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**KEYWORDS:** Hveem stabilometer, soil testing, asphalt, elastic modulus, highway materials

**Introduction**

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It is common for practitioners to discuss the R-value or the $S$-value without considering the pressure that is applied to the specimen during the preparation stage. The latter has an effect on the test results of “memory-capable” or viscoelastic materials like cohesive soils and asphaltic mixtures.

Stage I. Sample Preparation—Compacted soil samples are prepared in a 10.16 cm (4 in.) inner diameter cylindrical container at different moisture contents (and hence different degrees of saturation). A vertical pressure is then applied to the soil as water is allowed to ooze or exude from the specimen. When it is determined that the specimen is fully saturated (by sensors located below the top loading plate), the pressure is read. This is called the exudation pressure. The exudation pressure needed to fully saturate the soil, of course, depends on the moisture content at which the specimen was prepared. The exudation pressure required for a soil that is prepared at a lower moisture content would be much greater than for a soil that is prepared very wet. This exudation pressure affects the R-value in cohesive soils but has little effect on the R-value of cohesionless soil. As was mentioned earlier, the former is viscoelastic (and has “memory”) and the latter is not. The procedure limits the exudation pressure to 5512 kPa (800 psi) and below.

Asphalt mixtures are prepared in a different manner. They are compacted in a 10.16 cm (4 in.) inner diameter mold at an equivalent compaction pressure of 2412–3447 kPa (350–500 psi). Since asphalt concrete is viscoelastic, this compaction pressure affects the test value.

Stage II. Obtaining the Horizontal Pressure, $P_h$—The prepared soil specimen is placed in the stabilometer (Fig. 1). The test procedure follows the steps listed below:

(a) The top platen is held and a horizontal pressure of 34.5 kPa (5 psi) is applied.
(b) The top platen is then lowered until a vertical pressure of 1,103 kPa (160 psi) is achieved.
(c) The horizontal pressure, $P_h$, is then measured without allowing any horizontal strain—this is done with a constant fluid volume.

For asphalt mixtures, a vertical pressure of 2758 kPa (400 psi) is applied instead of 1103 kPa (160 psi).

Stage III. Obtain Volume Change Due to Loading in Terms of $D_2$—$D_2$ is the number of turns of a screw of diameter 4.05 cm (1.596 in.). Each turn of the screw is 0.254 cm (0.1 in.) thereby moving about 3.27 cm$^3$ (0.20 in.$^3$) of oil into the test chamber. In other words, if the specimen is of 10.16 cm (4 in.) diameter and 6.35 cm (2.5 in.) tall, each turn of the screw corresponds to a volumetric strain of 0.636 %. The test procedure follows the steps listed below:

(a) Continuing from the end of stage II, the top platen is raised until the vertical pressure is 552 kPa (80 psi).
(b) The horizontal pressure is then adjusted to 34.5 kPa (5 psi).
(c) The horizontal pressure is then increased to 689 kPa (100 psi) by turning the handle to move the screw and thus inject hydraulic fluid into the chamber. The number of turns of the handle, $D_2$, is recorded. The volume change of the specimen is related to $D_2$.

For asphalt mixtures, a vertical pressure of 2757 kPa (400 psi) is applied instead of 1103 kPa (160 psi).

Values from the Stabilometer Test

Two equations came out of the stabilometer test—one for asphalt concrete and another for soil materials: (a) the stabilometer value or $S$-value, and (b) the resistance value or $R$-value. The stabilometer value is used for asphalt concrete and is given as:

$$ S = \frac{22.2}{P_v D_2} + 0.222 $$

where:

$P_v$ = vertical pressure.
$P_h$ = horizontal pressure, and
$D_2$ = number of turns of the screw (to inject fluid oil into the chamber).

It was found in the 1930s that if the stabilometer value is between 30 and 35, the mix is only marginally stable. A stable mix would have a value of 35 and above. The $S$-value not used today and thus is not emphasized in this study.

