractive forces "glue" the soil particles together, thereby giving rise to a higher stiffness on the dry side compared to at maximum dry unit weight, which has a more dispersed structure than the dry-of-optimum soil.

2. Three distinct portions are apparent in Fig. 8b and 8c for Waipio, Fig. 9b and 9c for Kapolei and Fig. 10b and 10c for Mililani Mauka. The first portion is dry of the peak stiffness. The second portion is between the peak stiffness and the maximum dry unit weight, where the stiffness drops sharply. The rate of decrease lessens wet of optimum, which forms the third portion. From these results, it is clear that stiffness is not directly related to dry unit weight. This is consistent with the findings of Fiedler et al. (2000) who observed significant scatter in the same correlation.

3. Fig. 8a and 8c can be used to look at the change of modulus with dry unit weight at constant water content for the Waipio soil. At a water content of say 26%, stiffness values decrease with increasing compaction effort (and hence dry unit weight) whereas stiffness values increase with increasing compaction effort at a water content of 22%. The effect of dry unit weight on stiffness can

be clearly seen by interpolating Fig. 8a and 8c and replotting stiffness versus dry unit weight at constant water contents in Fig. 12. At low water contents, stiffness increases with dry unit weight to a certain point. Thereafter, subsequent increases in dry unit weight results in a decreasing GeoGauge™ stiffness. For Waipio soil, at water contents higher than 25% and dry unit weights above 14 kN/m^3, stiffness decreases with increasing dry unit weight. This reduction is associated with water contents that are wet of the optimum stiffness. Therefore, loss of stiffness can occur as a result of overcompaction (excessive dry unit weight) or as a result of using too high a water content. It appears that soil stiffness must be complemented with information on the water content relative to the optimum value if it were to be used for compaction control purposes. Soils from Kapolei, Mililani Mauka and Wahiawa show similar trends.

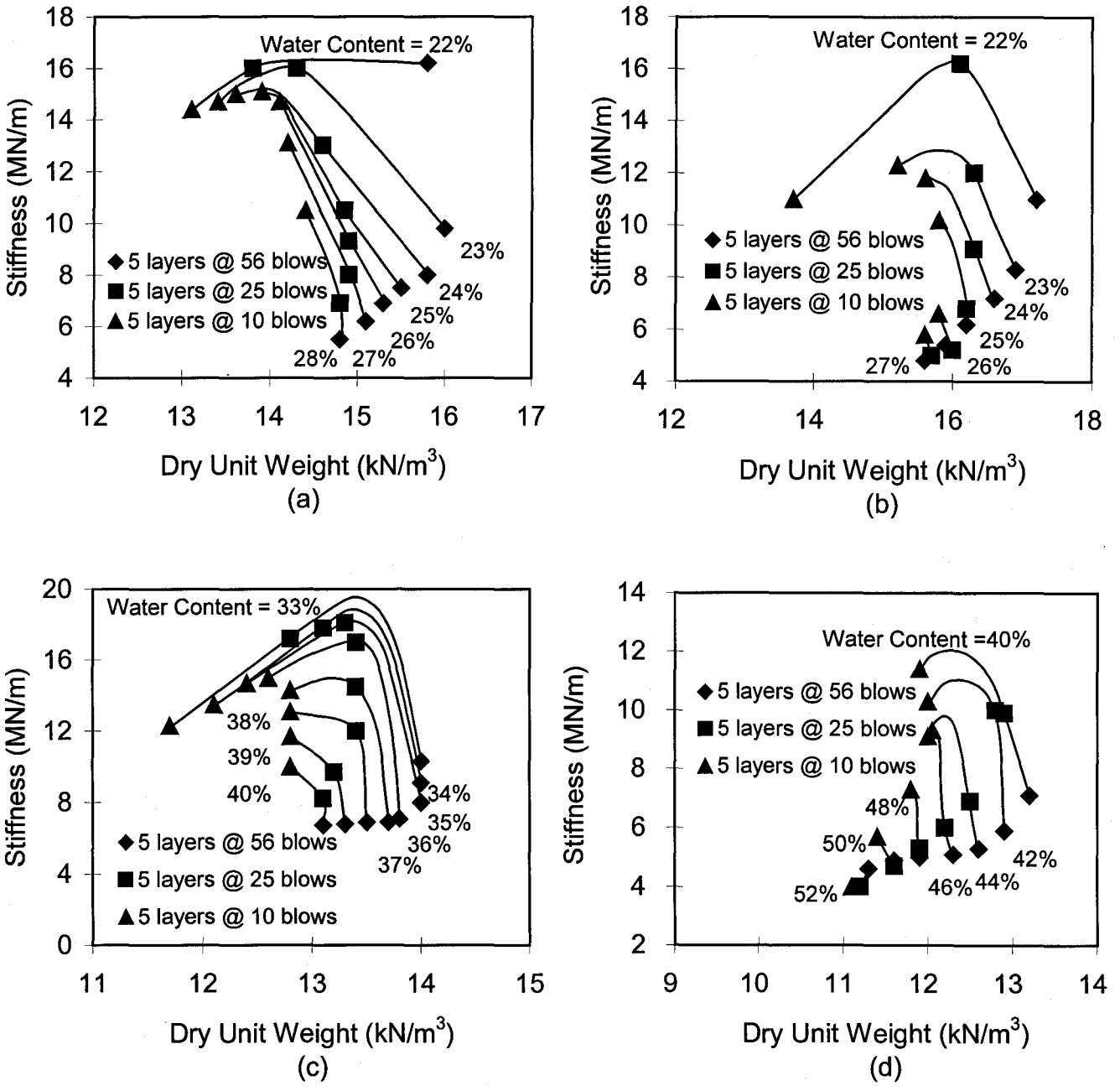


Figure 12. Stiffness vs. Dry Unit Weight at Constant Water Content for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa

4. All the peak stiffness values fall within a narrow range of degree of saturation, which are all less than the degree of saturation at optimum. Wu et al. (1984)

performed resonant column tests on several partially saturated cohesionless soils, where each sample was tested at several water contents after gradually drying the samples. They observed that the optimum degree of saturation varied between 5% and 20%. Using bender elements, Marinho et al. (1996) conducted similar tests on compacted London clay, and found that the optimum degree of saturation was between 75% and 85%. In this study, each sample was not gradually dried and tested over a range of water contents. Instead, each sample was tested at the molding water content, and each water content is associated with a different void ratio depending on its position on the compaction curve. Nevertheless, comparison of the values of optimum degree of saturation measured in this study with those from the study of others indicates consistency with the postulation of Wu et al. (1984) that an optimum degree of saturation for stiffness exists, and that it increases with decreasing effective grain size or D_{10}. These test results and results from other research are summarized in Table 6. Soils from Kapolei and Mililani Mauka also follow a similar trend as described above.

Table 6. Optimum Degree of Saturation for Several Soils

Soil	Liquid Limit (%)	Plastic Limit (%)	Clay fraction or Effective Size, D_{10} (mm)	Degree of Saturation at peak GeoGauge™ Stiffness (%)	Degree of Saturation at Maximum Dry Unit Weight (%)	Reference
London Clay (MH)	88	25	Clay fraction (% > 2μm) = 62%	75 to 85	N/A	Marinho et al. 1996
Mililani Mauka (MH)	98	44	Clay fraction (% > 2μm) = 65%			This study
5 layers @ 56 blows/layer				74	97	
5 layers @ 25 blows/layer				89	93	
5 layers @ 10 blows/layer				72	92	
3 layers @ 56 blows/layer				81	91	
Waipio (ML)	46	30	Clay fraction (% > 2μm) = 48%			This study
5 layers @ 56 blows/layer				76	89	
5 layers @ 25 blows/layer				66	89	
5 layers @ 10 blows/layer				71	89	
3 layers @ 56 blows/layer				71	88	
Kapolei (ML)	41	27	Clay fraction (% > 2μm) = 35%			This study
5 layers @ 56 blows/layer				88	95	
5 layers @ 25 blows/layer				70	93	
5 layers @ 10 blows/layer				65	88	
3 layers @ 56 blows/layer				70	89	
Glacier Way Silt	N/A	N/A	D_{10} = 0.0024	17.5	N/A	Wu et al. 1984
Glacier Way Sand	N/A	N/A	D_{10} = 0.03	10	N/A	
Flaky Sand	N/A	N/A	D_{10} = 0.085	9	N/A	
Beal Sand	N/A	N/A	D_{10} = 0.09	7.5	N/A	
Brazil Sand	N/A	N/A	D_{10} = 0.17	5	N/A	

5. The natural water contents for the four soils are shown in Table 7. The compaction and stiffness tests for Kapolei and Mililani Mauka were performed predominantly wet of the natural water content (i.e., testing was performed from dry to wet). For Waipio, the soil was tested on both air dry and wetted samples since the natural water contents range from 26% to 29% while the range of water contents at which the tests were conducted range from 20% to 33%. For the Wahiawa soil, the natural water content was significantly higher than those for the tested soil. Therefore, the Wahiawa soil was slowly air dried in increments from its in situ water content. It is believed that the soil from Wahiawa underwent irreversible changes upon drying, resulting in trends that did not resemble the other three soils. The Wahiawa soil test results and the effects of drying are detailed in Section 4.3.

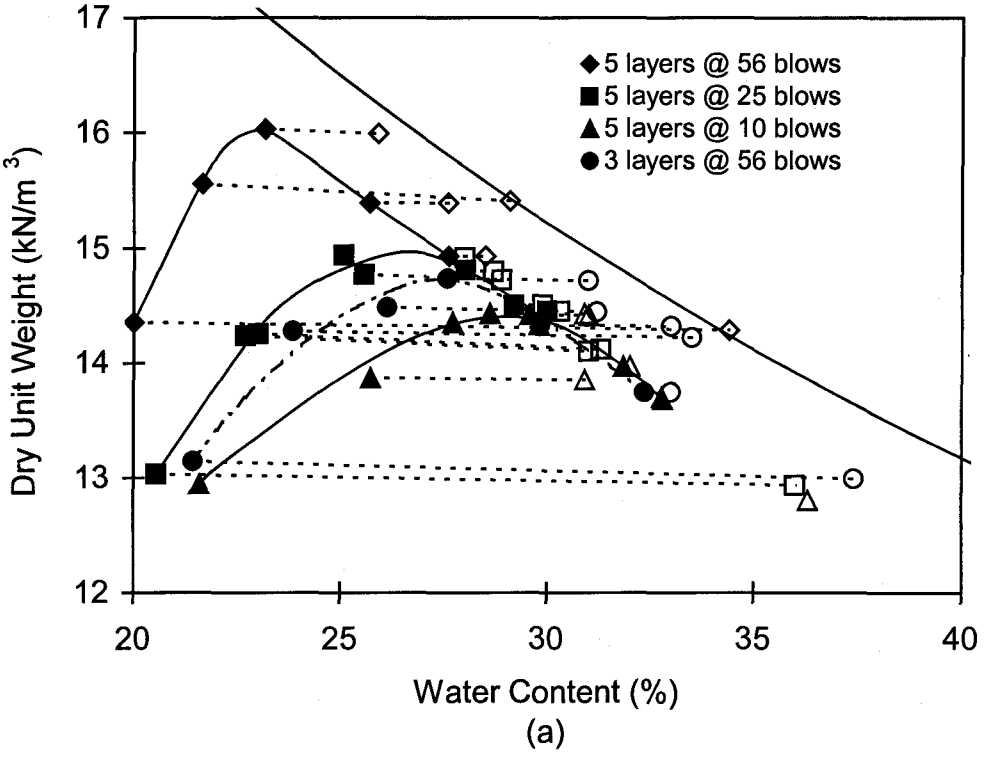
Table 7. Summary of In Situ Water Contents with Respect to the Optimum Values

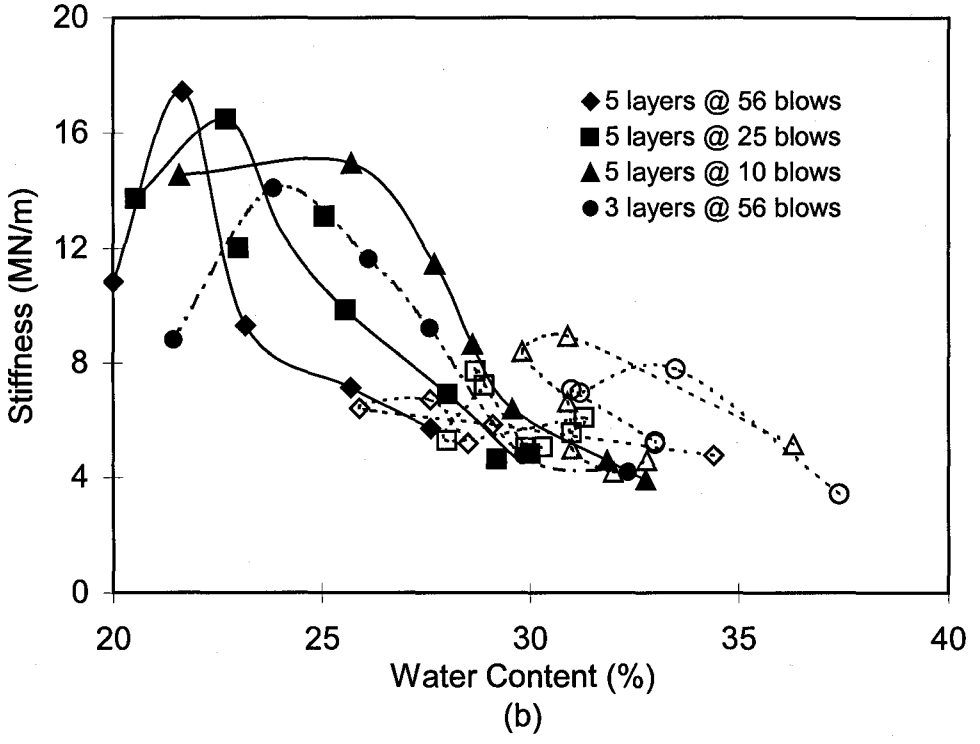
Location	In situ water content, w_n (%)	Optimum water content, w_{opt} (%)	Sample preparation
Waipio	26-29	23-29	$w_n \approx w_{opt}$
Kapolei	19-21	21-25	Mostly Wetting
Mililani Mauka	28-33	33-40	Mostly Wetting
Wahiawa	50-57	35-45	Mostly Drying

4.1.2 After 4 Days of Soaking

Plotted in Fig. 13 through 16 are the stiffness values after soaking versus dry unit weight and water content for the soils from Waipio, Kapolei, Mililani Mauka and Wahiawa, respectively. Stiffness values after soaking are shown as dashed lines for comparison with stiffness values immediately after compaction. The following observations are made:

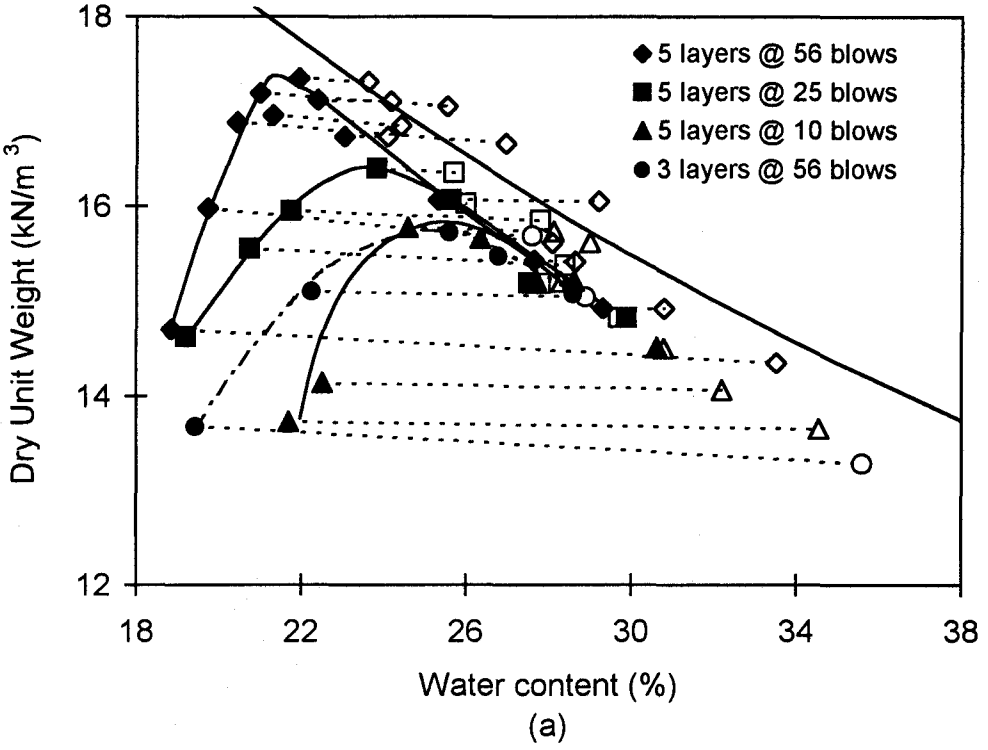
1. After soaking, water contents for the soil dry of optimum increase more significantly than those wet of optimum (Fig. 13a, 14a, 15a, and 16a). Some of the points plot to the right of the zero air void curve. This can be explained as follows. Water contents were determined not for the entire compacted specimen but were selected only from localized spots, especially from the top and the bottom, which tend to be wetter. It is likely that the middle of the specimen was not as wet and the water content for the entire specimen maybe have been overestimated as a result.
2. After soaking, the stiffness decreased. The decrease in stiffness is caused by (a) an increase in the water content, which in turn causes a decrease in soil suction, effective stress and hence soil stiffness; and (b) soil swell, which increases void ratio and decreases stiffness.
3. The decrease in stiffness for the dry-of-optimum soils is more significant than for soils wet of optimum (Fig. 17). This is because upon soaking the dry-of-optimum soils, the suction decrease is larger than for the wet-of-optimum soils, resulting in a drastic loss in stiffness.



