Modeling Long-Term Deformations of Unbound Pavement Materials Using the Miniaturized Pressuremeter Creep Data

ABSTRACT

This research was undertaken to study the predictive capability of the pressuremeter test for characterizing in situ creep behavior of unbound pavement layers. Although the creep potential of granular pavement materials is less pronounced than fine-grained soils, consideration of actual creep deformations in the pavement evaluation process will improve long-term pavement performance. In this investigation, the long-term deformations determined from laboratory one-dimensional creep tests were compared with those investigated by field pressuremeter tests. The pressuremeter test consisted of inflating a cylindrical probe incrementally up to a given stress level, and then maintaining the pressure constant for a 5-min single stage. During this stage, radial deformations of the soil cavity were recorded at each 30-s interval. The one-dimensional creep test was performed on remolded soil specimens through applying a constant stress level for 7 days. Comparison of creep parameters deduced from pressuremeter and creep tests data was based on the Singh–Mitchell creep model. The results showed that the average strain rates derived from in situ pressuremeter data are valid, and compare well with those predicted from the laboratory creep test. Thus, the pressuremeter device can be employed to assess field strain–time behavior of pavement systems in a fast and reliable approach.

Keywords
pressuremeter, creep, pavement, strain rate, time-dependent behavior

Introduction

Time-dependent change in any soil structure is often called creep. This phenomenon occurs when a soil mass is subjected to a constant loading condition for a long term. The magnitude and rate of...
these time-dependent deformations are influential on the service life of roadways. In general, studying the time-dependent response of pavement granular layers is significant not only to evaluate the serviceability of roadways, but also to gain fundamental information about changes in soil properties over time. These changes may include softening, hardening, strength loss, strength increase, or reforming conductivity of drainage pavement materials (Gidel et al. 2001; Liingaard et al. 2004).

A comprehensive literature review on the time-dependent behavior of soils was presented by Augustesen et al. (2004). The authors pointed out that coarse-grained soils can exhibit a considerable amount of time-dependent deformations just like fine-grained soils. However, creep behavior of cohesive materials is more pronounced because of their rheological characteristics. Two important factors were investigated for identifying the creep behavior of granular materials: stress dependency and strain versus time.

- For stress-dependency behavior, it was found that at a low confining pressure, the soil experiences shear deformations caused by sliding and rolling soil particles, whereas at a high confining pressure, the soil experiences compressive deformations caused by fracturing and crushing of soil particles.
- For strain–time behavior, it was found that at a low confining pressure, there is a linear relationship that exists between creep strain rates and logarithm of time. The same tendency was observed at a high confining pressure.

Experimental studies were conducted by several geotechnical authors to investigate in situ determination of the soil creep using field pressuremeter test. The first investigation was performed by Ladanyi and Johnston (1973). In this study, strain–time parameters of ice-rich frozen clay and silt soils were measured using the pressuremeter creep data. Two testing approaches were adopted in this study: the multistage pressuremeter test method with 15 min per stage, and the single-stage step-strained testing approach was implemented using TEXAM equipment. The stress readings recorded during decaying pressure is recorded over time. In this investigation, a pressuremeter relaxation test was performed by maintaining a constant radial strain, and the cavity acceleration.

Ladanyi and Johnston (1973) showed that the creep lines obtained from the data of the multistage pressuremeter creep test can be used to predict the strain creep rate of the power law model developed by Hult (1966):

$$\dot{\epsilon}_c = A \sigma_c^n t^b$$  \hspace{1cm} (1)

where \(A\) is defined as a creep factor according to:

$$A = \left( \frac{\dot{\epsilon}_c}{\sigma_c} \right)^{\frac{1}{n}}$$  \hspace{1cm} (2)

Substituting Eq 1 in Eq 2 results in the following expression:

$$\dot{\epsilon}_c = \left( \frac{\dot{\epsilon}_c}{\sigma_c} \right)^{\frac{1}{n}} \sigma_c^n t^b$$  \hspace{1cm} (3)

where:

- \(\dot{\epsilon}_c\) = equivalent creep strain rate,
- \(\sigma_c\) = equivalent creep stress,
- \(\dot{\epsilon}_c\) = an arbitrary strain rate, conventionally equal to \(10^{-5}\) (min \(^{-1}\)),
- \(\sigma_c\) = creep proof stress corresponding to the constant strain rate, \(\dot{\epsilon}_c\),
- \(b\) = creep time exponent representing the slope of a log-plot of strain rate versus time, and
- \(n\) = creep stress exponent representing the slope of a log-plot of strain rate versus stress.