The resistance value or $R$-value is used for soil materials and is the main focus of the paper here. The $R$-value is defined as:

$$ R = 100 - \frac{100}{\frac{2.5}{P_v D_2} \left( \frac{P_v}{P_h} - 1 \right) + 1} $$

The $S$-value and the $R$-value are similar in their purpose as measures of structural adequacy. It can be shown from Eqs 1 and 2 that the $R$-value and the $S$-value are related in the following manner:

$$ \frac{R}{100 - R} = 0.555 \frac{S}{100 - S} $$

Relating the $R$-value, the $S$-value, and the Elastic Modulus

The $R$-Value

The following section shows how the $R$-value can be related to the elastic modulus, $E$. Rearranging the $R$-value formula (Eq 2), gives:

$$ \frac{2.5}{D_2} \left( \frac{P_v}{P_h} - 1 \right) = \frac{R}{100 - R} $$
Consider a cylindrical test specimen in the test cell under a triaxial stress condition. The radial strain, $e_r$, is given by:

$$e_r = \frac{1}{E} [\sigma_r - v(\sigma_t + \sigma_z)]$$

(5)

where

$v = \text{Poisson's ratio},$

$E = \text{elastic modulus, and}$

$\sigma_r, \sigma_t, \sigma_z = \text{radial, tangential, and vertical stress, respectively}.$

The vertical strain, $e_z$, is given by:

$$e_z = \frac{1}{E} [\sigma_z - v(\sigma_t + \sigma_z)]$$

(6)

Subtracting Eq 6 from Eq 5 gives:

$$(e_z - e_v) E = (\sigma_z - \sigma_t) (1 - v)$$

(7)

The volumetric strain for the test specimen in the stabilometer test can be calculated from:

$$(e_r - e_v) \frac{\pi D^2}{4} L = C D_2$$

(8)

where

$D = \text{diameter of the sample, and}$

$C = \text{the conversion used to calculate the amount of fluid injected into the stabilometer chamber by turning the screw or handle one turn. According to Vallerga and Lovering [2], the volume of fluid moving into the chamber is given by 0.2D_2, and that C = 0.2.}$

Rearranging Eq 8 gives:

$$(e_z - e_v) = \frac{C D_2}{L} \frac{4}{\pi D^2}$$

(9)

Substituting Eq 9 into Eq 8 gives:

$$\frac{L}{D_2} (\sigma_z - \sigma_t) = E \frac{4}{(1 - v)} C \frac{4}{\pi D^2}$$

(10)

Writing in notations common to the R-value test, and dividing both sides by $P_h$, gives:

$$\frac{2.5}{D_2} \left( \frac{P_v}{P_h} - 1 \right) = E \frac{4}{(1 - v)} C \frac{4}{\pi D^2} \frac{1}{P_h}$$

(11)

Eliminating the left hand side of Eqs 4 and 11 by equating the two gives:

$$\frac{E}{(1 - v)} C \frac{4}{\pi D^2} \frac{1}{P_h} = \frac{R}{100 - R}$$

(12)

or

$$E = \frac{\pi D^2}{4C} (1 - v) \frac{R}{100 - R} P_h$$

(13)

The S-Value

The equation relating the S-value and the elastic modulus was obtained from Eq 3 and Eq 13, and is:

$$E = 0.555 \frac{\pi D^2}{4C} (1 - v) \frac{S}{100 - S} P_h$$

(14)

A Review of Engineering Parameters of Materials

The following sections describe relationships between cohesion, angle of internal friction, the at-rest lateral “earth” pressure coefficient, overconsolidation ratio, compaction-induced stress, and viscoelastic response.

Cohesion

Cohesion is a measure of the “stickiness” of clay. This characteristic is a manifestation of ionic attraction that takes place between clay particles and the surrounding water. Cohesion has also been attributed to van der Waal’s forces. “Apparent” cohesion refers to surface tension forces (from soil moisture or water) acting in the cohesionless soil mass.

The Angle of Internal Friction

Sometimes the word “resistance” is used instead of “friction” to describe the shearing resistance in soils. If we can visualize friction in a soil as the interaction between two sets of facing saw-tooth edges, then we can see why the angle of the tooth (emulating the grains) and the roughness of the edges of the tooth each contribute to the frictional resistance when the two edges are sheared. If the teeth are weak, asperity breaks occur. This idealized model is applicable to both cohesionless soils and cohesive soils.