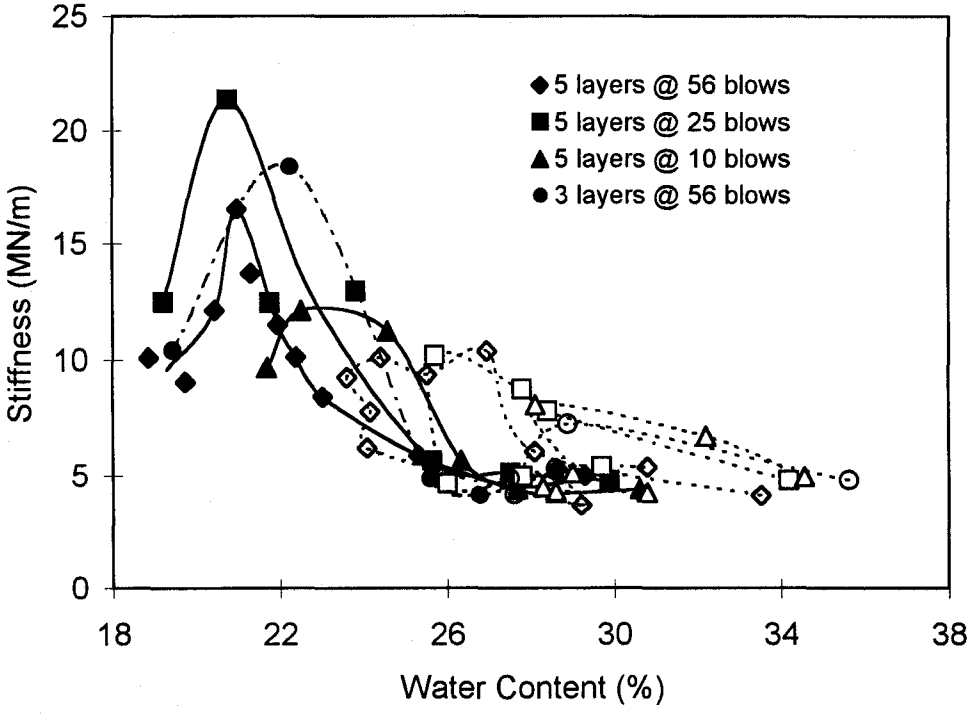


Solid symbols – Tested immediately after compaction
Hollow symbols – Tested after 4 days of soaking

Figure 13. Results of GeoGauge™ Stiffness Testing after 4 days of Soaking for Waipio Soil (a) Compaction curves (b) Stiffness vs. water content



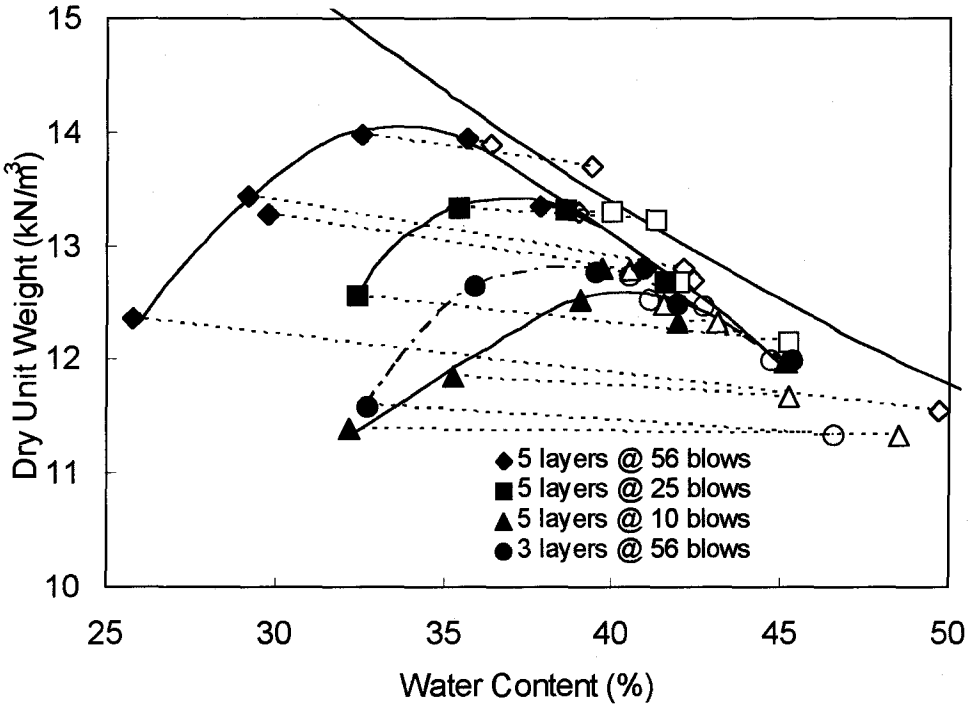
(a)



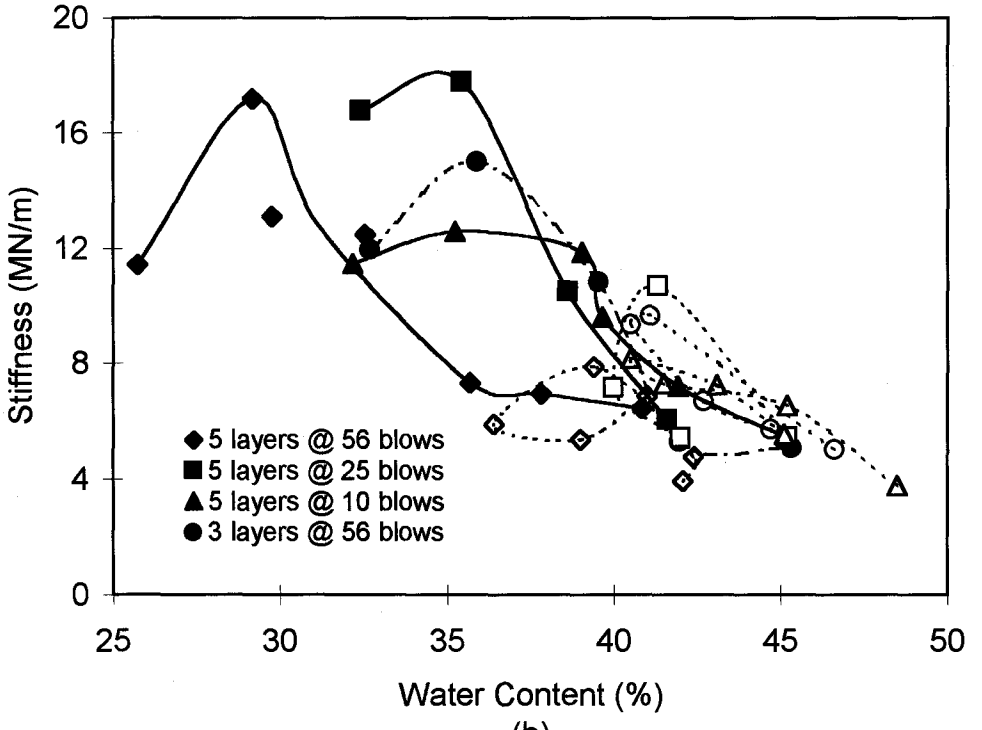
(b)

Solid symbols – Tested immediately after compaction
Hollow symbols – Tested after 4 days of soaking

Figure 14. Results of GeoGauge™ Stiffness Testing after 4 days of Soaking for Kapolei Soil (a) Compaction curves (b) Stiffness vs. water content



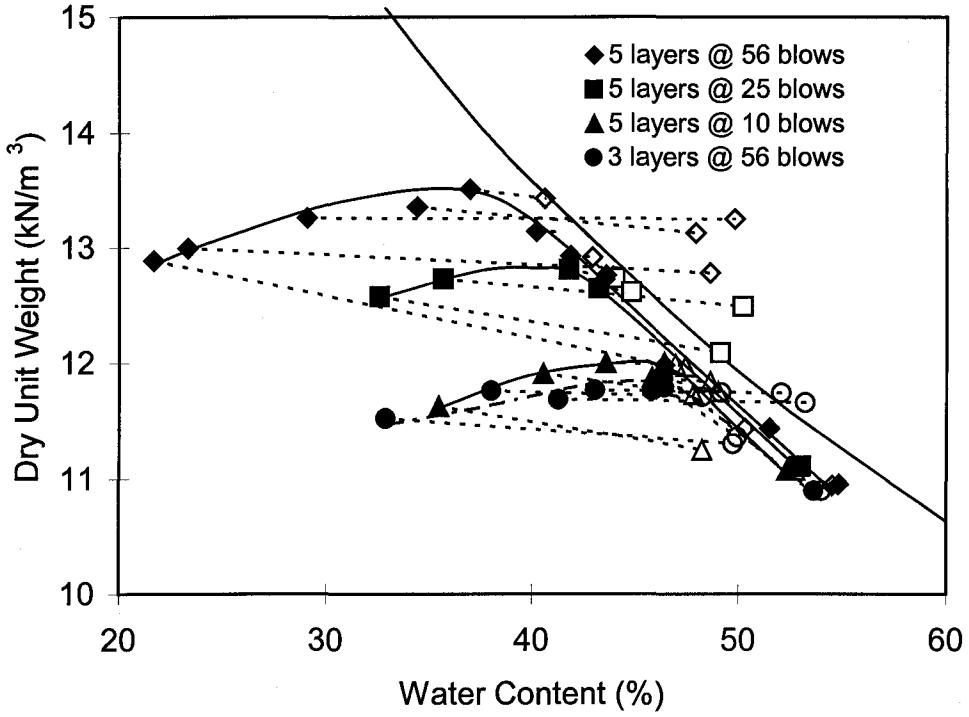
(a)



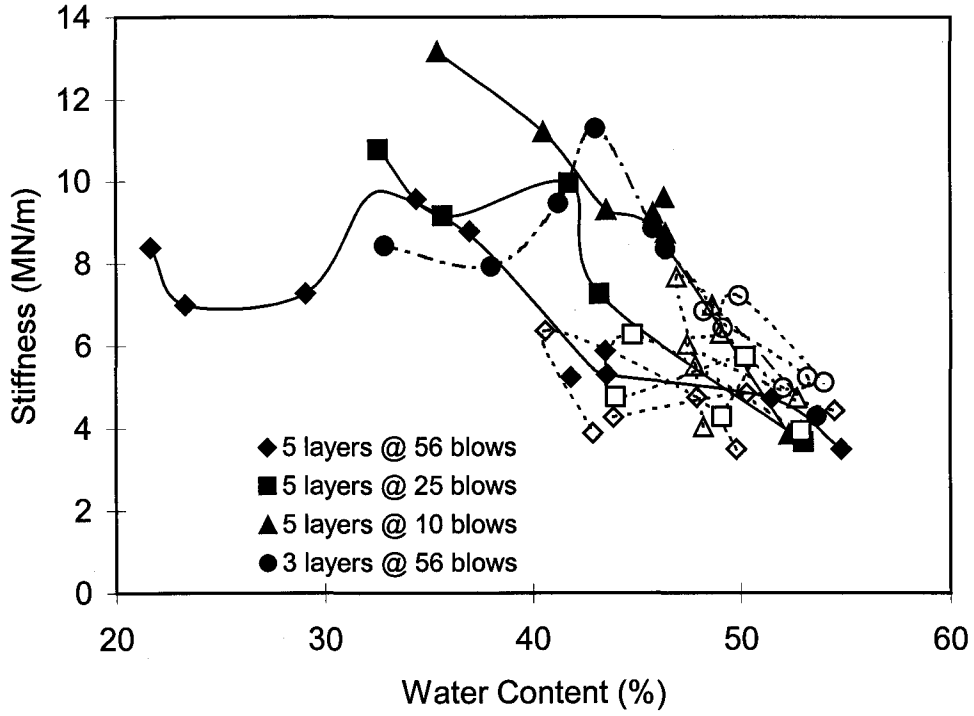
(b)

Solid symbols – Tested immediately after compaction
Hollow symbols – Tested after 4 days of soaking

Figure 15. Results of GeoGauge™ Stiffness Testing after 4 days of Soaking for Mililani Mauka soil (a) Compaction curves (b) Stiffness vs. water content



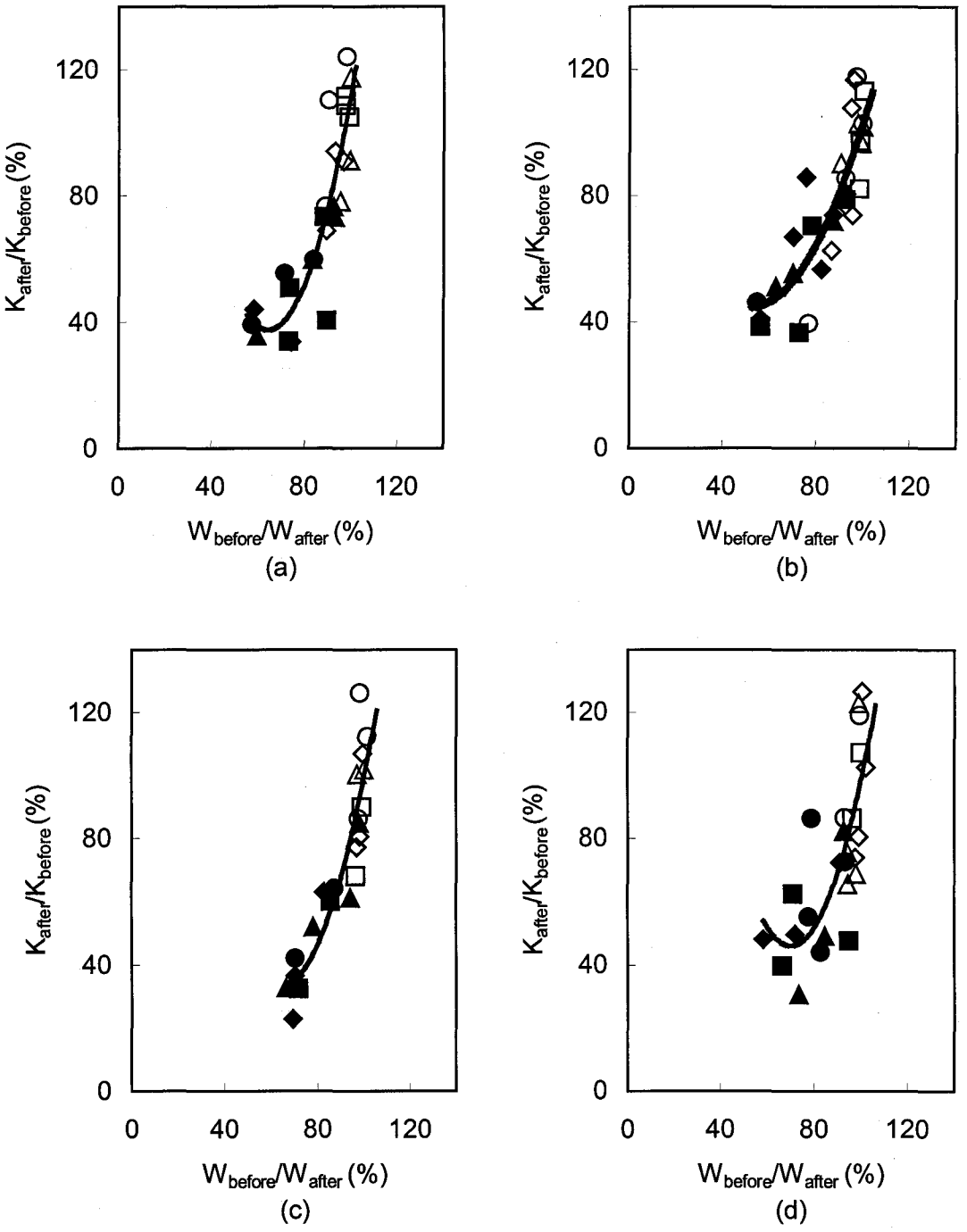
(a)



(c)

Solid symbols – Tested immediately after compaction
Hollow symbols – Tested after 4 days of soaking

Figure 16. Results of GeoGauge™ Stiffness Testing after 4 days of Soaking for Wahiawa Soil (a) Compaction curves (b) Stiffness vs. water content



LEGEND:
- ◆ 5 layers @ 56 blows - dry of optimum
- ■ 5 layers @ 25 blows - dry of optimum
- ▲ 5 layers @ 10 blows - dry of optimum
- ● 3 layers @ 56 blows - dry of optimum
- ◇ 5 layers @ 56 blows - wet of optimum
- □ 5 layers @ 25 blows - wet of optimum
- △ 5 layers @ 10 blows - wet of optimum
- ○ 3 layers @ 56 blows - wet of optimum

Figure 17. Normalized Stiffness versus Water Content before and after Soaking for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa.