Kjartanson et al. (1987) carried out a series of the creep pressuremeter tests in fabricated polycrystalline ice samples at temperature \(-2^\circ C\). In this study, the modified second-order fluid model was used to analyze the pressuremeter creep data:

$$\frac{\dot{\epsilon}_c}{\epsilon_0} = \frac{m}{m + 2} \left( \frac{r_o}{r_e} \right)^2 + \mu \left( \frac{r_o}{r_e} \right)^2 + \frac{P_c}{2} (m + 1) \left( \frac{r_o}{r_e} \right)^{-m} = 0$$  \hspace{1cm} (4)

where:

- \(\mu\), \(\alpha\), \(m\) = material coefficients,
- \(r_o\) = the initial cavity radius,
- \(\dot{\epsilon}_c\) = the cavity expansion rate, and
- \(\dot{\epsilon}_c\) = the cavity acceleration.

The parameter \(P_c\) is defined as the corrected cavity pressure, which is measured from the stress–strain curve of the pressuremeter test.

The mathematical solution of the complicated nonlinear differential expression presented in Eq 4, was discussed in detail by Kjartanson (1986). The results of the theoretical analysis showed that the multistage creep test had the potential to determine creep properties of the frozen materials in the field. In addition, it was found that using a modified second-order fluid model can accurately predict long-term creep of the frozen soils.

Using the pressuremeter relaxation test to evaluate creep properties of frozen sand soils was investigated by Ladanyi and Melouki (1993). The pressuremeter relaxation test was performed by maintaining a constant radial strain, and the decaying pressure is recorded over time. In this investigation, a step-strained testing approach was implemented using TEXAM pressuremeter equipment. The stress readings recorded during 16 min of the step-strained borehole relaxation test were analyzed using three interpretation techniques: isochronous curve method, reference stress method, and aging theory of creep method. It was found that the isochronous curve method presented in Eq 3 provides satisfying results. In contrast, the ageing creep theory and reference stress methods yielded different strain creep parameters. This deviation may be attributed to the fact that these techniques were developed to interpret relaxation behavior of metals.

Arenson et al. (2003) determined in situ strength properties and creep behavior of active rock glaciers in Switzerland. Seven
multistage creep pressuremeter tests were conducted at different depths using a 95-mm (width), and 14-mm (thickness) high-pressure dilatometer. Each test lasted up to 1020 min with a range of various stress levels. The obtained creep data were analyzed using the theoretical analysis method recommended by Ladanyi and Johnston (1973). The results indicated that the stress exponent $n$ of power-law model (see Eq 3), increases with increasing testing depth, whereas creep factor $A$ tends to decrease. Also, the results exhibited that the creep strain rate values decreased with depth, and increased exponentially with increasing borehole cavity pressure.

The objective of the research presented in this paper is to evaluate long-term deformations of pavement layers using the single-stage pressuremeter creep tests. The pressuremeter creep data were analyzed to determine in situ creep parameters, and then compared with those predicted from laboratory one-dimensional creep test. The phenomenological model proposed by Singh and Mitchell (1968) was used to interpret and describe stress–strain–time response of pavement materials.

### Applied Strain–Time Model

The slope of the strain–time curve can be divided into three distinct phases, according to the amount of shear deformations generated in a soil experiencing creep. As shown in Fig. 1, the three phases of a creep curve are: primary, secondary, and tertiary. The primary phase represents initial shear deformations, during which strain rate decreases with time. During secondary creep, shear deformations stay nearly constant over an extended period of time. After that, the strain rates start to continuously increase and this is denoted as the tertiary phase. This phase is usually followed by a failure creep rupture point. Fig. 1 shows general creep behavior of a soil under constant loading conditions.

