Use of Stiffness for Evaluating Compactness of Cohesive Pavement 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. Based on a series of low-strain GeoGauge™ stiffness measurements made under controlled laboratory conditions on compacted silts, the variation of stiffness with water content, dry unit weight, degree of saturation, volume change upon wetting, shear strength and soil plasticity is discussed. In general, the GeoGauge™ stiffness is not directly related to dry unit weight, peaks dry of optimum and decreases upon wetting. Soil specimens having a large stiffness also tend to be stronger but they also tend to swell more upon wetting implying that the shrink/swell potential is not optimized if stiffness is. These results help advance the understanding of the role of stiffness in assessing compactness of cohesive geomaterials.
1. INTRODUCTION

Low strain modulus of soils is routinely used in earthquake geotechnics. In pavement engineering, several researchers have used the low strain modulus to estimate resilient modulus (1, 2) for pavement design. More recently, there has been a push to use the low strain modulus as a means of assessing compactness of road embankments.

In situ methods for measuring the low strain soil modulus include:

(a) downhole and crosshole methods (3);
(b) seismic cone (4);
(c) suspension logging method (5);
(d) surface wave site investigation using a hammer as a source (6) or a steady state vibrator as a source (7);
(e) seismic refraction (8);
(f) p-wave ultrasonic testing (9);
(g) portable falling weight deflectometers (10); and
(h) soil stiffness device called GeoGauge™ (11).

Methods (d) through (h) require no drilling and can be used on surface soils. Therefore, they have good potential for use in assessing compactness of pavement geomaterials. This study was facilitated using the GeoGauge™.

Many factors influence soil moduli including stress and strain level, rate of loading, type of loading (monotonic versus cyclic, and initial loading versus unloading versus reloading), number of load cycles, stress path/stress-induced anisotropy, type of compaction, degree of compactness and moisture content. At very low strains, the soil behaves elastically and the mode of loading is inconsequential. Based on a series of tests on compacted soil samples prepared under controlled laboratory conditions, this paper will examine several factors that influence the low-strain soil stiffness. These factors include water content, dry unit weight, degree of saturation, volume change upon wetting, shear strength and soil plasticity.

2. EQUIPMENT AND TESTING

Produced by Humboldt Manufacturing Company, the GeoGauge™ is purported to measure the stiffness of the top 100 to 150 mm of the surficial 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. An internal harmonic oscillator imparts very small vertical displacements to the soil via the ring foot. The force and velocity time histories are measured at several frequencies from which, values of force and the corresponding displacement are obtained. The GeoGauge™ stiffness is calculated as the average force per unit displacement over the various test frequencies. Throughout this paper, the term “stiffness” refers to the GeoGauge™ stiffness.
Stiffness and CBR tests were performed on 116-mm-high specimens compacted in a 150-mm-diameter mold. The soil was compacted in 5 lifts using a 4.54 kg hammer in accordance with ASTM Standard D1883. A family of three compaction curves was obtained by varying the number of blows per lift from 10 to 25 to 56. During compaction, a 61-mm-high spacer was placed at the bottom of the mold. After compaction, the spacer was removed and the mold was inverted so that the bottom of the soil specimen is flush with the base of the mold, which was bolted to the floor. Stiffness measurements were made on the soil in the mold immediately after compaction. 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 re-measured. The surcharge load was then reapplied and CBR testing was performed.

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 (Figure 1). Use of the extension is non-standard, and the effects of its use on the stiffness were studied by performing duplicate GeoGauge™ tests in the field – one with and one without the extension. Based on 93 of these comparative measurements at several sites in Hawaii with cohesive soils, use of the extension results in a stiffness reduction of about 7% on average (Figure 2).

Test results for a soil from Waipio on the island of Oahu, Hawaii, is detailed herein. The soil was classified as ML [PI = 17%, LL = 45%, Gs = 2.90, clay fraction (% > 2 µm) = 52%] based on the Unified Soil Classification System.