4.1.3 Relationship between Low Strain Stiffness and Volume Change upon Wetting

Volume change below road pavements occurs when the soil absorbs or desorbs water. Volume change, especially in expansive and collapsing subgrades, can cause pavement distress and should ideally be minimized. Generally, swell in these soils tends to be higher when the soils are compacted dry of optimum (Seed 1959; Lawton 1989). 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 Fig. 18 and 19.

In Fig. 18a, swell for the Waipio soil decreases as the water content increases from 22% and above. In Fig. 18a, the swell contours indicate that volume change decreases with increasing water content as well as decreasing stiffness. Also, the maximum swell occurs close to the peak stiffness. Therefore, from a volume change standpoint, a high stiffness does not necessarily imply an ideal condition. The soils from the other locations exhibit similar trends as shown in Fig. 18 and 19. Therefore, to minimize volumetric expansion in compacted soils, they should be compacted (a) on the "wet side" and (b) such that the resulting stiffness is sufficiently low that there will be little tendency for volumetric expansion under the applied surcharge loading.

However, using too high a water content can compromise the strength of the soil (Seed 1959) as discussed in Section 4.1.4.

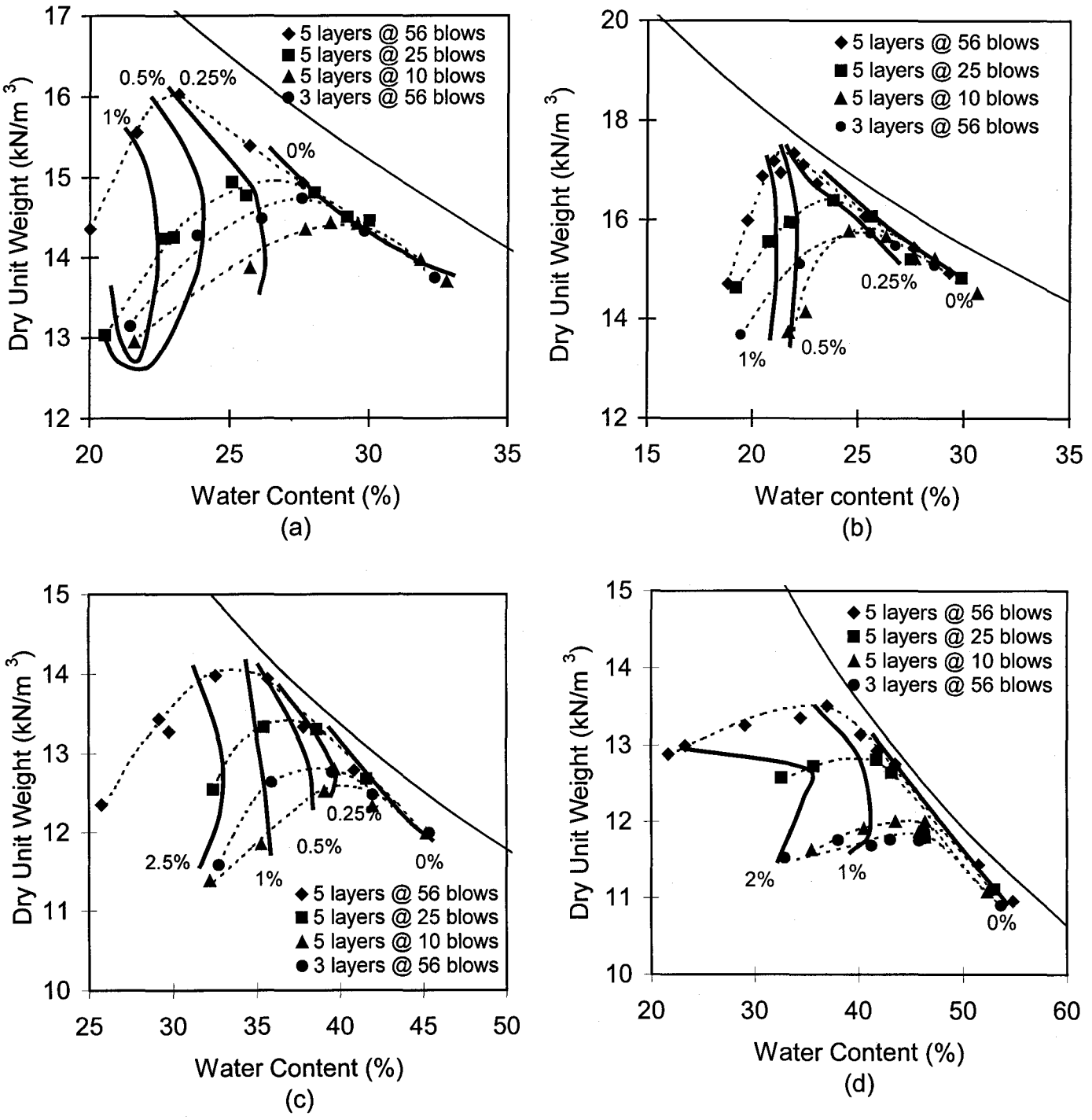


Figure 18. Swell Contour Lines in Compaction Curves for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa

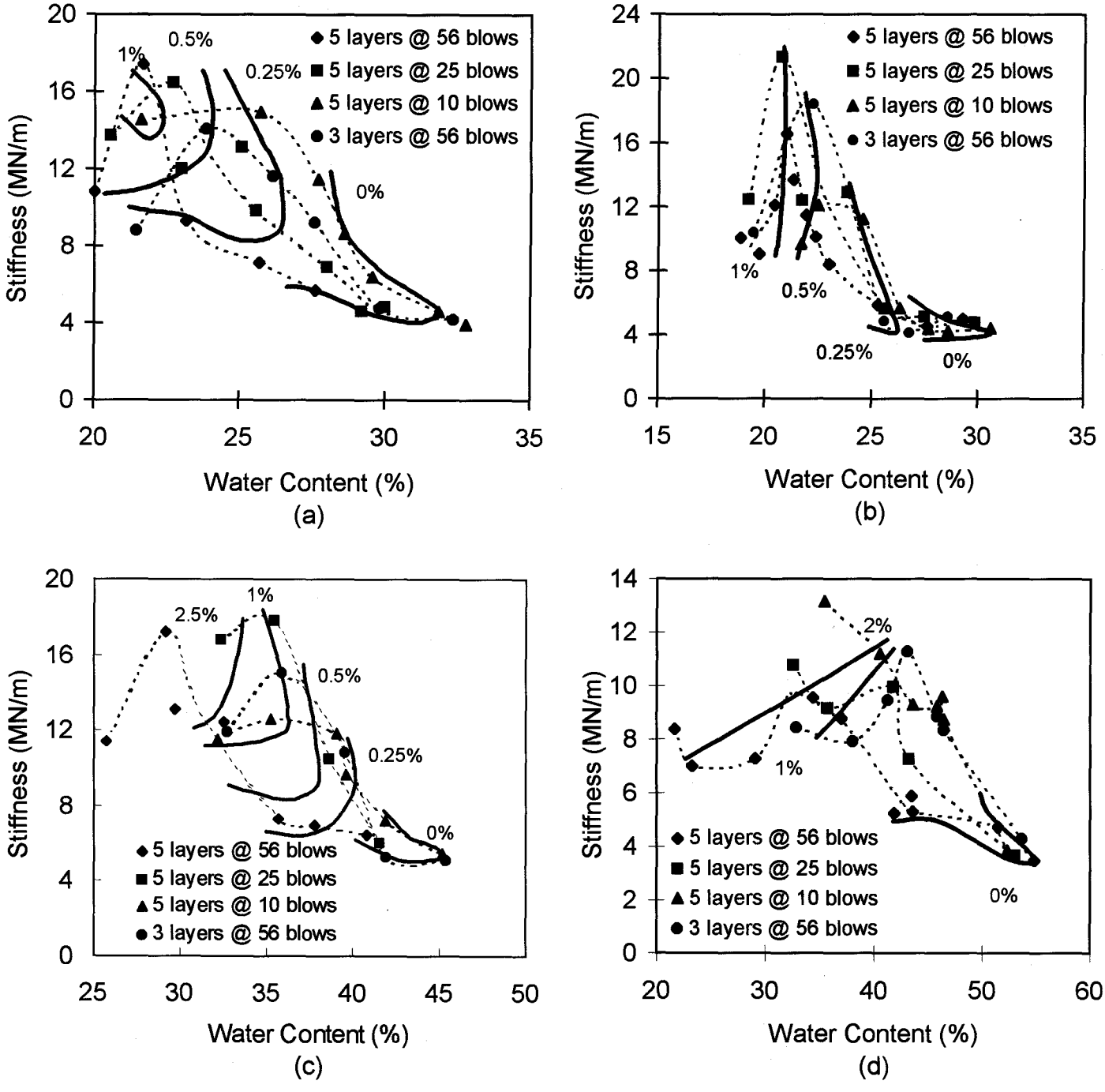
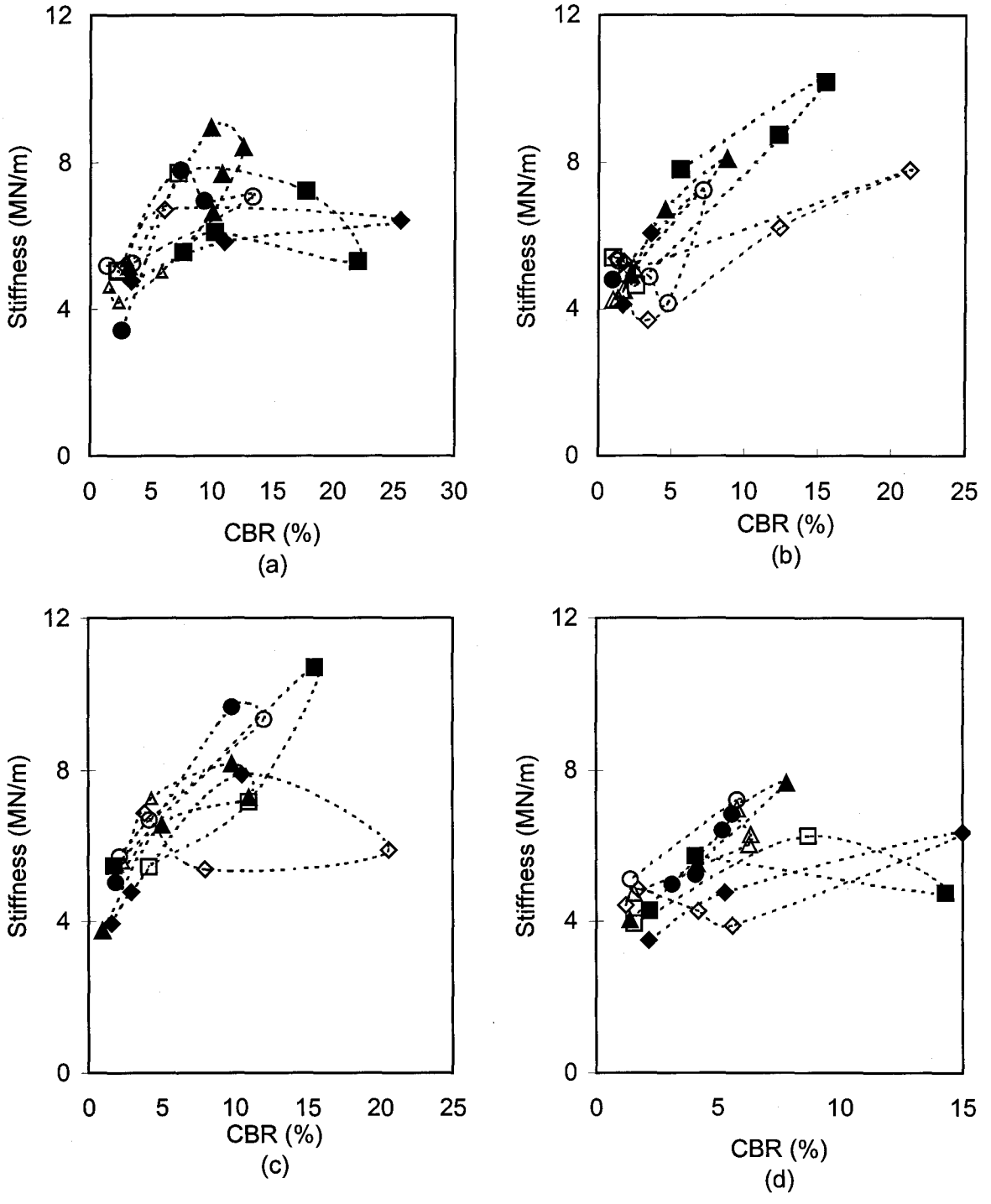


Figure 19. Swell Contour Lines in Stiffness vs. Water Content Plot for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa

4.1.4 Relationship between GeoGauge™ Stiffness and CBR

Seed et al. (1959) showed that the plot of strength versus water content for compacted cohesive soils is approximately z-shaped. The shear strength is greatest dry of optimum. It decreases sharply near the optimum water content and tends to level off to very low values wet of optimum. Since the peak stiffness occurs dry of optimum, the strength would correspondingly be high. As the molding water content increases, both stiffness and strength decrease. CBR tests (Hashiro, 2002) were performed in accordance with ASTM Standard D1883-94 on the same compacted samples after soaking for 4 days. A surcharge load of 6.82 kg was imposed on the sample during soaking and during CBR testing. Values of stiffness after soaking are plotted versus CBR in Figure 20. It is not expected that a direct relationship exists between stiffness values and the soaked CBR. According to the manufacturer, the GeoGauge™ provides a measure of soil stiffness at small displacements (< 0.00127 mm) while the CBR is measured at displacements of 2.5 to 5 mm. Since soil modulus is strain dependent, an improvement in the correlation between the two parameters would require information on the rate of moduli degradation with strain.



LEGEND:
◆ 5 layers @ 56 blows - dry of optimum
◇ 5 layers @ 56 blows - wet of optimum
■ 5 layers @ 25 blows - dry of optimum
□ 5 layers @ 25 blows - wet of optimum
▲ 5 layers @ 10 blows - dry of optimum
△ 5 layers @ 10 blows - wet of optimum
● 3 layers @ 56 blows - dry of optimum
○ 3 layers @ 56 blows - wet of optimum

Figure 20. Relationship between Stiffness and CBR for Soils from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa

4.2 Field Stiffness Tests

In situ tests (Fig. 21) were performed at Waipio, Kapolei, Mililani Mauka and Wahiawa. At each site, between 10 and 15 test locations were identified for testing. At each test location, the tests were conducted in the following sequence:

1. GeoGauge™ stiffness measurements
2. Nuclear gage testing (performed by Geolabs, Inc.)
3. Sand cone testing

In situ values of GeoGauge™ stiffness, dry unit weight and water content allow a comparison of the dry unit weight predicted via the GeoGauge™ with the more conventional sand cone and nuclear gauge methods using the methodology that is described in Section 4.3. A summary of the field test results is provided in Table 8.