Hypothetical models for strain–time behavior have been introduced by many authors. These models vary from simple empirical models to more sophisticated constitutive models.

In this work, a simplified empirical model proposed by Singh and Mitchell (1986) will be used to analyze creep data obtained from the in situ MPMT test. Mitchell (1993) noticed that if strain–time data are plotted on logarithm of strain rate versus logarithm of time, the resulting relationship was always linear (see Fig. 2). This relationship can be expressed as:

$$\ln \left( \frac{\dot{\varepsilon}}{\dot{\varepsilon}(t_1, D)} \right) = -m \cdot \ln \left( \frac{t}{t_1} \right)$$  \hspace{1cm} (5)

where:
- $\dot{\varepsilon}$ = the strain rate at any time,
- $\dot{\varepsilon}(t_1, D)$ = the strain rate at unit time and is a function of creep stress intensity,
- $m$ = the slope of the resulting line,
- $t_1$ = a reference unit of time, and
- $t$ = a particular time at which the creep behavior is desired.

As shown in Fig. 2, the slopes of the lines are independent of applied creep pressure, and the only effect from creep pressure was to move the lines upward and to the right.

Another relationship was proposed by Singh and Mitchell (1968) to include the effect of stress intensity on creep behavior. It was found that if logarithm strain rate values were plotted versus creep stress levels, the obtained curve takes the form of a hyperbolic (see Fig. 3). In the midrange of stress values, an approximate linear relationship was formed between strain rate and stress intensity, and this relationship can be presented as follows:

$$\ln(\dot{\varepsilon}) = \ln(\dot{\varepsilon}(t, D_0)) + zD$$  \hspace{1cm} (6)

where:
- $\dot{\varepsilon}$ = the strain rate at any time,
- $\dot{\varepsilon}(t_1, D)$ = a value of strain rate after the start of creep at $D = 0$,
- $z$ = the slope of the linear portion of log strain versus stress intensity, and
- $D$ = the creep stress intensity.
Combining Eq 5 and Eq 6 results in a simple three-parameter empirical model that can be used to predict time-deformation behavior of different types of soils:

$$\dot{\varepsilon} = A e^{\alpha (\frac{t_1}{t})^m}$$

where:

$A$ = obtained by projecting a straight linear portion of the relationship between the logarithm of strain rate and unit time to a value of $D = 0$.

The value of $A$ can also be denoted as $\dot{\varepsilon}(t_1, D)$, and is shown in Fig. 3. Mitchell (1993) mentioned that either the triaxial deviator stress or the axial odometer stress could be used as a specified creep stress. It was also stated that at least two creep tests are needed to determine creep strain parameters $A$, $\alpha$, and $D$.

Test Equipment and Procedures

MINIATURIZED PRESSUREMETER TEST

A modified version of the PENCEL pressuremeter probe, which was developed by Shaban and Cosentino (2016), was used in this investigation. The new pressuremeter probe is called miniaturized pressuremeter probe (MPMT). It has a short inflatable probe length equal to 6 in., which allowed it to be used in thin pavement layers. The MPMT consists of two main parts: control unit and cylindrical probe (see Fig. 4).

The general testing procedure proposed by Briaud (1992) was followed to evaluate field long-term deformations of unbound pavement layers. However, several modifications were added to the procedure to accommodate the small size of the MPMT probe. The testing method was executed in the following order:

Step 1

The whole system of the MPMT (i.e., control unit and probe) was saturated according to the testing process presented in ASTM D4719 (2007). The probe was saturated by injecting a de-aired water through the probe until no air bubbles were apparent, whereas the control unit tubing was saturated by pumping the de-aired water out of the cylindrical piston and drawing it back several times until all entrapped air is removed.

Step 2

After the completion of the saturation procedure, a membrane calibration is conducted to measure the amount of pressure required to expand flexible membrane without any resistance. The calibration was performed by inflating the probe in air using 5-cm³ volume increments until the volume reached 60 cm³. During the inflation process, the pressure readings were taken after each increment of water injected.