3. TEST RESULTS

Immediately After Compaction

Compaction curves are plotted in Fig. 3a. Shown in Figs. 3b, 3c and 3d are the post-compaction stiffness varying with dry unit weight, water content and degree of saturation, respectively. The following observations are made:

1. For all three compactive efforts, the stiffness and dry unit weight do not peak simultaneously. In fact, the stiffness always peaked dry of optimum. This can be explained by one or both of the following:
   (a) modulus generally increases with (i) increasing effective stress and (ii) decreasing void ratio (I3-22). The vertical effective stress of partially saturated soils can be estimated using the following expression (23):
   \[ \sigma_v' = \sigma_v - u_a + \chi (u_a - u_w) \]  
   where \( \sigma_v \) = total stress, \( u_a \) = pore air pressure, \( u_w \) = pore water pressure, \( u_a - u_w = \psi \) = matric suction and \( \chi \) = effective stress parameter. \( \chi \) is zero for dry soils and unity for saturated soils. On the other hand, the matric suction can be extremely high at low water contents and decreases to zero for saturated soils. Effective stress is governed by the product \( \chi \psi \). Thus, there must exist some value of degree of saturation at which \( \chi \psi \) or the
effective stress contribution from suction and hence, stiffness is a maximum. Modulus also increases with decreasing void ratio. As the water content decreases from the optimum along the compaction curve, the void ratio increases but yet, the stiffness peaks. Therefore, the influence of increasing effective stress (due to increased suction), which has a tendency to increase modulus, far outweighs the effects of increasing void ratio, which has a tendency to reduce stiffness, resulting in the peak; and

(b) Cohesive soils dry of optimum tend to be flocculated. As the water content increases, the soil structure becomes more dispersed (24). 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 higher stiffnesses on the dry side.

2. Three distinct portions are apparent in Figs. 3b and 3c. 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. (11) who observed significant scatter in the same correlation.

3. Fig. 3a and 3c can be used to look at the change of stiffness with dry unit weight at constant water content. At a water content of say 26%, stiffness values decrease with increasing compactive effort (and hence dry unit weight). For the Waipio soil, stiffness degradation with unit weight is apparent at water contents in excess of 24%. At lower water contents (say 21.5%), the reverse occurs; i.e., stiffness increases with compactive effort when the blows increase from 10 to 56 although the increase is relatively small (about 20%). The effect of dry unit weight on stiffness can be clearly seen by interpolating Fig. 3a and 3c and reploting stiffness versus dry unit weight at constant moisture contents (Fig. 4). 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 stiffness (w = 23%, 24%, 25%) or a leveling off of stiffness (w = 22%). At water contents higher than 25% and dry unit weights above 14 kN/m³, 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.

4. All the stiffness values for the Waipio soil peaked within a narrow range of degree of saturation from 66% to 76%, all of which are less than 85%, which is the degree of saturation for the line of optimums. Wu et al. (25) 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. (26) 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. that an optimum degree of saturation for stiffness exists, and that it increases with decreasing effective grain size or
for stiffness exists, and that it increases with decreasing effective grain size or $D_{10}$ (Table 1). Test results for an MH soil from Mililani Mauka in Oahu are also shown in Table 1.

**After 4 Days of Soaking**

Values of water content and dry unit weight after 4 days of soaking are shown in Fig. 5a. Plotted in Figs. 5b, 5c and 5d are the stiffness values after soaking versus dry unit weight, water content and degree of saturation, respectively. Stiffness values before soaking are shown as dashed lines for comparison. The following observations are made:

1. After soaking, water contents of the soil dry of optimum increase more significantly than those wet of optimum (Fig. 5a). The degree of saturation increased to between 85% and 100% (Fig. 5d).
2. After soaking, the stiffness decreased to within a very narrow range of 4 and 9 MN/m. 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. 6). 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 more drastic loss in stiffness.

For compacted cohesive geomaterials, this implies that it is desirable to compact wet of optimum if stiffness decrease is to be minimized due to rain or other means of wetting.

**4. RELATIONSHIP BETWEEN LOW STRAIN STIFFNESS AND VOLUME CHANGE UPON WETTING**

Volume change below road pavements occurs upon loading as well as upon changes in water 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 in these soils tends to be higher when the soils are compacted dry of optimum (27, 28). Using swell measurements for the Waipio soil 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. 7a and 7b.