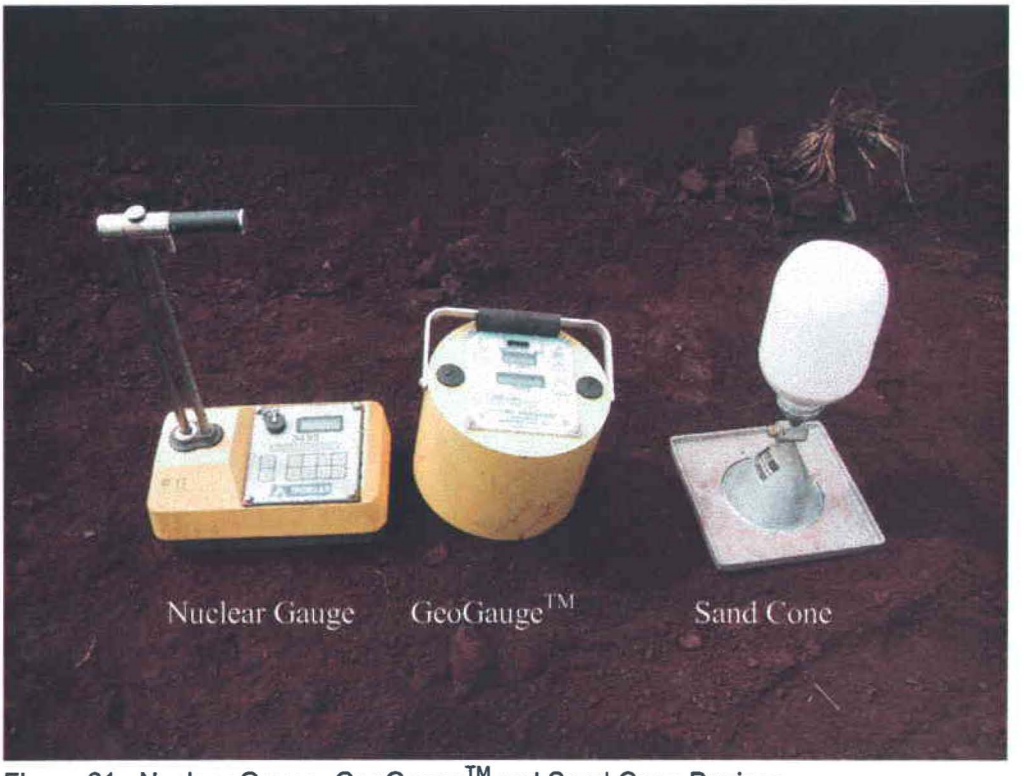


Figure 21. Nuclear Gauge, GeoGauge™ and Sand Cone Devices

Table 8. Summary of GeoGauge™, Nuclear Gauge and Sand Cone Test Results

Location	Dry Unit Weight (kN/m³)		Water Content (%)		GeoGauge™ Stiffness (MN/m)	
	Nuclear Gauge	Sand Cone	Nuclear Gauge	Sand Cone	w/o ring foot extension	w/ ring foot extension
Waipio						
1	11.8	12.4	28.0	28.6	5.01	Not performed
2	12.2	10.3	27.6	28.2	5.05	Not performed
3	11.6	9.7	28.3	28.9	4.61	Not performed
4	11.2	Not performed	28.3	Not performed	4.53	Not performed
5	11.9	13.5	28.0	26.6	5.10	Not performed
6	12.0	10.7	27.9	28.4	8.38	Not performed
7	13.8	15.0	26.6	27.0	9.07	Not performed
8	13.7	14.8	27.4	28.0	7.73	Not performed
9	13.3	11.4	25.8	26.5	7.62	Not performed
10	11.5	12.4	29.9	28.8	6.99	Not performed
Kapolei						
1	14.6	11.9	22.5	20.1	10.88	9.43
2	14.3	Not performed	22.8	Not performed	10.01	10.89
3	14.4	Not performed	23.2	Not performed	11.63	10.98
4	13.7	Not performed	25.7	Not performed	9.96	10.23
5	14.0	13.6	25.8	19.4	11.73	10.49
6	13.8	15.3	25.2	18.9	9.74	10.37
7	14.1	Not performed	24.9	Not performed	9.28	8.30
8	14.5	Not performed	24.4	Not performed	11.81	11.28
9	14.8	Not performed	23.8	Not performed	11.65	11.39
10	14.5	Not performed	23.3	Not performed	10.41	9.70
11	14.2	14.0	23.3	20.7	12.35	10.61
12	14.6	Not performed	23.0	Not performed	10.23	9.20
13	14.0	13.1	23.7	21.3	8.60	9.05
14	14.9	Not performed	23.0	Not performed	11.29	9.19
Mililani Mauka						
1	12.9	9.1	31.5	28.2	7.75	7.62
2	12.7	8.0	31.9	29.7	7.87	9.11
3	12.7	11.3	34.3	28.1	7.89	8.00
4	13.2	11.4	33.5	30.2	8.45	8.91
5	12.9	10.9	34.5	29.8	10.94	9.69
6	12.8	14.0	34.3	30.2	10.25	8.94
7	12.3	10.7	34.2	30.9	9.34	9.67
8	13.3	12.1	36.2	31.4	8.95	8.18
9	13.1	13.1	36.1	31.9	8.80	8.83
10	12.9	11.4	32.9	32.1	6.60	6.31
11	12.2	Not performed	36.2	Not performed	7.09	7.04
12	12.9	Not performed	36.1	Not performed	5.46	4.75
13	12.0	Not performed	36.0	Not performed	6.67	6.96
14	12.1	11.0	38.3	33.4	7.22	7.28
15	12.3	Not performed	38.0	Not performed	12.30	11.61

Table 8. (Continued) Summary of GeoGauge™, Nuclear Gauge and Sand cone Test Results

Location	Dry Unit Weight (kN/m³)		Water Content (%)		GeoGauge™ Stiffness (MN/m)	
	Nuclear Gauge	Sand Cone	Nuclear Gauge	Sand Cone	w/o ring foot extension	w/ ring foot extension
Wahiawa						
1	10.7	9.8	63.7	52.0	9.83	7.71
2	11.3	10.4	54.6	50.6	6.59	5.09
3	11.1	10.2	56.0	52.9	4.55	3.45
4	10.8	9.6	54.1	50.6	3.35	3.01
5	10.6	9.7	59.8	52.3	4.43	4.29
6	10.9	10.4	61.3	56.9	4.82	4.41
7	10.4	8.4	59.2	55.3	4.14	3.72
8	10.8	9.3	59.0	50.7	3.38	3.12
9	11.2	10.3	57.0	51.1	3.15	2.26
10	10.5	9.8	57.9	52.3	4.83	4.35

A comparison of sand cone and nuclear gauge test results is shown in Fig. 22

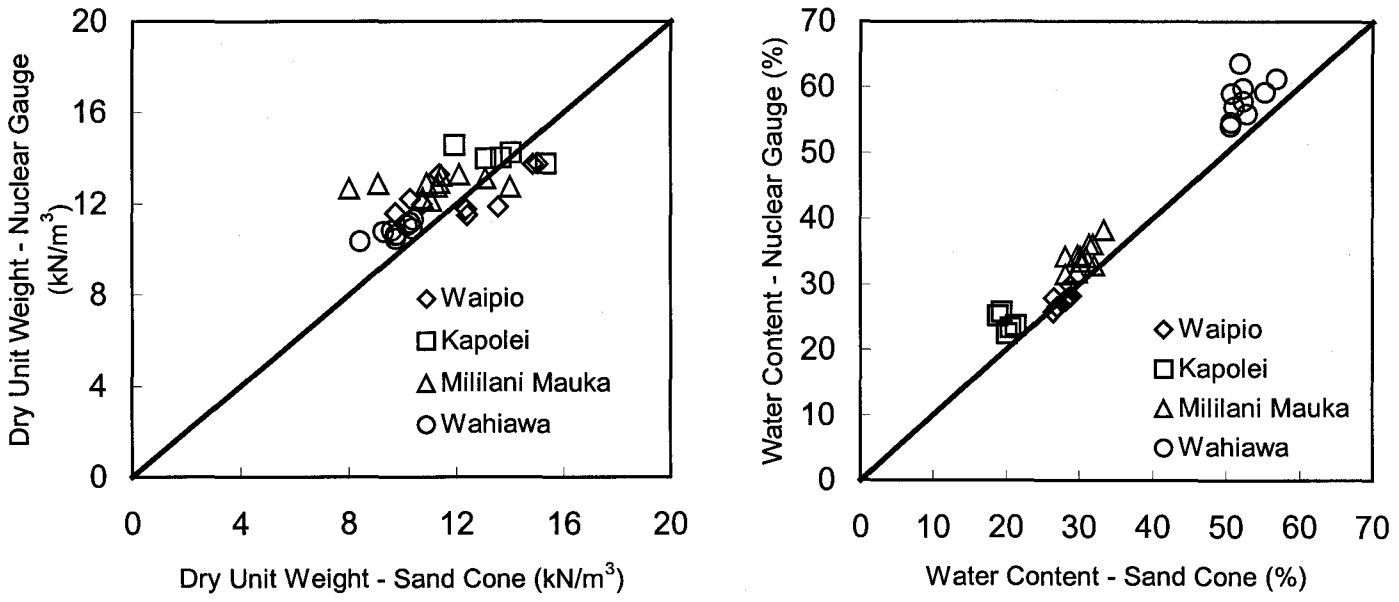


Figure 22. Comparison of Dry Unit Weight and Water Content from Nuclear Gauge and Sand Cone Tests

4.3 Relationship between GeoGauge™ Stiffness, Dry Unit Weight and Water Content

There has been a recent push by the FHWA towards adoption of the in-place stiffness as a means of assessing compactness of geomaterials. Stiffness on its own cannot be related to dry unit weight. For example in Figures 8b, 9b, 10b and 11b, a given stiffness value can correspond to several values of dry unit weight depending on the water content. Therefore, to successfully relate stiffness and dry unit weight, the water content must be known. Until the modulus is adopted completely for assessing compactness of geomaterials, a relationship between stiffness, dry unit weight and water content provides a useful understanding of their inter-relationship in the interim.

Based on the work of Egorov (1965) for a ring footing on a soil that approximates a homogeneous, isotropic, linear elastic half space, the stiffness, K is related to the Young's modulus of the soil, E, as follows:

$$K = \frac{F}{\delta} = \frac{ER_o}{\omega[1 - v^2]} \qquad (2)$$

where F = force on the ring, δ = ring displacement, ω = constant that is a function of the ratio R_i/R_o, R_i, R_o = inside and outside radius of the ring, respectively, and ν = Poisson's ratio of the soil.

For the laboratory compacted specimens, stiffness values were measured in a mold instead of a "half space." Therefore, a correction factor relating the soil stiffness in the mold to the free-field stiffness would be useful. An axi-symmetric finite element analyses was performed to simulate static loading of a ring footing on the soil in the mold assuming the soil is linearly elastic. Both fixed (rough) and free (smooth) boundary conditions of the soil/mold interface were assumed. The actual boundary conditions will likely tend towards the fixed case because of adhesion between the soil and the mold. In general, soils with higher degrees of saturation are likely to have a higher Poisson's ratio. However, for Poisson's ratio up to 0.4, the soil stiffness measured in the mold is approximately twice the free-field value (Fig. 23). This correction applies for all values of soil stiffness.

The effects of dynamic loading of the GeoGauge™ were not studied. In fact, the instrument may have generated waves that can reflect from the wall and base of the mold. The effects of reflection may have affected the wave propagation velocities and thus the soil stiffness. Moreover, the GeoGauge™ measures the dynamic force on the footing as opposed to the static force. This includes the force due to the soil inertial mass. Stiffness measurements of the soil in the mold have not been backed up by more accepted tests. Ideally, boundary effects of the mold should be investigated by performing low strain modulus measurements using other devices, such as bender element testing, to verify the trends observed with the GeoGauge™.

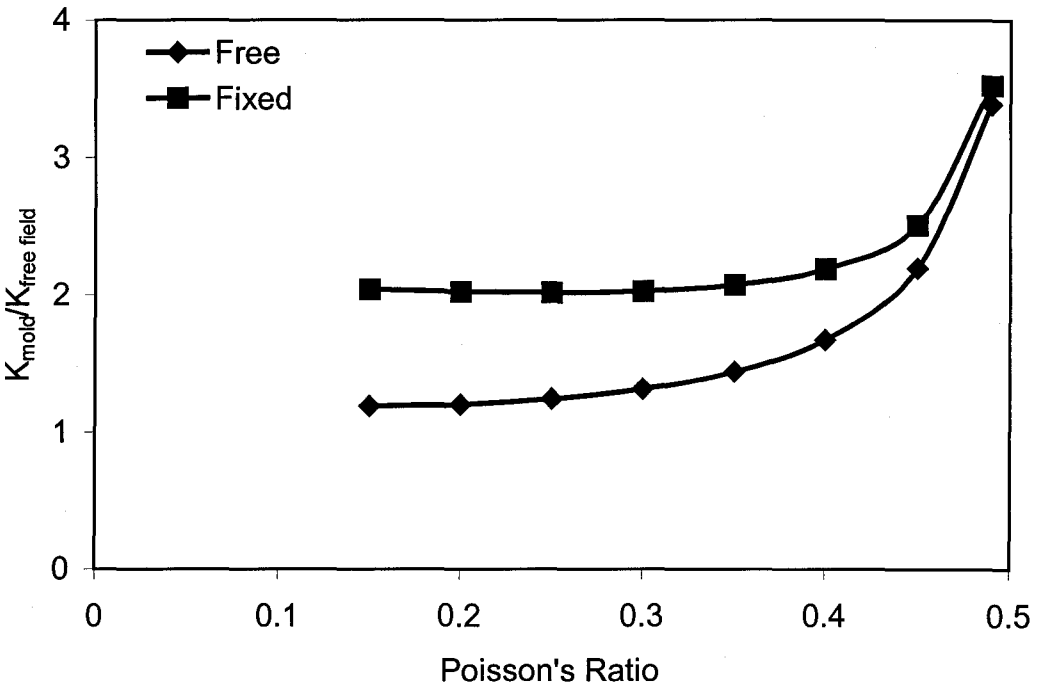


Figure 23. Correction Factor for Stiffness in the Mold

For linear elastic materials, the shear and Young's moduli are related as follows:

$$E = 2G(1+v) \tag{3}$$

where G is the shear modulus. Combining Equations (2) and (3), the shear modulus is related to stiffness as follows:

$$G = \frac{K\omega(1-v)}{2R_o} \tag{4}$$

Several models that relate shear modulus with effective stress and void ratio have been proposed (Hardin & Richart, 1963; Hardin & Black, 1968; Marcuson & Wahls, 1972; Shibata & Soelarno, 1975; Hardin, 1978; Kokusho et al. 1982; Hryciw & Thomann, 1993; Lo Presti, 1995; Bellotti et al. 1996 and Lo Presti et al. 1997). The general form of these models is:

$$G = Ap_a f(e) g(\sigma') \tag{5}$$

where A is a constant, p_a is atmospheric pressure, f(e) is a void ratio function and $g(\sigma')$ is a dimensionless effective stress function. Lo Presti (1995), Bellotti et al. (1996) and Lo Presti et al. (1997) used the following form of f(e):

$$f(e) = e^{C_2} \tag{6}$$

where C_2 is a negative constant. Fioravante & Capoferri (2001) suggested the following form of $g(\sigma')$ for axi-symmetric loading:

$$g(\sigma') = \left(\frac{\sigma_v'}{p_a}\right)^{n_v} \left(\frac{\sigma_r'}{p_a}\right)^{n_r} \tag{7}$$

where σ_v' and σ_r' are the vertical and radial effective stresses, respectively, and n_v and n_r are constants. By setting $\sigma_r' = K_0\sigma_v'$, where K_0 is the at-rest lateral earth pressure coefficient,

Equation (7) becomes:

$$g(\sigma') = K_0^{n_r} \left(\frac{\sigma_v'}{p_a}\right)^{(n_v + n_r)} \tag{8}$$

Combining Equations (4), (5), (6) and (8), the relationship between the GeoGauge™ stiffness, void ratio and effective stress can be expressed as:

$$K = C_1 e^{C_2} \left(\frac{\sigma_v'}{p_a}\right)^{C_3} \tag{9}$$

where C_1, C_2 and C_3 are constants determined using linear regression. Void ratio and dry unit weight are related as follows:

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 \tag{10}$$

Therefore, by substituting Equation (10) into (9), the dry unit weight can be estimated from stiffness and σ_v' as follows:

$$\gamma_d = \frac{G_s \gamma_w}{1 + \left[\frac{K}{C_1 \left(\frac{\sigma_v'}{p_a} \right)^{C_3}} \right]^{1/C_2}} \tag{11}$$

For partially saturated soils, σ_v' is estimated using Equation (1). The matric suction can be estimated by running a series of suction tests. The data can be used to develop a family of soil water characteristic curves (SWCC), which relate the soil matric suction to water content. The SWCC is discussed further in Section 4.3.1. Khalili & Khabbaz (1998) proposed the following to estimate the effective stress parameter, χ:

$$\chi = \left[\frac{\psi}{a} \right]^{-0.55} \leq 1 \tag{12}$$

where a = air entry value of the matric suction. The air entry value of the matric suction can be inferred from the SWCC.