Step 3

To complete the MPMT system calibration procedure, it was required to measure volume losses in the control unit and the probe by conducting the system expansion calibration. This step was performed by inflating the probe inside a thick-walled metal tube. First, the probe was inserted in a 1.30-in. diameter tube, then the probe was pressurized by injecting water at a rate of 0.33 cm³/s. During the injection process, the pressures are recorded at each 5-cm³ increment up to 2500 kPa.

Step 4

After completing the calibration processes, a borehole was prepared by driving a 1.30-in. diameter hollow circular steel tube into a pavement layer. The tube was removed after reaching the desired test depth. Then, the probe was inserted into the pre-drilled borehole.
**Step 5**
The operator injected 5-cm³ increments of water from the control unit into the probe until a volumetric expansion of 45 cm³ was reached. Once, the volume of 45 cm³ was achieved, a constant pressure was maintained for 5 min. During this period, the increase in radial deformation of the probe was recorded every 30 s.

**Step 6**
After the completion of the creep portion, the probe was inflated continuously to about 1.35 times its original volume, or to a maximum 60-cm³ volume.

Fig. 5 shows a typical MPMT stress–strain curve. The MPMT raw data test obtained by performing the above steps were corrected to identify reference volume and pressure applied on the walls of the MPMT borehole. These corrections include:

Volume correction:

\[ V_{Corrected} = V_{Raw} - V_{Membrane} \]  

Pressure correction:

\[ P_{Corrected} = P_{Raw} - P_{Membrane} + P_{Hydrostatic} \]

**ONE-DIMENSIONAL CREEP TEST**
The one-dimensional creep test was performed on a 6-in. diameter compacted specimen of granular pavement materials. The soils collected from different roadway projects were compacted at equilibrium field moisture contents according to ASTM D1557-12e1 (2012). After completing the sample preparation, the soil samples were placed inside a set of pneumatic loading devices developed by Cosentino et al. (2012). For evaluating creep behavior of pavement materials, each creep test lasted for 7 days under a constant stress level. The creep stress level adopted in this work was 172 kPa (25 psi), which represents an average value of the traffic loading conditions encountered in most flexible pavement systems.

The pneumatic loading device (PLD) consists of a pneumatic loading piston mounted to the top of a fabricated loading frame. The loading frame was constructed using two aluminum beams joined by two steel rods. The loading piston is connected to a tank of nitrogen gas through a system of galvanized steel pipes. The applied gas pressure can be controlled by an air regulator attached to a precision pressure gauge.

The PLD is equipped with loading and unloading valve to switch the supply pressure on and off. A steel-bearing ball was used as a single contact point to transfer pressure to the tested sample. Furthermore, a 0.5-in.-thick aluminum plate was placed on the sample surface to uniformly distribute the applied creep pressure. A string linear potentiometer, which can be used to measure displacements up to 1 in., was attached to the aluminum plate to measure longitudinal deflections of the specimen.

Fig. 6 presents the main components of the PLD.

The collection of creep data was done using LabView software (National Instruments, Austin, TX). The program was created to monitor and store three basic parameters: deflection, time, and strain. Deflection and time data are recorded, and the average strain is calculated based on the sample height. These parameters are recorded every second during the first 2 min of testing. After that, the time intervals are doubled (i.e., 2, 4, 8, 10..., etc.) until the sampling intervals reach 4 h. Then, the data is progressively recorded at 4-h intervals until the completion of the test. The recorded data is saved as a .cvs file, which can be later viewed in different software packages like Microsoft Excel.

Fig. 7 shows typical strain–time curve obtained from the one-dimensional creep test.

**Testing Site Locations**
The results reported in this work were developed using the data from four field testing sites in Central and South Brevard County, Florida. The first field experimental measurements were conducted on a 550-m (1800-ft) test section located on St. Johns Heritage Parkway-Phase II, in Western Palm Bay. This site consists of a natural subgrade soil that is about 9 m (30 ft) deep and contains sand and deposits of shells and fossils. The grain size distribution of subgrade soil in this segment was classified as poorly graded sand (SP), according to the Unified Soil Classification System (ASTM D2487-11 2011), and type A-3 soil according to the American Association of State Highway and Transportation Officials (AASHTO M145-91 2012). The average field wet density of the subgrade soil was 18.9 kN/m³.