In Fig. 7a, volume change decreases as the water content increases from 22% and above. In Fig. 7b, the swell contours indicate that volume change decreases with increasing water content as well as decreasing stiffness. Also, the maximum volume change occurs close to the peak stiffness. Therefore, from a volume change standpoint, a high stiffness does not necessarily imply an ideal condition. 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 as discussed below.
5. RELATIONSHIP BETWEEN LOW STRAIN STIFFNESS AND SOIL STRENGTH

Seed et al. (24) 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. A plot of CBR, an index of shear strength (1), versus the post-soaking water content is shown in Fig. 8c. The CBR decreases with increasing moisture content. Comparing soils having the same CBR in Fig. 8b and 8c, soils compacted dry of optimum tend to have lower dry unit weights and higher water contents than those compacted wet of optimum.

The low strain stiffness after soaking is plotted versus CBR in Fig. 9. From this figure, a direct relationship does not exist between low strain stiffness and 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 stiffness is strain dependent, a correlation between the two parameters would require information on the rate of moduli degradation with strain.

From a strength perspective, a high “dry-of-optimum” stiffness is desirable. From a volumetric change standpoint, a lower stiffness wet of the maximum dry unit weight is desirable. Therefore, engineers have to carefully target an intermediate stiffness such that the strength of the geomaterial is adequate and the volume change is within acceptable limits.

6. EFFECTS OF PLASTICITY ON LOW STRAIN STIFFNESS

Test results for an MH soil (PI = 55%, LL = 98%, Gs = 2.98, clay fraction = 58%) sampled from Mililani Mauka on Oahu are shown in Fig. 10. While the observed trends are similar to the Waipio soil, the following differences were observed:

(a) the maximum dry unit weights for the MH soil were approximately 2 kN/m³ lower;
(b) the optimum water contents for the MH soil are approximately 10% higher;
(c) the maximum swell observed in the MH soil was 7% (1% for Waipio silt);
(d) the maximum CBR corresponding to the optimum at 56 blows per layer was 21% for the MH (25.5% for Waipio silt);
(e) the peak stiffness values were not significantly different for the two soils.

Based on this comparison, it cannot be concluded at this time that plasticity affects the peak GeoGauge™ stiffness values.
7. EFFECTS OF THE MOLD ON THE GEOGAUGE™ STIFFNESS

Based on the work of Egorov (29) 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:

$$ K = \frac{F}{\delta} = \frac{ER_o}{\omega[1 - \nu^2]} $$

where $F = \text{force on the ring}$, $\delta = \text{ring displacement}$, $\omega = \text{constant that is a function of the ratio } R_i/R_o$, $R_i, R_o = \text{inside and outside radius of the ring, respectively}$, and $\nu = \text{Poisson’s ratio of the soil}$. In this study, 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. For a soil in the mold having the same $E$ and $\nu$ as the free-field, the GeoGauge™ stiffness of a soil in the mold is related to the value in the free-field as follows:

$$ \frac{K_{\text{mold}}}{K_{\text{ff}}} = \frac{\omega_{\text{ff}}}{\omega_{\text{mold}}} $$

where $K_{\text{mold}}$ and $K_{\text{ff}}$ are values of stiffness in the mold and the free-field, respectively and $\omega_{\text{mold}}$ and $\omega_{\text{ff}}$ are the constants corresponding to the ring footing in the mold and the free-field, respectively. Based on the dimensions of the GeoGauge™ ring foot, $\omega_{\text{ff}}$ is 0.5644 (30). An axisymmetric 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. Values of $\omega_{\text{mold}}$ were then evaluated for a range of $E$ and $\nu$. 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. 11). This correction applies to all values of soil Young’s modulus.

The effects of dynamic loading of the GeoGauge™ are not included in the finite element analysis described above. In fact, the instrument may have generated waves that can reflect from the wall and base of the mold. These reflections may affect 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. The dynamic force 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. Therefore, a worthwhile future study is to investigate the boundary effects of the mold by performing low strain modulus measurements using other devices, such as bender element testing, to verify the trends observed with the GeoGauge™.
8. SUMMARY AND CONCLUSIONS

When deriving a family of compaction curves for CBR testing, GeoGauge™ stiffness measurements were also made immediately after compaction and after 4 days of soaking. From this test program, the following conclusions can be summarized:

(a) A peak GeoGauge™ stiffness is observed for each compaction effort. These peak stiffness values occur dry of optimum.
(b) Stiffness increases with increasing dry unit weight at low water contents up to a point beyond which, it decreases with increasing dry unit weight at higher 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.
(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 adopted in specifications for compaction jobs.

Several fundamental factors that influence low strain stiffness have been discussed. Understanding these influences will help advance the use of low strain stiffness in evaluation of compactness of pavement geomaterials.