4.3.1 Soil Suction and the Soil-water Characteristic Curve

Studies by Marinho et al. (1995), Phillip and Cameron (1996), Picornell and Nazarian (1998), and Gehling et al. (1998) have shown that the stiffness of partly saturated soil is influenced by suction. This further reinforces the fact that effective stress, which is related to suction for partially saturated soils, must be introduced if a good correlation between G_{max} and γ_d is desired

The soil-water characteristic curve defines the relationship between the soil matric suction and water content. It is generally S-shaped and provides a measure of the water holding (or storage) capacity of the soil as the water content changes (Vanapalli et al., 1999). Several equations have been proposed in the literature to model the soil-water characteristics. Some of these fitted functions are summarized in Table 9. The three more common methods used in geotechnical engineering are the Brooks and Corey (1964), Fredlund and Xing (1994), and van Genuchten (1980) models.

Table 9. Soil-water Characteristic Curves

Reference	Equation		Parameters
Brooks and Corey (1964)	$\theta = \theta_r + (\theta_s - \theta_r)\left(\frac{\psi_a}{\psi}\right)^{\lambda}$	(13)	θ = volumetric water content. θ_s = saturated volumetric water content. θ_r = residual volumetric water content ψ_a = air-entry value of matric suction or bubbling pressure. λ = pore size index.
Fredlund & Xing (1994)	$\theta = \frac{C\theta_s}{\left[\ln\left\{e + \left(\frac{\psi}{a}\right)^n\right\}\right]^m}$ $C = 1 - \frac{\ln\left(1 + \frac{\psi_r}{\psi}\right)}{\ln\left(1 + \frac{1{,}000{,}000}{\psi_r}\right)}$	(14) (15)	ψ_r= matric suction at the residual volumetric water content C = correction function a = soil parameter primarily a function of the air-entry value of the matric suction e = natural base of logarithm = 2.71828 m = soil parameter primarily a function of the residual water content n = soil parameter primarily a function of the rate of water extraction from the soil when ψ air-entry value
Van Genuchten (1980)	$\theta = \theta_r + \frac{\theta_s - \theta_r}{\left[1 + (\alpha\psi)^n\right]^m}$	(16)	α = soil parameter primarily a function of the air-entry value of the matric suction (kPa^{-1} if ψ is in kPa) m = soil parameter primarily a function of the residual water content = $1 - n^{-1}$. n = soil parameter primarily a function of the rate of water extraction from the soil when ψ air-entry value.
Gardner (1958)	$\theta_w = \theta_r + \frac{\theta_s - \theta_r}{1 + \left(\frac{\psi}{\psi_a}\right)^b}$	(17)	b = a soil parameter which is primarily a function of the rate of water extraction from the soil, once the ψ air-entry value
Williams et al. (1983)	$\ln\Theta = A + B\ln\psi$	(18)	$\Theta = \frac{\theta - \theta_r}{\theta_s - \theta_r}$ A = fitting parameter B = fitting parameter
Farrel and Larson (1972)	$\psi = \psi_a \exp[\alpha(\theta_s - \theta_w)]$	(19)	α = empirical constant
Assouline et al. (1998)	$\theta_s = \theta_L + (\theta_s - \theta_L)\left[1 - \exp\left\{-\xi\left(\frac{1}{\psi} - \frac{1}{\psi_L}\right)^{\eta}\right\}\right]$	(20)	ψ = capillary head ψ_L = capillary head that corresponds to a very low water content, at which the hydraulic conductivity is negligible. θ_L = volumetric water content at capillary head ψ_L. η = fitting parameter. ξ = fitting parameter.

The soil-water characteristic curve is affected by the following factors: soil structure, texture, mineralogy, stress history and compaction method (Vanapalli et al., 1999). However, the three most influential factors are: (1) soil type. Soils containing more fines have a higher air-entry value, a higher residual volumetric water content and a slower tendency for the water content to change when suction increases; (2) void ratio and hence compactive effort. Increasing the compactive effort results in smaller pores, higher air-entry suction and a slower tendency for the water content to change when suction increases (Tinjum et al., 1997) and (3) initial molding water content. Compacted cohesive soils do not exist as a uniform mass of soil particles. Rather aggregation of particles form, separated by comparatively large air voids (Croney et al., 1958; Barden and Sides, 1970; and Tinjum et al., 1997). This is more pronounced dry of the optimum water content. The effect of aggregation is to lower the air-entry value of matric suction (Tinjum et al., 1997 and Vanapalli et al., 1999).

4.3.2 Suction Measurement: Equipment and Procedure

Both the pressure plate apparatus and the filter paper method were used in this research to measure matric suction. The pressure plate apparatus was used to define the soil-water characteristics at low values of matric suction (0 to 500 kPa). The filter paper method is used to extend the soil-water characteristic curve to higher suction values (> 500kPa). Tests were performed on soil samples at optimum and at 95% relative compaction based on modified and standard

compaction. The 95% relative compaction samples were prepared dry-of- and wet-of-optimum.

4.3.2.1 Pressure Plate Test

Two pressure plate apparatus (Fig. 24) manufactured by Soilmoisture Equipment Corp., each having a porous ceramic plate with an air-entry value of 500 kPa, were used for measuring the matric suction in accordance with ASTM Standard D2325-68. Pertinent points of the soil preparation and test procedures are highlighted below:

1. Soil specimens were prepared at the target dry unit weight and water content using static compaction. They were 66 mm in diameter and 20 mm high, housed within a steel ring for lateral support on the porous ceramic plate.
2. The soil samples were then soaked for 48 hours. A small vacuum was used to accelerate the saturation process. The soil was not prevented from swelling during saturation.
3. After expelling excess water on the ceramic plate, an initial volume reading was taken.
4. After sealing the lid, the air pressure in the cell was set to the desired value and the amount of water coming out from the specimen was recorded. The amount of water coming out from the specimens was adjusted for evaporation by simultaneously recording the change in volume of water in a control burette over time.

5. Step 4 was repeated for a range of desired matric suction values, which were typically 10 kPa, 20 kPa, 50 kPa, 100 kPa, 200 kPa, and 400 kPa.
6. The final water content and weight of solids of the specimen was then determined.
7. The water content at each value of matric suction was then back calculated to derive the initial portion of the SWCC.

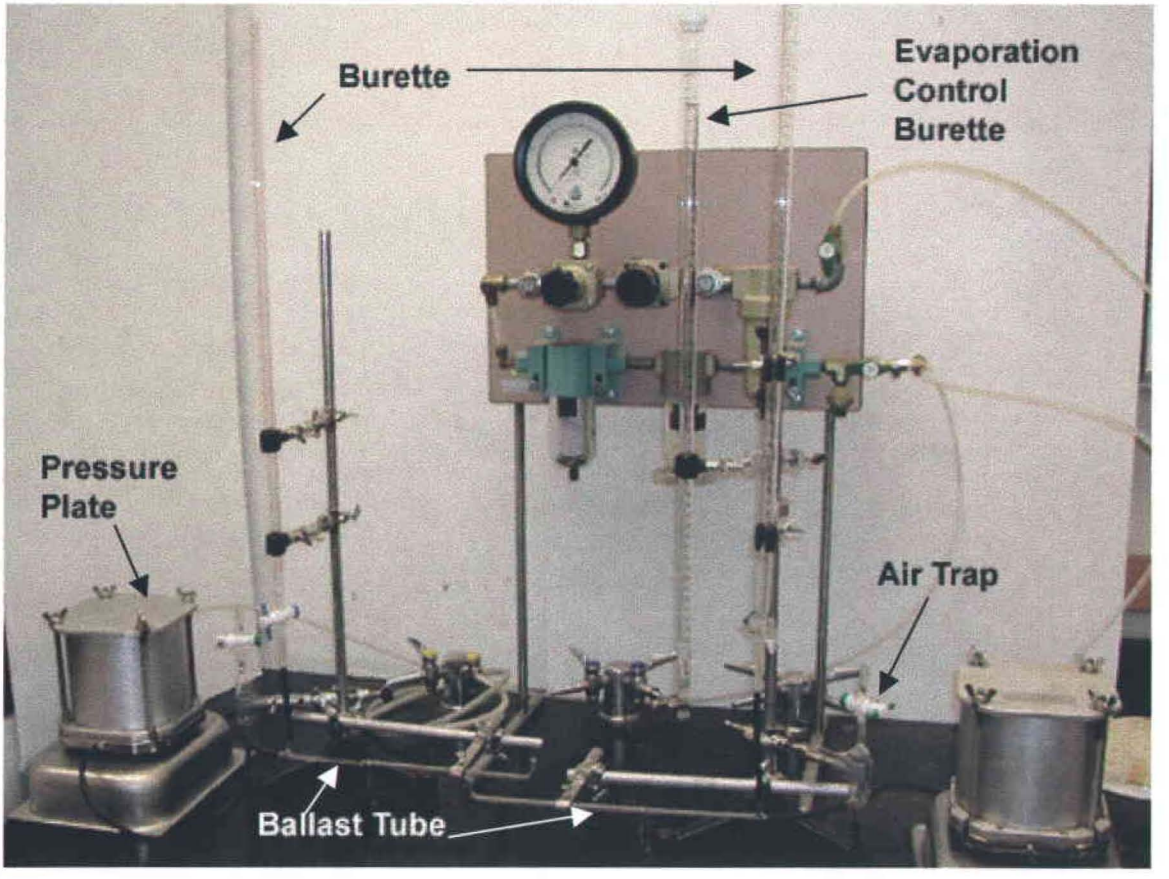


Figure 24. Pressure Plate Apparatus

4.3.2.2 Filter Paper Method

The filter paper method was performed in accordance with ASTM Standard: D5298-94. Highlights of the soil preparation and test procedures are as follows:

1. Two soil specimens were prepared in the same steel ring as used for the pressure plate test. The specimens were then extruded from the ring.
2. Three sheets of Whatman No. 42 filter paper were used for each determination. A 50-mm diameter filter paper was placed in between two 60-mm diameter sheets to avoid soil contact. The set of filter papers were then placed in between the two soil specimens.
3. The soil specimens with filter paper were then placed in an air-tight metal container for 7 days and stored at room temperature for equilibration of moisture. The container has an inside diameter of 68mm and a height of 42mm. The tightness of fit (small clearances of the sample in the can) is necessary to minimize the air inside the container.
4. Upon equilibration, the soil specimens plus filter paper were removed. The wet weight of the 50mm diameter filter paper was measured (using a Scientech SA80 scale with an accuracy of ± 0.0001 g) within a few seconds to avoid loss of moisture. The moist weights of the soil specimens were also measured.
5. The soil samples and the 50-mm diameter filter paper were then oven-dried at 105° C. During oven-drying, the filter paper was placed in a separate clean moisture can. The dry weight of the filter paper was

determined after 4-6 hours, whereas the weight of the soil solids was determined after 24 hours.

6. Using the calibration curve by Swarbrick (1992) in Fig. 25, the matric suction was then estimated based on the water content of the filter paper.
7. The water content of the soil specimens corresponding to the matric suction from step (6) were then determined.
8. Steps 1 through 7 were repeated for a range of water contents to obtain the portion of the SWCC at high matric suction values in excess of 500 kPa.

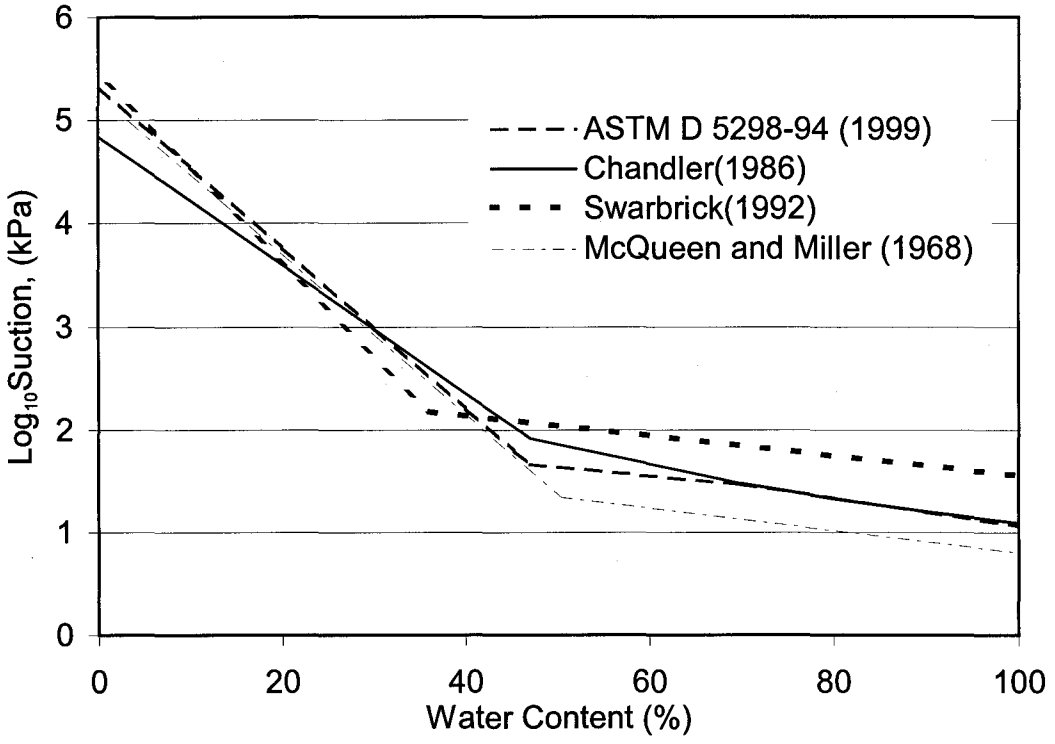


Figure 25. Filter Paper Calibration Curves

4.3.2.3 Test Results

Only the matric suction for the Waipio soil was determined in this study. Determining the matric suction for the soils from Kapolei, Mililani Mauka and Wahiawa are beyond the scope of this work. The matric suction contours for the Waipio soil are plotted in Fig. 26.

Using a least-squares algorithm, both the van Genuchten (1980) and Fredlund & Xing (1994) models were used to fit the matric suction versus water content data to derive the SWCC equations. The SWCC are shown in Fig. 27 and 28 for the van Genuchten and Fredlund and Xing models, respectively. The fitted parameters for these models are tabulated in Table 10, along with the coefficients of determination (R^2).