A reconstructed section of Babcock St. (SR 507) starting from Melbourne Avenue to Silver Palm Avenue was selected as the second field test site. The improved segment of the road includes 31 cm (12 in.) sand layer stabilized with 10 % cement.
The stabilized sand was also classified as poorly graded sand (SP) according to USCS, and type A-3 soil according to AASHTO. The average field wet density of the subgrade soil was 21.3 kN/m³.

The third field measurements were carried out on 762-m (2250-ft) well-compacted base course layers located on St. Johns Heritage Parkway-Phase I. The first phase includes 2.5 miles of two-lane access road that starts from Malabar Road and runs north to Emerson Drive in Palm Bay. This phase was constructed by placing a 25-cm (10-in.) base layer of untreated cemented coquina, which was classified as well graded sand (SW) with gravels according to USCS, and type A-1-b soil according to AASHTO. The average field wet density of the base layer was 21.5 kN/m³.

The fourth field site was an improved U.S. Highway 1 segment in Rockledge, starting from Carleton Drive to River Height Drive. The improved section was constructed by placing 25 cm (10 in.) of limerock over 38 cm (15 in.) of stabilized subgrade sand soil. The limerock base course was classified as well-graded sand (SW) with gravel according to USCS, and type A-1-a soil according to AASHTO. The average field wet density of the base layer was 21.9 kN/m³.

Table 1 includes a summary of gradations, laboratory and field densities, and moisture contents of the subgrade and base course materials that were tested in this research.

Results and Analysis

MPMT CREEP DATA

The MPMT creep test results were analyzed in this section in terms of the time-dependent soil response model introduced by Singh and Mitchell (1968). The creep parameters (i.e., m, A,x) have been calculated from the interpretation of 5 min single stage creep data.

In this research, an axial creep strain was replaced by a radial strain because the pressuremeter uses the change in radius of the MPMT probe to define cavity expansion of the borehole test:
where:

\( e_c = \) pressuremeter creep strain,
\( R_i = \) radius of MPMT probe at the beginning of the creep test, and
\( R_{i+1} = \) radius of MPMT probe at any creep time.

The 5-min creep strain values calculated from Eq 10 were extrapolated for longer time periods (i.e., 7 days) using a logarithmic curve fitting. The extrapolated strain values were used to measure creep strain rates for 7 days. Then, the strain rates were plotted versus time on a log–log plot resulting in a straight line. According to Eq 5, the slope of the creep straight line is denoted as strain–time exponent \( m \).

To determine the parameters \( A \) and \( \alpha \), the log of strain rates were plotted versus radial creep pressure yielding an exponential curve. According to Eq 5, the \( y \)-intercept of the exponential curve is \( A \), and \( x \) is the slope of the linear portion of the curve. The results of the foregoing analysis for the selected roadway sites are presented as follows.

The results from testing the subgrade soils showed that the \( m \) values varied from 0.829 to 0.856. The average \( m \) values were 0.840 and 0.843 measured based on creep strain data of Heritage Parkway-Phase II and Babcock St., respectively. As shown in Figs. 8a and 9a, the MPMT strain rates are in the range of 0.0001 to 0.1 (%/min). Also, the MPMT creep stress parameters \( A \) and \( \alpha \) were determined from the plot of log

**TABLE 1** Basic physical properties of unbound pavement materials.

<table>
<thead>
<tr>
<th>Geotechnical Property</th>
<th>Subgrade Soils</th>
<th>Base Course</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Parkway-Phase II</td>
<td>Babcock St.</td>
</tr>
<tr>
<td>Dry unit weight</td>
<td>18.6 kN/m(^3)</td>
<td>18.4 kN/m(^3)</td>
</tr>
<tr>
<td>OMC</td>
<td>12 %</td>
<td>7.5 %</td>
</tr>
<tr>
<td>Field unit weight</td>
<td>19.80 kN/m(^3)</td>
<td>21.3 kN/m(^3)</td>
</tr>
<tr>
<td>Uniformity Coefficient</td>
<td>2.82</td>
<td>2.12</td>
</tr>
<tr>
<td>Curvature coefficient</td>
<td>1.23</td>
<td>0.96</td>
</tr>
<tr>
<td>Gravel fraction (GF)</td>
<td>3 %</td>
<td>3 %</td>
</tr>
<tr>
<td>Fine content</td>
<td>2 %</td>
<td>1 %</td>
</tr>
<tr>
<td>Soil classification</td>
<td>A-3</td>
<td>A-3</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>SP</td>
</tr>
</tbody>
</table>

**FIG. 8** Determining parameters of Singh–Mitchell model based on subgrade MPMT creep data of Heritage Parkway-Phase II.