Misconceptions—It is a common misconception that clays always have a measurable cohesion. The fact is that the effective cohesion in normally consolidated clays is zero or very close to zero. It is also a common misconception that clays do not have friction. Saturated clays do not have a friction angle. Unsaturated clays always have a friction angle. Also, overconsolidated clays have both cohesion and a friction angle.

Figure 2 shows the effective friction angle of normally consolidated soils grouped according to the Unified Soil Classification System. For coarse-grained soils [4], the friction angle is seen to vary with the relative density. As the packing gets tighter, the frictional resistance increases. Also, as the grain size increases, the friction angle increases. Additionally, a well-graded soil has

*plasticity index (%) for clayey soils

<table>
<thead>
<tr>
<th>Plasticity Index</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Average of data from Kenney [6]</td>
</tr>
<tr>
<td>C</td>
<td>Calculated from Kd data from Massarch [7]</td>
</tr>
</tbody>
</table>

*Note: Montmorillonite has PI of more than 100%, Illite has PI of around 30 to 50%, and Kaolinite has PI of around 15 to 20% - Ooles [8]*

FIG. 2—Angle of internal resistance according to soil classification.
greater frictional resistance than a poorly-graded one at the same relative density.

Figure 2 also shows the friction angles of clays with different plasticity indices. Stronger clump is related to a lower plasticity index, PI. Consequently, it can be expected that the friction angle of low PI clay would be higher than that of high PI clay. It is also noted that according to reported data [3,6], the effective friction angle of clay may range from about 20° for a plasticity index PI of 100 or about 35° for a PI close to zero. The φ to PI relationship was interpreted by us from the equation relating $K_o$ and PI that Massarsch [7] proposed.

**The At-Rest Lateral Earth Pressure Coefficient, $K_o$**

The at-rest lateral earth pressure coefficient is an intermediate state of stress that depends on the "nature of a soil and its geological history" [9]. The at-rest coefficient, $K_o$, is:

$$\sigma_h' = K_o \sigma_c'$$  \hspace{1cm} (15)

where
- $\sigma_h'$ = horizontal stress, and
- $\sigma_c'$ = vertical stress which is usually the overburden pressure (from the weight of the soil).

The simplified form of the equation proposed by Jaky [10,11] for a cohesionless soil is:

$$K_o = (1 - \sin \phi')$$  \hspace{1cm} (16)

It appears from test data that this equation is also applicable to unsaturated clays.

**The Overconsolidation Pressure**

Normally consolidated soil refers to a soil responding to a load- ing beyond which it has previously experienced. When a soil is subjected to a vertical pressure beyond that of the current overburden pressure, the soil aggregate support structure is changed. Schmidt [12,13] and Alpan [14] suggested that the at-rest earth pressure coefficient should be adjusted according to the overconsolidation ratio, OCR, in the following manner:

$$K_o^\circ = K_o (OCR)^n$$  \hspace{1cm} (17)

where $K_o^\circ$ is the adjusted at-rest coefficient. The exponent, $n$, ranges from 0.4–0.6 [14,15] with the latter being for dense sands [16]. From Eq 2 and assuming $n = 0.5$, get:

$$K_o^\circ = K_o \sqrt{OCR}$$  \hspace{1cm} (18)

and

$$K_o^\circ = (1 - \sin \phi') \sqrt{OCR}$$  \hspace{1cm} (19)

**Compaction-Modified At-Rest Coefficient**

Compaction alters the soil aggregate support structure and also induces residual stresses in soils. Duncan and Seed [17] have discussed the latter in a comprehensive manner. Table 1 shows typical values of $K_o$, $\phi'$, and an equivalent overconsolidation ratio, OCR*, for different soil conditions.

The $K_o$ values of compacted clays are higher than those for naturally occurring clays. One explanation would be to view $K_o$ as

<table>
<thead>
<tr>
<th>TABLE 1—At-rest coefficient and related soil parameters.</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_o^{\circ}$</td>
</tr>
<tr>
<td>Normally consolidated clay</td>
</tr>
<tr>
<td>Over-consolidated clay</td>
</tr>
<tr>
<td>Compacted clay</td>
</tr>
<tr>
<td>Heavily compacted clay</td>
</tr>
</tbody>
</table>

* From Lee et al. [18].

having two parts and the compaction modified at-rest coefficient, $K_{o-c}$, to be:

$$K_{o-c} = \frac{\sigma_h' + K_o \Delta \sigma_c'}{\sigma_c'}$$  \hspace{1cm} (20)

or

$$K_{o-c} = K_o + \frac{K_o \Delta \sigma_c'}{\sigma_c'}$$  \hspace{1cm} (21)

where $\Delta \sigma_c'$ is the vertical transient compaction pressure.