Table 10. Summary of SWCC Parameters for Waipio Soil

Compaction Water Content (%)	Dry Unit Weight (kN/m^3)	van Genuchten model (1980)				Fredlund and Xing model (1994)				
		α (kPa^{-1})	n	θ_r	R^2	a (kPa)	n	m	ψ_r (kPa)	R^2
23 (dry)	14.0 (SP)*	0.1222	1.1506	0.0201	0.9891	79.16	0.4845	0.9671	248271	0.9939
27.4 (opt)	14.6 (SP)*	0.0637	1.1322	0.0586	0.9956	359.4	0.4795	1.1542	217120	0.9976
31.2 (wet)	14.0 (SP)*	0.2784	1.1008	0.0227	0.9994	485.9	0.3765	1.7250	122845	0.9995
20.95 (dry)	15.1 (MP)*	0.0097	1.3101	0.0009	0.9949	171.6	0.8955	0.8734	6405	0.9999
23.2 (opt)	16.0 (MP)*	0.0015	1.4154	0.0000	0.9980	955.9	1.0820	0.9721	16770	0.9999
26.4 (wet)	15.0 (MP)*	0.0050	1.3720	0.0071	0.9970	487.3	0.8775	1.1396	13305	0.9990

*SP and MP = standard and modified Proctor compaction effort, respectively

Based on the soil-water characteristic curves published by Tinjum et al. (1999) and Vanapali et al. (1999), the specimens compacted wet of optimum had the highest air-entry value, followed by the soil at optimum and the soil dry of optimum, which is somewhat consistent with observations for the Waipio soil.

Several researchers have proposed universal models for the soil-water characteristics. One such model is discussed in detail herein. Using test data on four compacted soils, Tinjum et al. (1999) proposed a universal model based on van Genuchten's (1980) equation. The advantage of using the van Genuchten's equation is that only two parameters (α and n) are needed to fully define a soil-water characteristic curve if θ_r is 0. They related α and n to several soil and compaction parameters as follows:

$$\log(\alpha p_a) = -1.127 - 0.017PI - 0.092(w - w_{opt}) - 0.263C + \log(p_a) \quad (21)$$

$$n = 1.06 + 0.002PI - 0.005(w - w_{opt}) \quad (22)$$

where p_a = atmospheric pressure, PI = plasticity index, w = gravimetric water content, w_{opt} = gravimetric optimum water content and C = a categorical variable for compactive effort = -1 for standard Proctor and +1 for modified Proctor. Tinjum et al. (1999) plotted the van Genuchten SWCC parameters versus plasticity index as shown in Fig. 29. Superimposed on this plot are the calculated van Genuchten parameters for the Waipio soil.

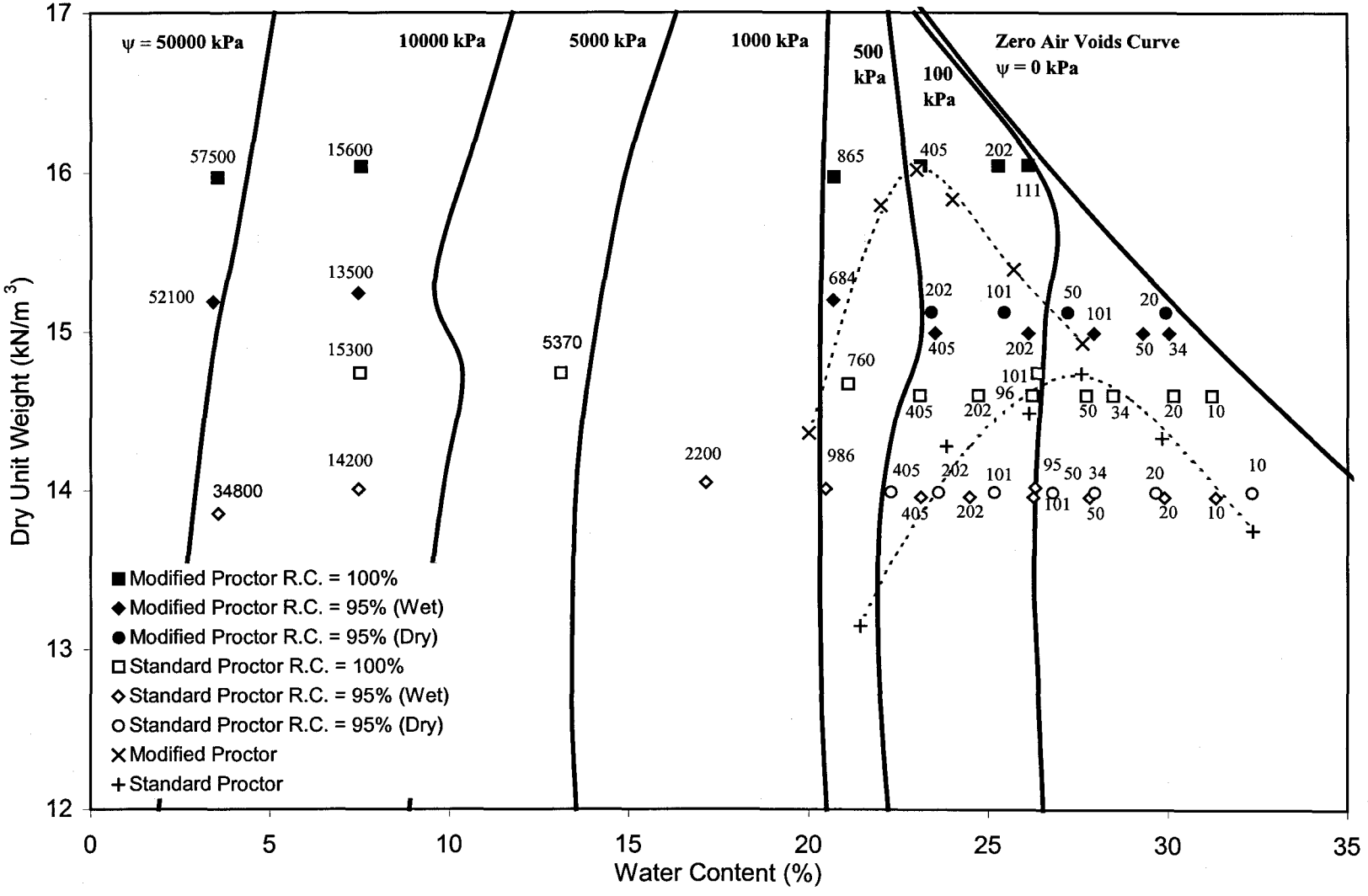


Figure 26. Matric Suction Contour for Waipio Soil

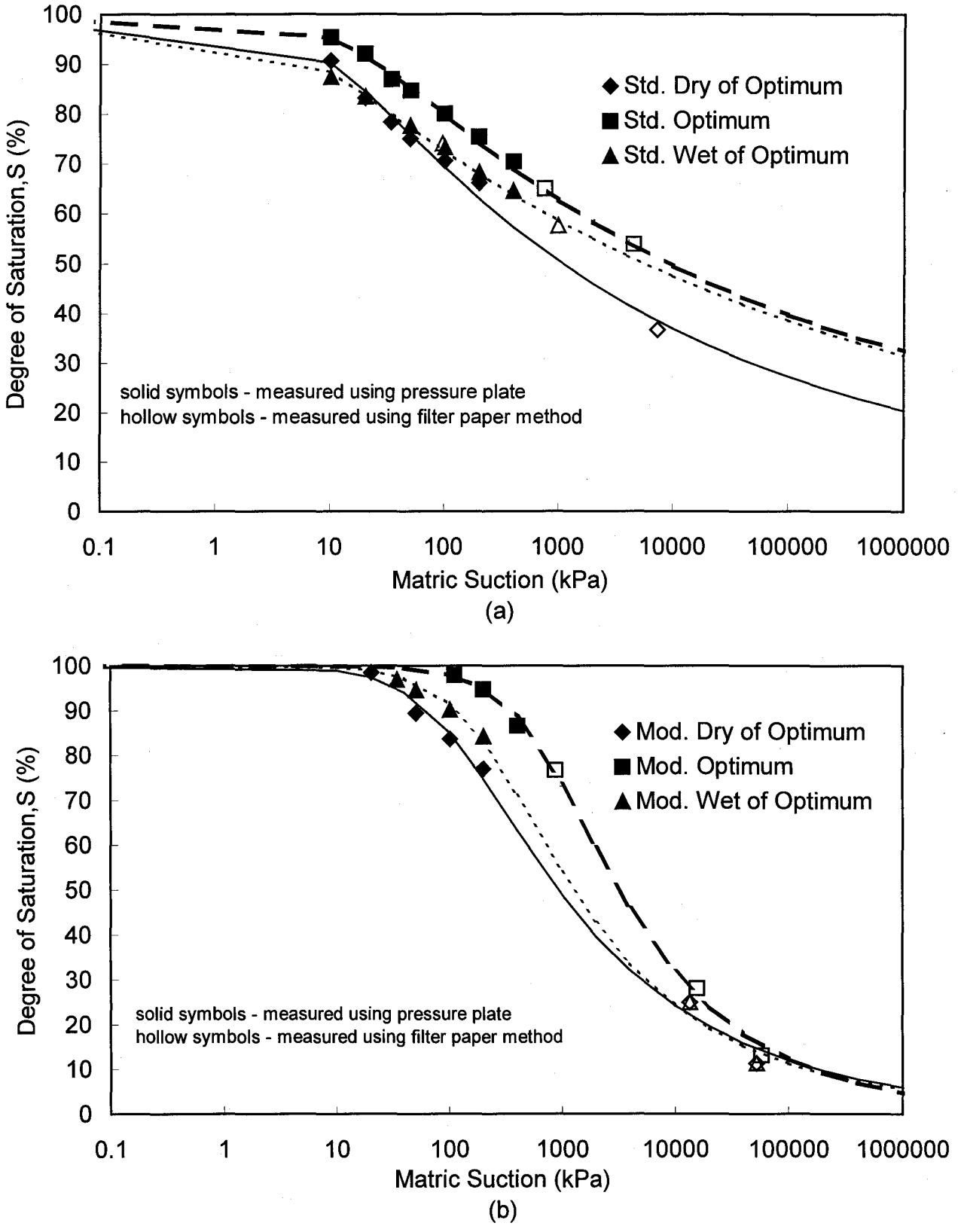


Figure 27. SWCC using van Genuchten's Model (1980) (a) Standard Proctor Compaction Effort (b) Modified Proctor Compaction Effort

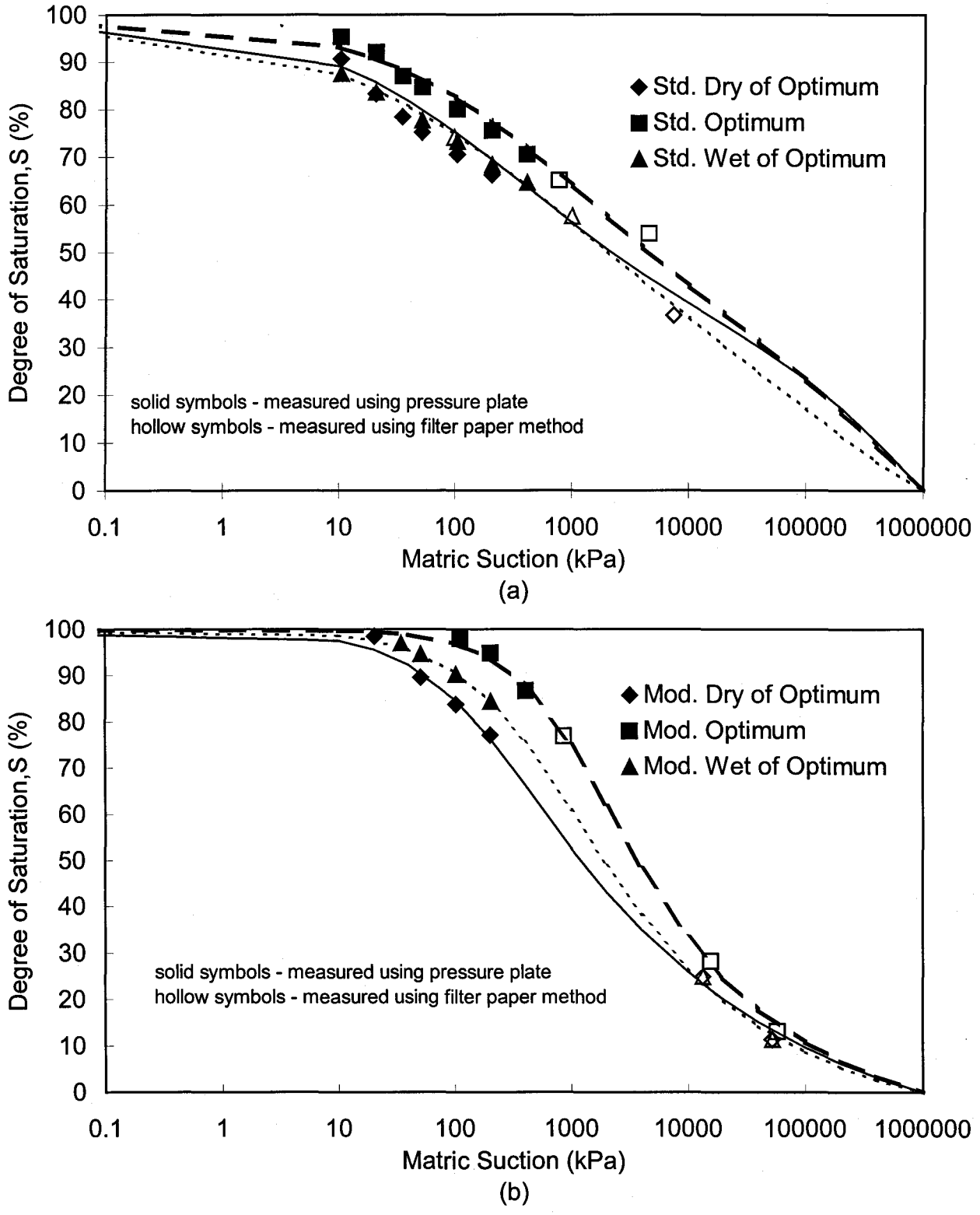


Figure 28. SWCC using Fredlund and Xing's Model (1994) (a) Standard Proctor Compaction Effort (b) Modified Proctor Compaction Effort

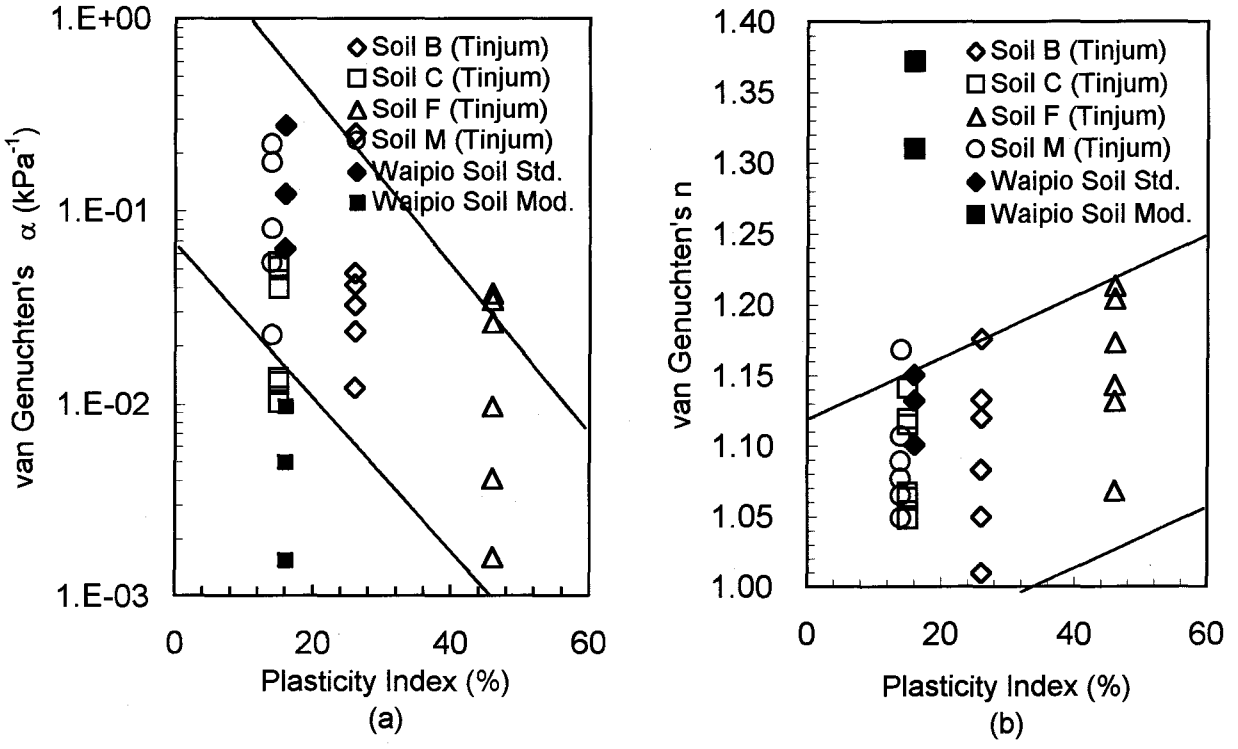


Figure 29. Plasticity Index versus: (a) van Genuchten's α; (b) van Genuchten's n

As illustrated in Fig. 29, an increase in the plasticity index will result in a decrease in α and an increase in n. For the Waipio soil, the van Genuchten parameters for the standard Proctor samples fall within the trend lines while those for the modified Proctor samples do not. This may be due to excessive swell during soaking for the modified Proctor samples. Because of swell in the samples and because of erroneous SWCC parameters for the Waipio soil, a comparison of the in situ dry unit weight with the predicted values from the GeoGauge™ stiffness is not feasible.