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The results from testing the base course materials indicated that the \( m \) values ranged from 0.831 to 0.869. The average \( m \) values were 0.850 and 0.841 determined based on creep strain data of Heritage Parkway-Phase I and U.S. Highway 1, respectively. As shown in Figs. 10a and 11a, the MPMT strain rates are in the range of 0.0001 to 0.1 (%/min). Also, the MPMT creep stress parameters \( A \) and \( \alpha \) were determined from the plot of log
strain rate versus time at reference creep time equal to 1 day (see Figs. 10b and 11b). The average $A$ values were $4.0 \times 10^{-4}$ (%/min) and $3.0 \times 10^{-4}$ (%/min) for Heritage Parkway-Phase I and U.S. Highway 1, respectively. The average $a$ values were $6.0 \times 10^{-4}$ (1/kPa) and $8.0 \times 10^{-4}$ (1/kPa) measured based on creep stress data of Heritage Parkway-Phase I and U.S. Highway 1, respectively. It was also found that the larger strain rates were occurred at high stress levels.

**ONE-DIMENSIONAL CREEP DATA**

The results of laboratory creep data showed that pavement materials exhibit instantaneous deformations during the first day of the test, after which the soil deforms slowly at nearly constant strain rate. The axial strains were analyzed to determine creep parameters of the Singh and Mitchell model. The slope of the creep straight line of the strain rate versus time was evaluated to assess the magnitude of the strain exponent ($m$). In addition, the strain–time curve for each soil type was obtained at three stress levels (83, 124, 172 kPa) to evaluate the effect of stress level on creep behavior. The results obtained from one-dimensional creep tests for four roadway projects are discussed in the following subsection.

For subgrade soils, the results indicated that the average $m$ values determined based on the creep strain data of Heritage Parkway-Phase II and Babcock St. were 0.890 and 0.870, respectively. **Figs. 12a and 13a** show the average creep strain lines of the subgrade soils. Also, the creep stress parameters $A$ and $a$ were obtained by performing three creep tests at three different stress levels (83, 124, 172 kPa). For Heritage Parkway-Phase II, the creep stress parameters $A$ and $a$ were $3.0 \times 10^{-4}$ (%/min) and $59 \times 10^{-4}$ (1/kPa), respectively. As shown in **Fig. 12b**, the strain rates varied from $4.2 \times 10^{-4}$ to $7.1 \times 10^{-4}$ (%/min) corresponding to creep stresses varying from 83 to 172 kPa. For Babcock St., the slope of the resulting curve $a$ was $51 \times 10^{-4}$ and the $y$-intercept $A$ was $3.0 \times 10^{-4}$ using reference time $t_1 = 1$ day. The magnitude of strain rates are in the range of
4.01 \times 10^{-4} \text{ to } 6.3 \times 10^{-4} \text{ (%/min)} \text{ corresponding to creep stresses ranging from } 83 \text{ to } 172 \text{ kPa (see Fig. 13b).}

For base course layers, the results showed that the granular materials experienced relatively low deformations over the first testing day; thereafter, the creep curve was flattened out at approximately constant strain equal to \(28 \times 10^{-4} \%\). The slope of logarithmic fitting curve of strain rates \((m)\) was 0.956 and 0.890 for Heritage Parkway-Phase I and U.S. Highway 1, respectively. Figs. 14a and 15a show average strain–time relationships for base materials of both sites. The creep stress parameters \(A\) and \(\alpha\) were derived from the plot of the log strain rate versus the three selected stress level. The results revealed that the value of the \(y\)-intercept \((A)\) was \(2.0 \times 10^{-4} \text{ (%/min)}\) for both sites, whereas the slopes of the creep stress curve of Heritage Parkway-Phase I and U.S. Highway 1 were \(68 \times 10^{-4} \text{ (1/kPa)}\) and \(34 \times 10^{-4} \text{ (1/kPa)}\), respectively. Figs. 14b and 15b show exponential curves plotted between the log of strain rates versus stress levels at time \(t_1 = 1\) day.