**Viscoelastic Behavior of Clays and Asphalt Concrete**

Another view of compaction-induced stress is to consider the material to be viscoelastic. Clays and asphalt concrete are viscoelastic whereas cohesionless materials like sand and gravels are not. When a sand or gravel is compacted in a confined space, stresses would be induced and these stresses are locked in soil arches. However, when the confinement is removed, the induced stress would be relieved. At the microscopic level, the structure of clays relies on chemical interaction. At the macroscopic level, much of the response of clay relies on the movement of soil moisture. Response of clays to loading is therefore time-dependent.

The time-dependent response of asphalt is different from that of clay. Asphalt is a polymeric material and the stretching, buckling, and slipping of polymeric chains is time-dependent. Additionally, the movement of the oils held in the polymeric assembly contributes to the viscous behavior. Asphalt concrete can be viewed as a very cohesive unbound material, much like clay, and as such it may be possible to model asphalt concrete like a soil. Chua and Lytton [19] discuss viscoelastic modeling of soils. Chua and Roo [20] discuss viscoelastic modeling of asphalt concrete.

**Typical Values of Poisson’s Ratio and Friction Angle of Soils and Highway Materials**

Tables 2 and 3 show typical values of Poisson’s ratio of soils and highway materials. Table 4 shows typical values of angle of internal friction for soils.

**The R-value, S-value, and Elastic Modulus Equations**

**Proposed Equations**

Two equations are proposed here. It is assumed here that the basic information for the material of interest, at a minimum, would include:

1. the $R$-value,
2. the exudation pressure,
3. a soil classification, and
4. an overconsolidation ratio (which is 1.0 for a normally consolidated soil).
TABLE 2—Typical values of Poisson’s ratio for soil materials.

<table>
<thead>
<tr>
<th>Type of Soil Materials</th>
<th>Poisson’s Ratio, ( \nu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, saturated</td>
<td>0.4–0.5</td>
</tr>
<tr>
<td>Clay, unsaturated</td>
<td>0.1–0.3</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>0.2–0.3</td>
</tr>
<tr>
<td>Silt</td>
<td>0.3–0.35</td>
</tr>
<tr>
<td>Sand, dense</td>
<td>0.2–0.4</td>
</tr>
<tr>
<td>Sand, coarse</td>
<td>0.15</td>
</tr>
<tr>
<td>Sand, fine</td>
<td>0.25</td>
</tr>
</tbody>
</table>

* From Bowles [21], p. 67, Table 2-7.

TABLE 3—Typical values of Poisson’s ratio for highway materials.

<table>
<thead>
<tr>
<th>Highway Materials</th>
<th>Shell Oil Co.</th>
<th>Shell Oil Co. Revised</th>
<th>Asphalt Institute</th>
<th>Kentucky Highway</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt concrete</td>
<td>0.5</td>
<td>0.35</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Granular base</td>
<td>0.5</td>
<td>0.35</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>Subgrade</td>
<td>0.5</td>
<td>0.35</td>
<td>0.45</td>
<td>0.45</td>
</tr>
</tbody>
</table>

* From Yoder and Witczak [22].

TABLE 4—Typical angle of internal friction.

<table>
<thead>
<tr>
<th>Type of Soil Materials</th>
<th>Angle of Internal Friction, ( \phi )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>40°–55</td>
</tr>
<tr>
<td>Medium size</td>
<td></td>
</tr>
<tr>
<td>Sandy</td>
<td>35–50</td>
</tr>
<tr>
<td>Sand</td>
<td></td>
</tr>
<tr>
<td>Loose dry</td>
<td>28–34</td>
</tr>
<tr>
<td>Loose saturated</td>
<td>28–34</td>
</tr>
<tr>
<td>Dense dry</td>
<td>35–46</td>
</tr>
<tr>
<td>Dense saturated</td>
<td>33–44</td>
</tr>
<tr>
<td>Silt or silty sand</td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>20–22</td>
</tr>
<tr>
<td>Dense</td>
<td>25–30</td>
</tr>
<tr>
<td>Clay</td>
<td></td>
</tr>
<tr>
<td>Saturated</td>
<td>0</td>
</tr>
<tr>
<td>Unsaturated</td>
<td>20–42</td>
</tr>
</tbody>
</table>

* From Bowles [21], p. 60, Table 2-5.