Chapter 5 Effects of Drying on Soil Properties and GeoGauge™ Stiffness

A series of index tests including grain size distribution, Atterberg limits, specific gravity, sand equivalent and compaction tests were also performed on the soil samples after oven drying to see if they undergo irreversible changes. These were then compared to the values for the in situ soil to study the change in properties as a result of oven drying.

5.1 Atterberg Limits

Liquid and plastic limits were determined in accordance with ASTM Standard D4318-98 for the soil samples after oven drying and the test results are summarized in Table 11.

Table 11. Atterberg Limits for Oven Dry Soil Samples

Location	Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Waipio	1	43	31	12
Kapolei	4	36	25	11
Mililani Mauka	C	58	35	23
	7	58	37	20
	10	67	42	25
	Average	61	38	22
Wahiawa	40	63	45	17
	46	65	43	22
	31	71	48	24
	40B	55	42	12
	50	60	44	17
	58	69	44	25
	Average	63	44	19

The plasticity index and liquid limits are plotted in Fig. 30 along with the results for the in situ soil for comparison. In general, the Atterberg limits decrease after oven drying. The decrease is more significant for the Mililani Mauka and Wahiawa soils or the high plasticity silts.

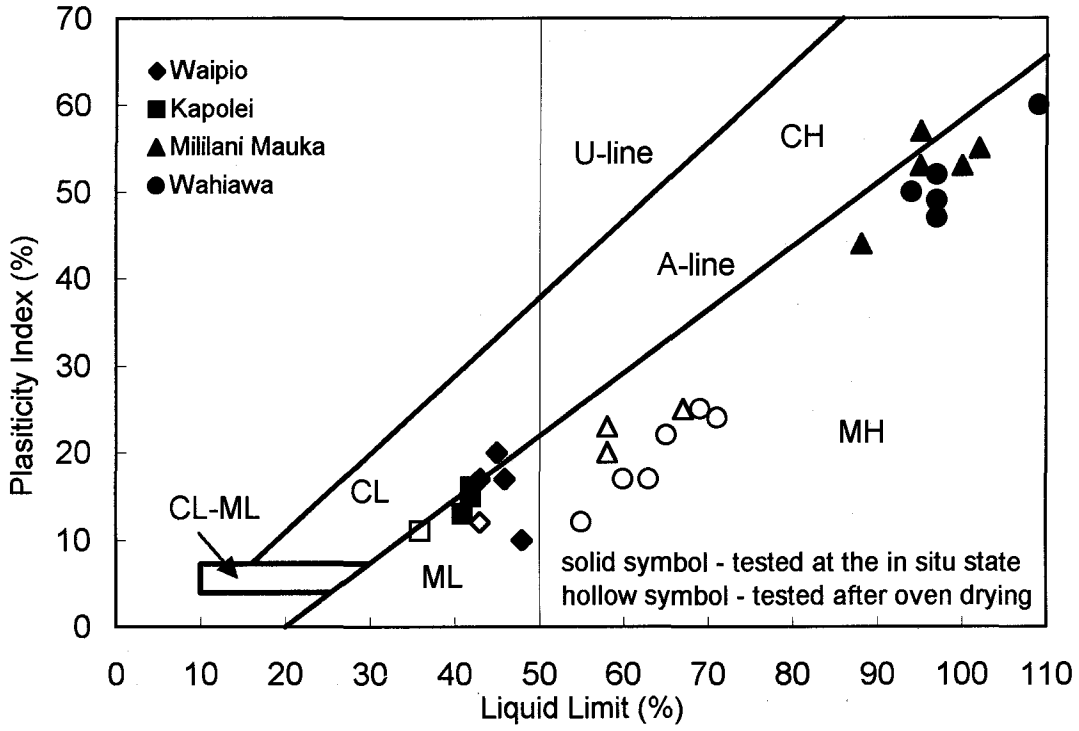
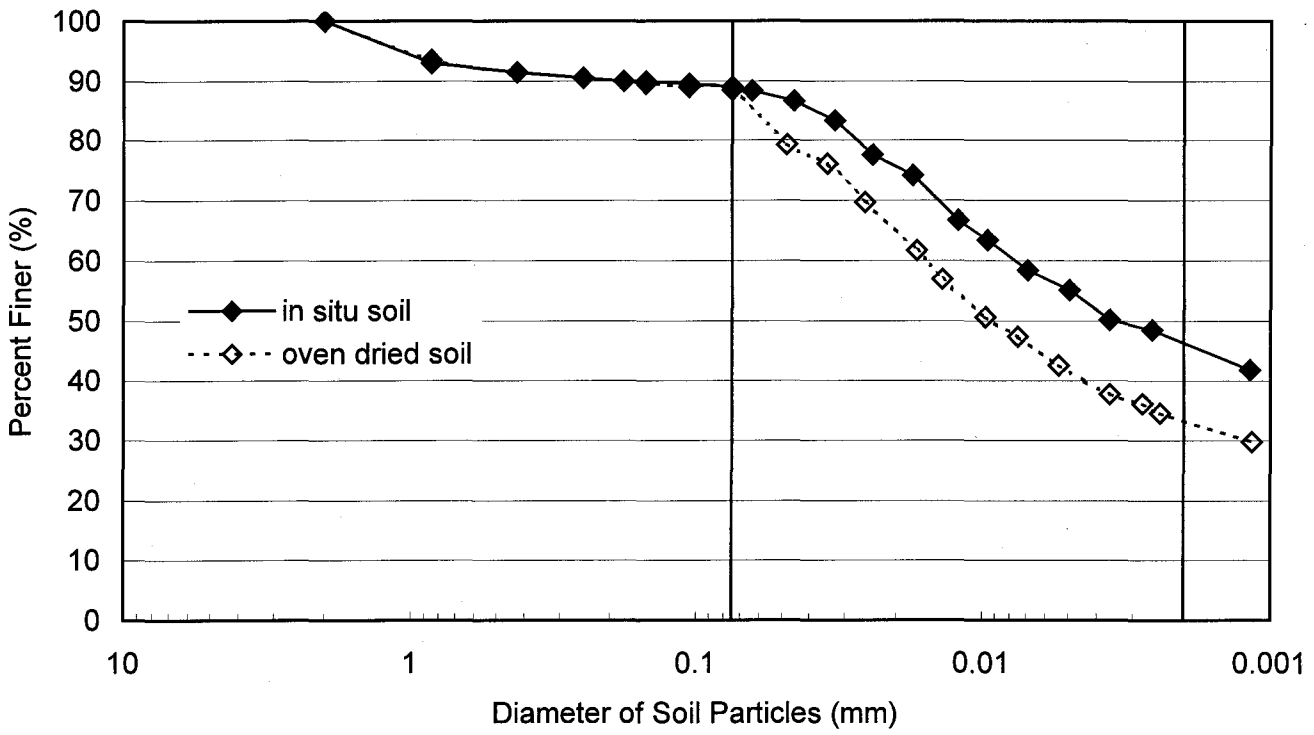


Figure 30. Plasticity Chart for In Situ and Oven Dry Soil Samples

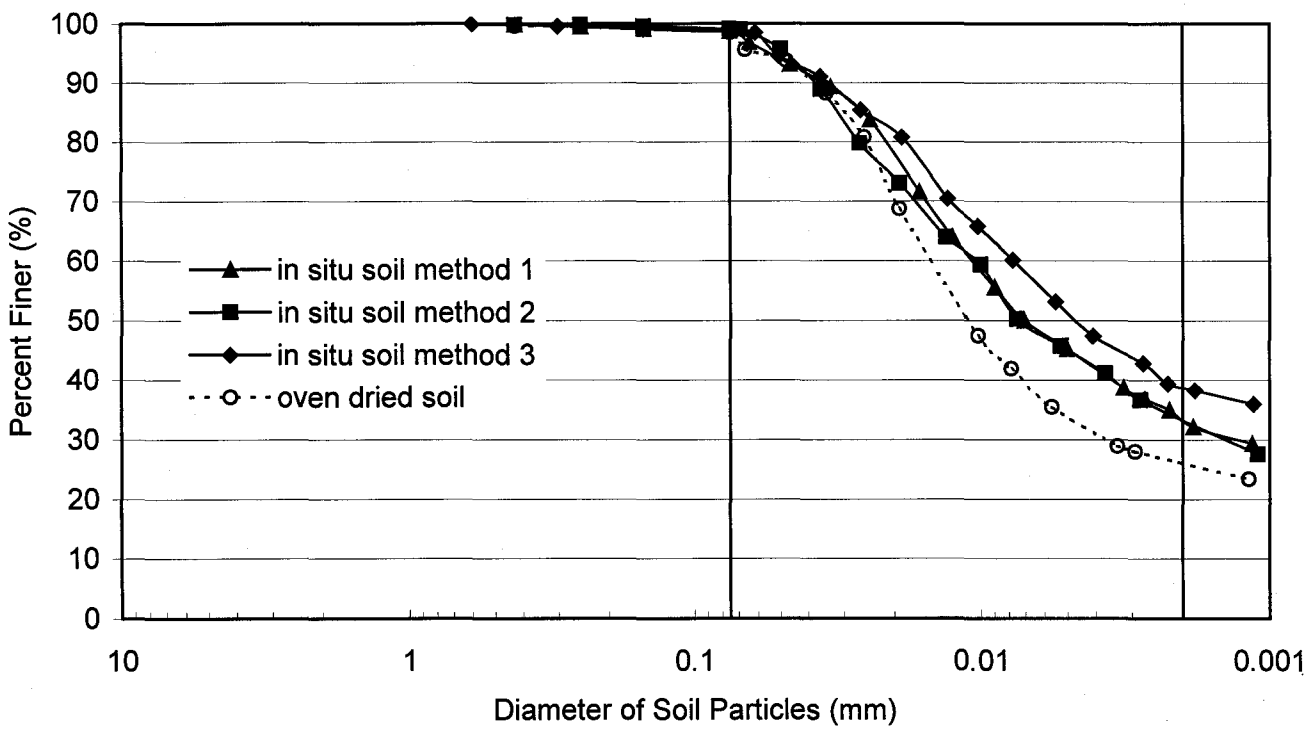
5.2 Grain Size Distribution

Grain size distributions were obtained for the oven dry soils in accordance with ASTM Standard D422-63 and plotted in Fig. 31 as dashed lines with hollow symbols. Superimposed on this plot are the grain size distributions for the in situ soil from Fig. 4 for comparison. From Fig 31, the sand, silt and clay fractions

were determined and summarized in Table 12. In general, oven drying resulted in an increase in silt fraction and a decrease in clay fraction.

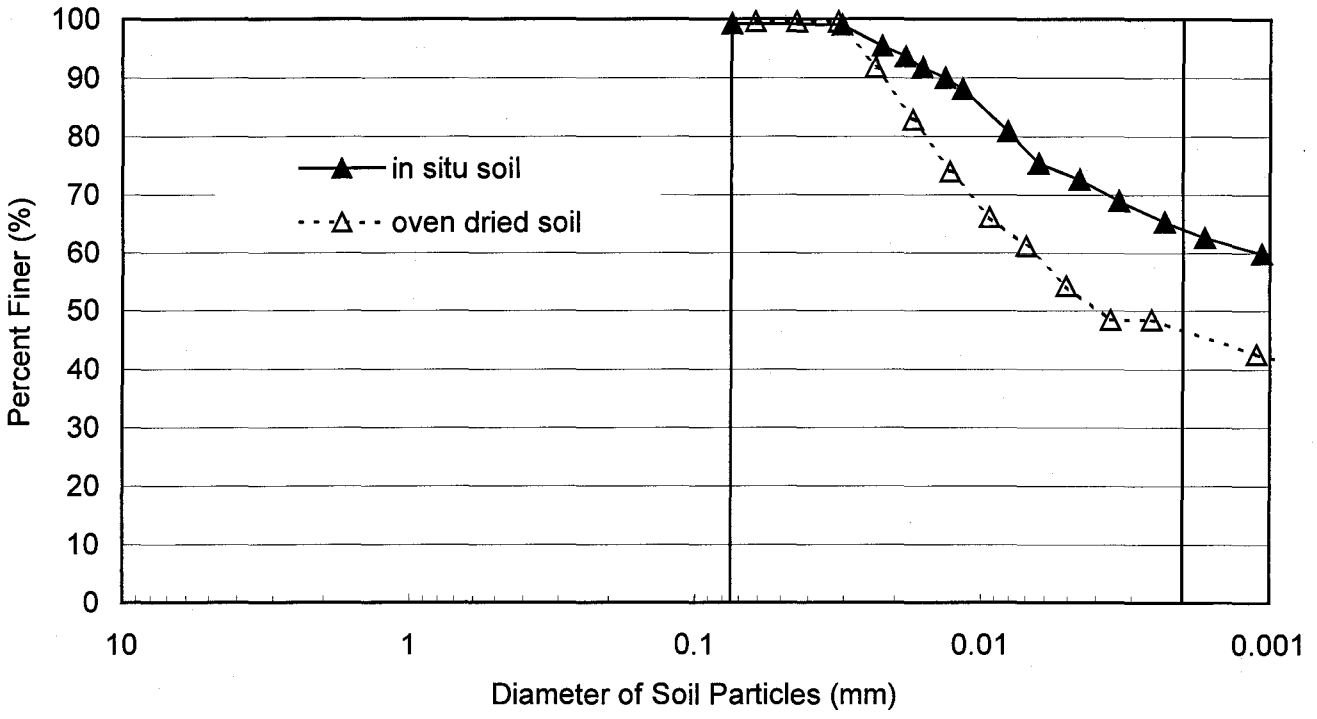


(a)

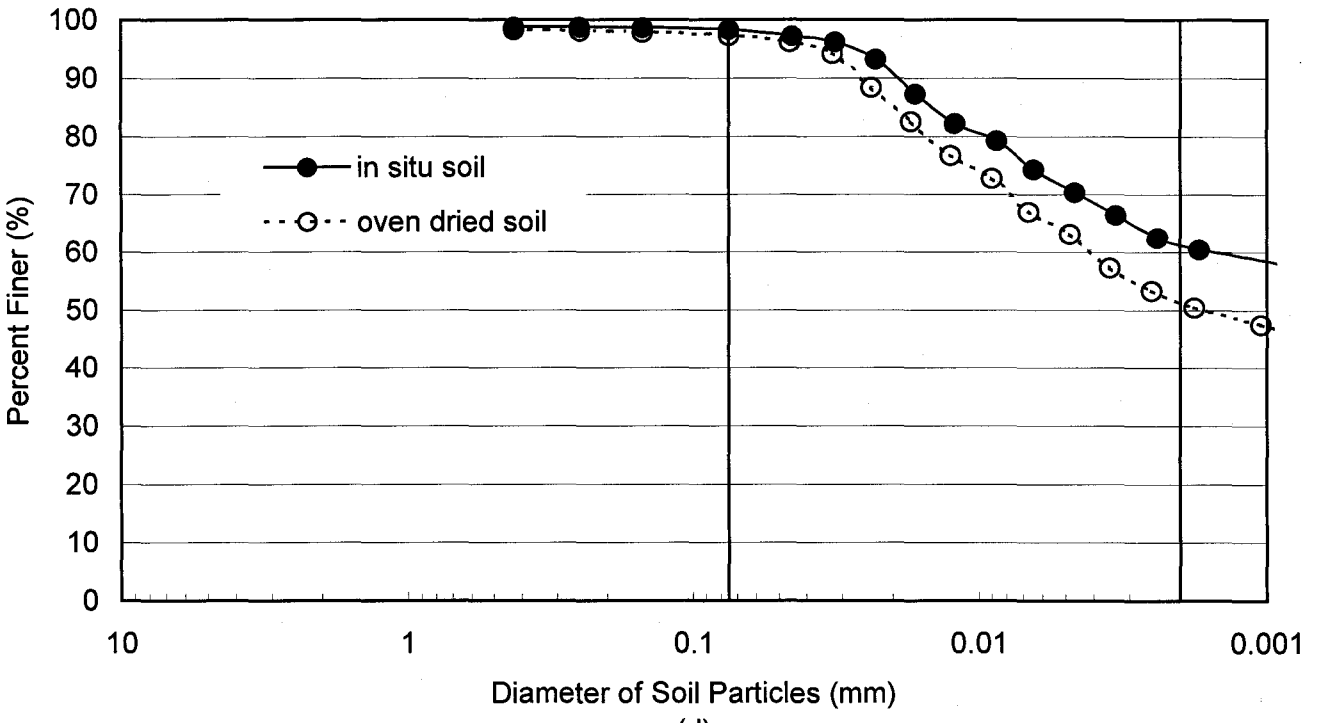


(b)

Figure 31. Grain Size Distributions for Soil Samples from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa



(c)



(d)

Figure 31. (Continued) Grain Size Distributions for Soil Samples from (a) Waipio (b) Kapolei (c) Mililani Mauka and (d) Wahiawa

Table 12. Soil Classification and Breakdown of Soil Type for both the In Situ and Oven Dry Samples

Location	Waipio		Kapolei		Mililani Mauka		Wahiawa	
	In Situ	Oven-dried	In Situ	Oven-dried	In Situ	Oven-dried	In Situ	Oven-dried
Sand Fraction (%)	12	12	1	1	1	1	1	1
Silt Fraction (%)	41	44	64	73	34	53	39	46
Clay Fraction (%)	47	34	35	26	65	46	60	53
Group Symbol	ML	ML	ML	ML	MH	MH	MH	MH
USCS Group Name	Silt	Silt	Silt	Silt	Elastic silt	Elastic silt	Elastic silt	Elastic silt

5.3 Specific Gravity

The specific gravity of the oven-dried soil samples were measured in accordance with ASTM Standard D854-98. Results are summarized in Table 13 along with those for the in situ samples for comparison.