ACKNOWLEDGEMENTS

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REFERENCES


TABLE 1 Optimum Degree of Saturation for Several Soils

<table>
<thead>
<tr>
<th>Soil</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Clay fraction or Effective Size, D$_{10}$ (mm)</th>
<th>Opt. Degree of Saturation (%)</th>
<th>Reference</th>
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<tbody>
<tr>
<td>London Clay (MH)</td>
<td>88</td>
<td>25</td>
<td>Clay fraction (% &gt; 2µm) = 62%</td>
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<tr>
<td>Mililani Mauka Silt (MH)</td>
<td>98</td>
<td>55</td>
<td>Clay fraction (% &gt; 2µm) = 58%</td>
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<td>This study</td>
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<td></td>
<td>56 blows/layer</td>
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<td></td>
<td>89</td>
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<tr>
<td></td>
<td>25 blows/layer</td>
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<td></td>
<td>72</td>
<td></td>
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<td></td>
<td>10 blows/layer</td>
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<tr>
<td>Waipio Silt (ML)</td>
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<td>17</td>
<td>Clay fraction (% &gt; 2µm) = 52%</td>
<td>76</td>
<td>This study</td>
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<td>56 blows/layer</td>
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<td></td>
<td>66</td>
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<tr>
<td></td>
<td>25 blows/layer</td>
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<td></td>
<td>71</td>
<td></td>
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<td></td>
<td>10 blows/layer</td>
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<tr>
<td>Glacier Way Silt</td>
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<td>-</td>
<td>D$_{10}$ = 0.0024</td>
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<td>Glacier Way Sand</td>
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<td>D$_{10}$ = 0.03</td>
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<td>Brazil Sand</td>
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<td>-</td>
<td>D$_{10}$ = 0.17</td>
<td>5</td>
<td>(25)</td>
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- Data not applicable
FIGURE 1 GeoGauge™ and the ring foot extension (a) Original ring foot (b) Ring foot with extension (c) Close-up of ring foot extension (d) GeoGauge™ in mold using ring foot extension
FIGURE 2 Comparison of the GeoGauge\textsuperscript{TM} stiffness with and without the ring foot extension.

$K_{\text{with extension}} = 0.9348 K_{\text{without extension}}$

Coefficient of determination ($R^2$) = 0.9304
Figure 3 Results of stiffness testing immediately after compaction for Waipio silt (a) Compaction curves (b) Dry unit weight vs. stiffness (c) Stiffness vs. water content and (d) Stiffness vs. degree of saturation.
FIGURE 4 Stiffness versus dry unit weight at constant water content.
FIGURE 5 Results of stiffness testing after 4 days of soaking for Waipio silt (a) Compaction curves (b) Dry unit weight vs. stiffness (c) Stiffness vs. water content and (d) Stiffness vs. degree of saturation.
FIGURE 6 Normalized stiffness versus water content before and after soaking.
FIGURE 7 Swell contour lines in (a) compaction curves and (b) stiffness versus water content plot.
FIGURE 8 CBR test results for Waipio silt (a) Compaction curves (b) Dry unit weight after soaking versus CBR (c) CBR versus water content after soaking and (d) CBR versus degree of saturation after soaking.
FIGURE 9 Variation of stiffness with CBR after soaking.
FIGURE 10 Results of stiffness testing immediately after compaction for Mililani Mauka silt (a) Compaction curves (b) Dry unit weight vs stiffness (c) Stiffness vs water content and (d) Stiffness vs degree of saturation.
FIGURE 11 Correction factor for stiffness in the mold based on linear elastic finite element analysis of a ring footing applying a static load.
LIST OF FIGURES

TABLE 1 Optimum Degree of Saturation for Several Soils

LIST OF TABLES

FIGURE 1 GeoGauge™ and the ring foot extension (a) Original ring foot (b) Ring foot with extension (c) Close-up of ring foot extension (d) GeoGauge™ in mold using ring foot extension
FIGURE 2 Comparison of the GeoGauge™ stiffness with and without the ring foot extension.
FIGURE 3 Results of stiffness testing immediately after compaction for Waipio silt (a) Compaction curves (b) Dry unit weight vs. stiffness (c) Stiffness vs. water content and (d) Stiffness vs. degree of saturation.
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