The presumptive material parameter inputs, which can be estimated from information presented in the preceding section, include:

1. a Poisson’s ratio, and
2. an angle of internal friction.

In the AASHO Road Test, \( R \)-value studies of highway soil materials were done under three categories: (a) embankment soil, (b) subbase materials, and (c) base course materials. Asphalt concrete was also studied. Materials are considered here in two categories, instead of the three categories referred to in the AASHO Road Test; they are:

1. cohesive materials, which would include clayey subgrade, and less likely, clayey subbase and clayey base course; and
2. cohesionless materials, which would include a nonclayey subgrade, subbase, or base course.

It was also observed from data obtained from the AASHO Road Test that for each material, there appears to be a minimum elastic modulus, \( E_\circ \). This would be a value that would be added to the theoretically calculated value. While the minimum elastic modulus is a product of regression analysis, it is reasonable to assume that a solid material could not have a zero modulus. The proposed equations are:

\[
E = \frac{\pi D^2}{4C} \left(1 - \nu\right) \frac{R}{100 - R} \times (1 - \sin \phi') \sqrt{OCR} \left(1 + \frac{\Delta \sigma_{ex}}{P_v^*}\right) P_v^* + E_\circ \tag{22}
\]

where

\[
D = 10.16 \text{ cm (4 in.) specimen diameter},
\]

\[
C = 0.2 \text{ conversion factor (used to calculate volume change),}
\]

\[
P_v^* = 1103 \text{ kPa (160 psi), (last applied vertical pressure), and}
\]

\[
\Delta \sigma_{ex} = \text{exudation pressure, which is indicative of water content.}
\]

The main difference between the two would be in the use of the exudation pressure in the calculation. Practical note: For cohesionless materials, \( \Delta \sigma_{ex} \) should be set to zero because the residual stress from the exudation stress would have been relieved when the specimen is removed from the preparation mold.

For asphalt concrete, the equation is:

\[
E = 0.555 \frac{\pi D^2}{4C} \left(1 - \nu\right) \frac{S}{100 - S} \times (1 - \sin \phi') \left(1 + \frac{\Delta \sigma_{comp}}{P_v^*}\right) P_v^* + E_\circ \tag{23}
\]

where

\[
D = 10.16 \text{ cm (4 in.) specimen diameter},
\]

\[
C = 0.2 \text{ conversion factor (used to calculate volume change),}
\]

\[
P_v^* = 1103 \text{ kPa (160 psi), (last applied vertical pressure), and}
\]

\[
\Delta \sigma_{comp} = \text{compaction stress.}
\]

The values of \( E_\circ \) for clay, granular subgrade, granular course, and asphalt concrete are 13 790 kPa, 13 790 kPa, 17 237 kPa, and 344 738 kPa (2000 psi, 2000 psi, 7500 psi, and 50 000 psi), respectively.

The Need to Estimate the Horizontal Pressure

An important input in Eqs 13 and 14 is the horizontal pressure experienced by the material, \( P_h \). In an ideal condition, \( P_h \) would be the same as the horizontal pressure measured in the fluid chamber surrounding the specimen. However because of friction at the ends, stress redistribution, and compaction-induced stresses present in the material, the horizontal stress is different from the oil pressure in the jacket.

The value for the horizontal pressure can be selected in several ways:

1. use the last \( P_h \) value measured in the \( P_o/P_h \) portion of the test, or
2. estimate \( P_h \) from \( P_v \) using the at-rest lateral earth pressure coefficient, \( K_o \), as the multiplier.

The latter is preferred.
A Comment about the Exudation Pressure and Stiffness—Exudation pressure, which is read during the sample preparation phase of the test, reflects the moisture content of the sample rather than the in situ moisture content. A drier soil would require a higher exudation pressure and vice versa.