5.4 Sand Equivalent

The sand equivalent tests were performed on oven-dried soil samples in accordance with AASHTO T176-97, the results of which are summarized in Table 14.

Table 13. Summary of Specific Gravity for the In Situ and Oven Dry Soil Samples

Location	Sample No.	Specific Gravity	
		In Situ	Oven Dry
Waipio	1	2.82	2.85
	2	2.99	2.95
	3	2.90	Not performed
	4	2.90	Not performed
	Average	2.90	2.9
Kapolei	4	2.91	3.06
	23	2.97	Not performed
	26	3.09	Not performed
	27	3.04	Not performed
	Average	3.00	3.06
Mililani Mauka	1	2.96	Not performed
	2	2.94	Not performed
	7	3.01	2.99
	10	3.01	Not performed
	Average	2.98	2.99
Wahiawa	25	2.99	Not performed
	25B	2.94	Not performed
	35	3.06	Not performed
	55	3.26	Not performed
	56	3.22	Not performed
	31	Not performed	3.09
	40	Not performed	3.25
	51	Not performed	2.94
	58	Not performed	3.39
	58B	Not performed	3.17
	40	Not performed	3.37
	46	Not performed	2.98
	Average	3.09	3.17

Table 14. Summary of Sand Equivalent for the In Situ and Oven Dry Soil Samples

Location	Sample No.	Sand Equivalent (%)	
		In Situ	Oven Dry
Waipio	1	8	Not performed
	5	11	Not performed
	18	13	Not performed
	22	Not performed	16
	Average	10	16
Kapolei	4	8	12
	23	8	Not performed
	26	7	Not performed
	27	10	Not performed
	Average	8	12
Mililani Mauka	1	11	Not performed
	2	9	7
	7	10	11
	10	16	Not performed
	9	Not performed	10
	Average	11	9
Wahiawa	56	14	Not performed
	35	14	Not performed
	55	13	Not performed
	56B	18	Not performed
	31	Not performed	21
	40	Not performed	20
	46	Not performed	19
	51	Not performed	21
	58	Not performed	19
	Average	14	20

In general, oven drying results in a slight increase in sand equivalent.

5.5 Compaction Test

Only the Wahiawa soil was dried down for compaction testing. The Wahiawa soil was tested at three different values of initial water content. First, the in situ soil was tested from wet to dry - (in situ). Second, the soil was oven dried and then tested from dry to wet - (oven dry). Third, the soil was dried to an intermediate

water content approximately equal to half the in situ natural water content (27%) - (intermediate). The compaction test results are plotted in Fig. 32.

In general, 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. In this case, the maximum dry unit weight for the "intermediate" soil is higher than the oven-dried soil.

5.6 GeoGauge™ Stiffness

GeoGauge™ stiffness measurements were performed on the in situ, intermediate and oven dry Wahiawa specimens immediately after compaction. The results are plotted in Fig. 33.

From Fig. 33, some stiffness values increase dry of the initial peak stiffness instead of decreasing as observed for the other soils. This may be explained as follows: as the soil dries, they undergo irreversible changes and tend to be less plastic and coarser. A less plastic and coarser soil possibly resulted in a higher stiffness, which may explain why the curves rise up dry of the initial peak stiffness.

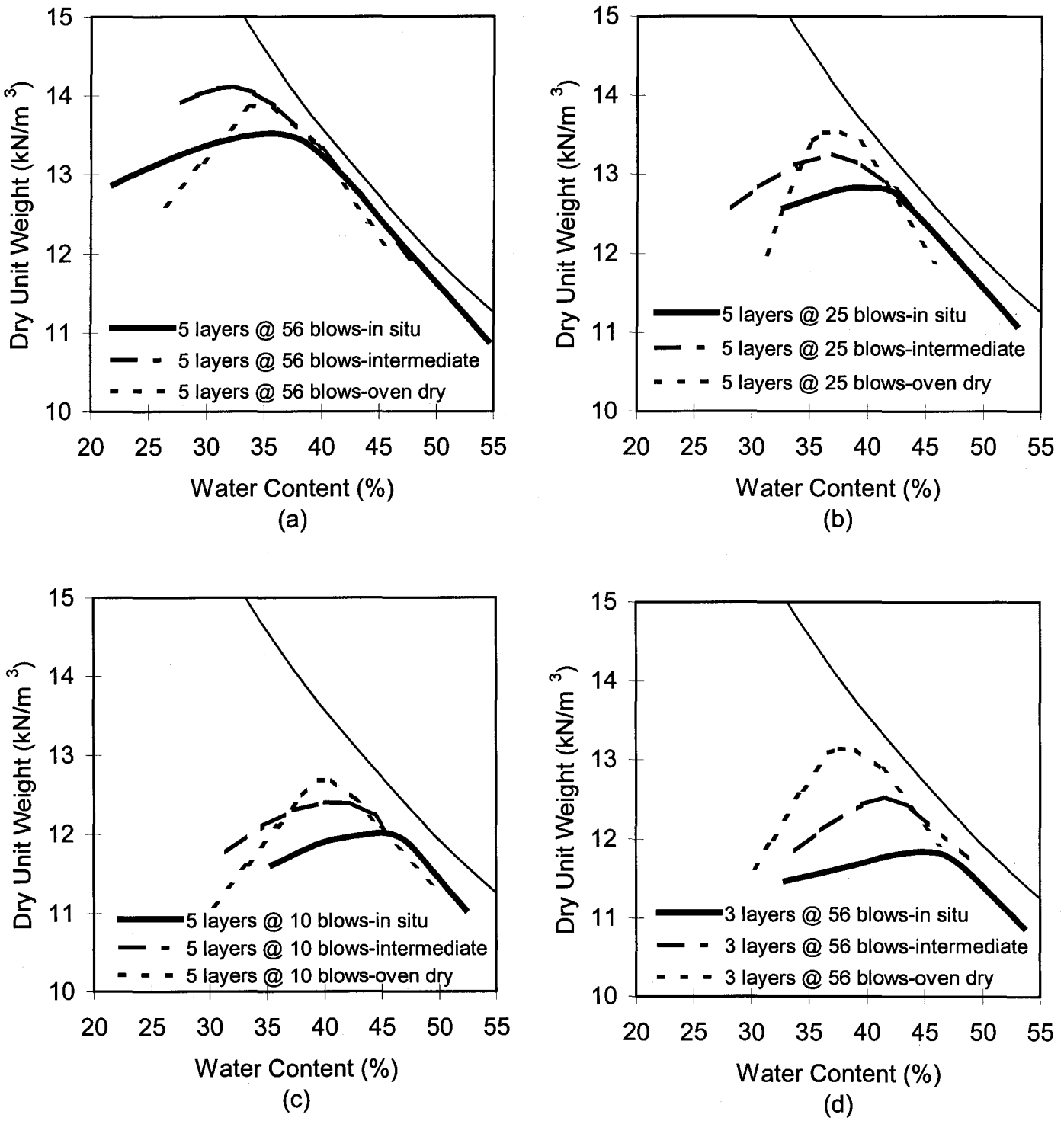


Figure 32. Compaction Curves for Wahiawa Soil Using Different Compaction Effort (a) 5 layers @ 56 blows (b) 5 layers @ 25 blows (c) 5 layers @ 10 blows and (d) 3 layers @ 56 blows

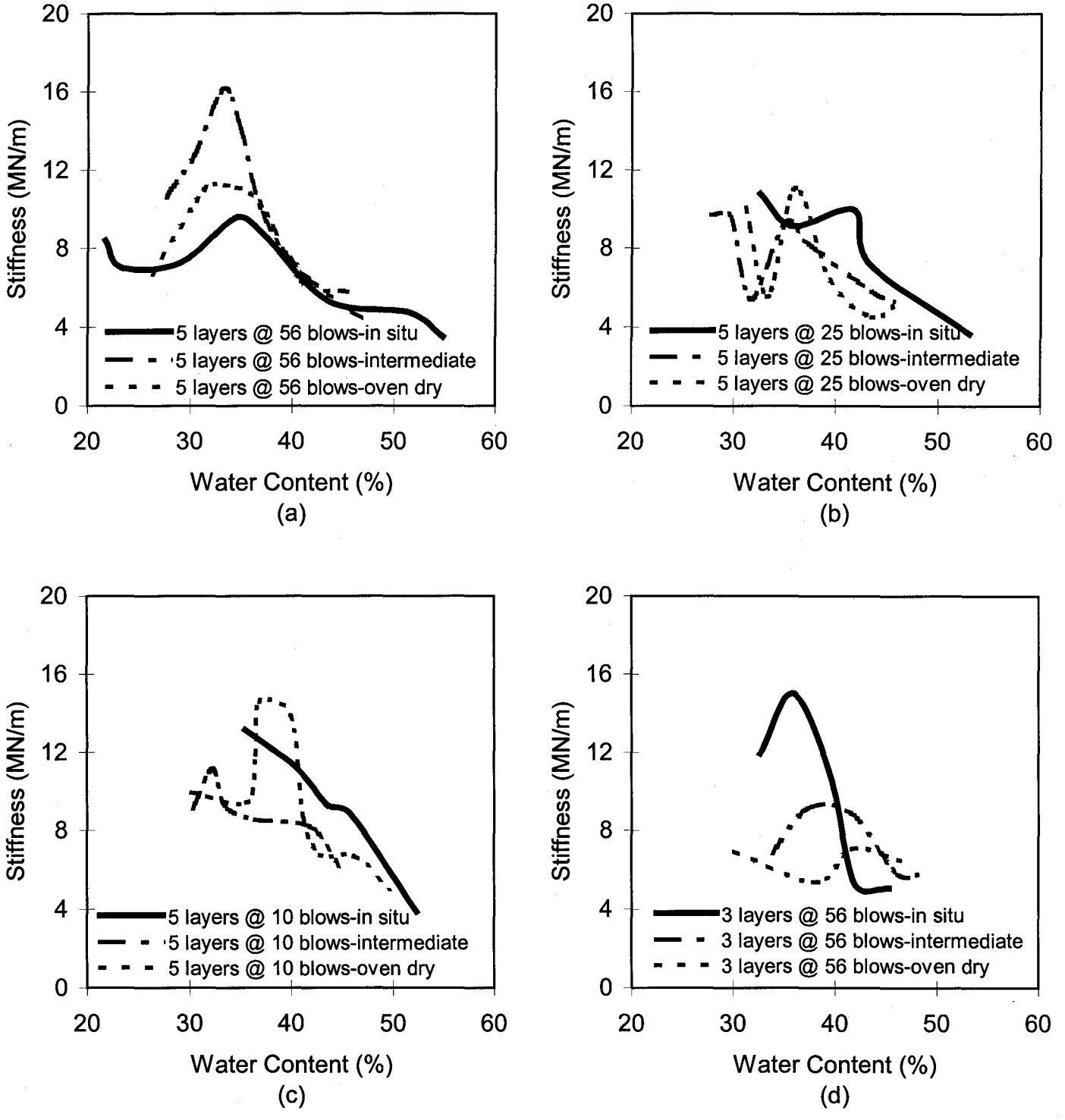


Figure 33. Results of GeoGauge™ Stiffness Testing Immediately after Compaction for the Wahiawa soil (a) 5 layers @ 56 blows (b) 5 layers @ 25 blows (c) 5 layers @ 10 blows and (d) 3 layers @ 56 blows

5.7 Summary

Changes in soil properties were observed upon drying the Waipio, Kapolei, Mililani Mauka and Wahiawa soils. These trends are summarized in Table 15.

Table 15. Summary of the Effects of Drying on Soil Properties

Soil Property	Change in Soil Properties upon Drying	Exception
Liquid limit (%)	Decrease	
Plastic limit (%)	Decrease	
Plasticity index (%)	Decrease	
Silt fraction (%)	Increase	
Clay fraction (%)	Decrease	
Specific gravity	No significant change	
Sand equivalent (%)	Increase	Mililani Mauka
Maximum dry unit weight for Wahiawa soil	Increase	5 layers @ 56 blows
Optimum water content for Wahiawa soil	Decrease	5 layers @ 56 blows
Peak stiffness for Wahiawa soil	No clear trend	

Several soils 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.

Many Hawaiian soils contain allophanes, halloysites and sesquioxides. Upon drying, these soils undergo irreversible changes, resulting in permanent

alterations in soil properties; i.e., the soil behaves differently even after re-wetting. The chemical changes in the soils tested are beyond the scope of this work.

In this research, 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 (i.e., during compaction testing), the soil was tested from wet to dry. Test results show that soil properties from those four locations change by varying degrees as a result of drying. In general, the soil becomes less plastic, coarser (downward shift in grain size curve and higher sand equivalent), and exhibits a higher maximum dry unit weight and a lower optimum water content.

Chapter 6 Summary and Conclusions

From this test program, the following conclusions can be summarized:

(a) Stiffness peaks dry of optimum.

(b) Stiffness increases with increasing dry unit weight at low water contents and decreases with increasing dry unit weight at high water contents.

(c) There is no direct relationship between stiffness and dry unit weight. A stiffness value can correspond to several values of dry unit weight depending on the water content. A relationship between stiffness, dry unit weight and water content was derived in this study. This relationship requires detailed information on the SWCC. However, further research is required to develop a generalized SWCC model for compacted soils.

(d) Stiffness values decrease upon wetting. The decrease in stiffness for soils dry of optimum is more significant than for soils wet of optimum.

(e) Soil specimens having large stiffness values tend to undergo more volumetric change upon wetting. Therefore, the soil shrink/swell potential is not optimized if stiffness is.

(f) The GeoGauge™ provides an alternative method for compaction control that uses stiffness instead of dry unit weight. However, more work is still required before stiffness can be fully used as a means of compaction control especially with regards to specifications. For example, in general a compacted soil with a high stiffness tends to have a large shear strength. However, from conclusion (e), soils with a large stiffness tend to swell

more upon wetting. These conflicting trends have to be reconciled before stiffness can be used in specifications for compaction jobs.

(g) Some tropical soils can undergo irreversible changes upon drying. The soils tested in this study become less plastic, coarser and exhibit a higher maximum dry unit weight and a lower optimum water content upon drying.

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