One problem with the compaction-induced stress, which results either from compacting the sample or from performing the exudation pressure part of the test, is that this stress may not be there in the ground. If this is the case, there would be a lower elastic modulus value associated with the elastic modulus.

Illustration: Calculating $E$ from $R$—A clay has the following characteristics:

1. Poisson’s ratio $= 0.5,$ and
2. angle of internal friction $= 0^\circ$

Note: the friction is related to the plasticity index and the cohesion (see Fig. 2). Since clay is a viscoelastic material, it is assumed here that the test procedure was completed without a long span of a lapsed time between test steps, in which case the memory of the exudation pressure would still be intact.

Applied vertical pressure, $P_v = 1103 \text{ kPa (160 psi)}$
Exudation pressure, $\Delta \sigma_{ex} = 2068 \text{ kPa (300 psi)}$
$R = 20.$

To determine the elastic modulus, first estimate the overconsolidation pressure, $OCR.$ Since the exudation pressure is 2068 kPa (300 psi), which is higher than the last applied vertical pressure, $P_v,$ of 1103 kPa (160 psi), the $OCR = 2068/1103 = 1.875.$ Note: In the case of cohesionless materials, there is no need to consider the exudation pressure, and $OCR = 1.0.$ Therefore, assuming $E_o = 13790 \text{ kPa (2000 psi)},$ from Eq 22, the elastic modulus, $E = 47919 \text{ kPa (6950 psi)}.$

Evaluation of the New $R$-value Equation with Laboratory Data

The New Mexico state highway department performed stabilometer testing on three types of soils, which, according to AASHTO classification, are: A-1-b, A-2-4, and A-2-6. Probable Unified Soil Classification System designations are SW, GM, and GC, respectively. For the calculation of the elastic modulus from the $R$-value, the following assumptions are made:

1. no compaction-induced stress (because of the coarse grained nature of the material)
2. the friction angle for gravel is $40^\circ,$ and for sand is $35^\circ.$

Figure 3 shows the calculated elastic modulus based on the laboratory test data and the determined $R$-values for eight highway soil materials. It can be seen that the relationship is somewhat
geometric and that the elastic modulus approaches infinity as $R$ approaches 100 (Note: a perfectly rigid material has an $R$-value of 100). On the low end of the $R$-value, the elastic modulus is assigned a base value, $E_0$. Each soil sample was tested at three different moisture contents. The higher elastic modulus and $R$-value correspond to the higher moisture content.

Figure 4 shows how the exudation pressure is affected by the moisture content for the eight soil samples.

Figure 5 shows the influence of moisture content on the $R$-value. In one case (sand), it can be seen that the $R$-value ranges from about 15.3 to 70.5 for moisture contents of 12.5 % to 9.2 %, respectively. The elastic modulus varies from 55 158 kPa–137 895 kPa (8000 psi to 20 000 psi).

The sensitivity of the $R$-value to moisture contents raises a difficult choice for highway engineers: which $R$-value to report. Nevertheless, with the proposed equations, one can now better assess laboratory test data and this should help in the decision-making process.

Conclusion

Presented herein are rational equations that relate the $R$-value, $S$-value, and the elastic modulus, $E$. It was shown in the development of the equations that the elastic modulus is dependent on the following:

1. the $R$- or $S$-value,
2. the exudation pressure (as reported in the sample preparation),
3. the overconsolidation ratio (which can be estimated from the compaction pressure),
4. the Poisson’s ratio (which can be estimated), and
5. the angle of internal friction of the material (which can be estimated).
The compaction pressure would also affect the calculation if the material is time-dependent or viscoelastic.

A sampling of laboratory test data was also presented to demonstrate how the R-value is related to the elastic modulus for three different highway soil materials. It is also shown that the exudation pressure is indicative of the moisture content of the test samples and that the R-value and elastic modulus are quite sensitive to the moisture content.

At the present time, the new equations are being used to review the test data from the AASHO Road Test with the aim of obtaining better predictions of pavement layer coefficients used in the current AASHTO pavement design method. The finite-element method is being used to simulate the Hveem stabilometer test and this would provide more insights into how the elastic modulus changes during the different test steps.

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References