Comparison Analysis Between Creep and MPMT Data

The creep results obtained from MPMT test were compared with those determined from the laboratory one-dimensional creep test to evaluate the creep predictive capability of the MPMT test. The comparison analysis between MPMT and creep data has been performed in two major ways:

- The creep exponent \((m)\) was determined and compared because this parameter has a primary influence on creep prediction.
- The strain rate \((\dot{\varepsilon})\) measured in terms of the time-dependent soils response model had to be considered as representing the actual prediction of creep deformation.

**CREEP EXPONENT \((M)\)**

**Heritage Parkway-Phase II**

The slope of the strain rate lines \((m)\) determined using the MPMT tests were within 6% of experimental data observed
using the laboratory creep test. Table 2 lists the ratio of predicted to measured exponent \(m_{MPMT}/m_{Creep}\) of the subgrade soil of Parkway-Phase II. The results indicated that the highest ratio was 1.00, whereas the lowest ratio was 0.90. As shown in Table 2, the \(m\) values predicted from MPMT tests are slightly lower than those estimated from the creep test. The average decrease in \(m\) values was about 0.053.

**Babcock St.**

The \(m\) values determined using the MPMT tests were within 8% of experimental data observed using the laboratory creep test. As reported in Table 2, the ratio of predicted to measured exponent \(m_{MPMT}/m_{Creep}\) ranged from 0.87 to 0.95. Also, it was noticed that the \(m\) values predicted from MPMT tests are slightly lower than those estimated from the creep test. The average decrease in \(m\) values was about 0.071.

**Heritage Parkway-Phase I**

The \(m\) values measured using MPMT tests were within 10% of experimental data observed using the laboratory creep test. Table 2 summarizes the ratio of predicted to measured exponent \(m_{MPMT}/m_{Creep}\) of base materials of Parkway-Phase I. As shown in Table 3, the highest ratio was 0.99, whereas the lowest ratio was 0.86. The results showed that the \(m\) values predicted from MPMT tests are slightly lower than those estimated from the creep test. The average decrease in \(m\) values was about 0.093.

**U.S. Highway 1**

The \(m\) values measured using the MPMT tests were within 6% of the experimental data observed using the laboratory creep test. As reported in Table 3, \(t\) is the ratio of predicted to measured exponent \(m_{MPMT}/m_{Creep}\) ranging from 0.91 to 1.0. Also, it was found that the \(m\) values predicted from MPMT tests are slightly lower than those estimated from the creep test. The average decrease in \(m\) values was about 0.057.
The long-term deformations based on strain rate were predicted using creep data derived from both field MPMT and laboratory creep tests. The obtained strain rates were projected for three different pavement planning lives (i.e., 1, 10, and 20 years) to evaluate permanent deformations that might occur during the life-cycle of these unbound granular pavement layers. Another important factor that has been taken into account was the magnitude of the stress level specified in Eq 7. To ensure the accuracy of the strain rates, the stress levels generated under standard traffic load were used instead of using a stress intensity selected based on materials strength. Mitchell (1993) defined stress level as 30% to 90% of ultimate strength of the tested materials. Table 4 lists the values of compressive stress distributed under a standard 356 kN (80 kip) truck load.

The maximum compressive stresses reported in the above table were substituted into the following models to produce strain rates for each roadway project (see Table 5).

Heritage Parkway-Phase II
The values predicted using the MPMT data were larger than those predicted from the laboratory creep test. Fig. 16a reports strain rate values projected for the three different evaluation periods for the subgrade soils of Parkway-Phase II. The lab creep values were 36% and 37% lower than the MPMT values determined at 10 and 20 years, respectively. After 20 years, the average predicted strain for MPMT and creep was 0.026 and 0.016, respectively.

Babcock St.
The values estimated using the MPMT data were larger than those predicted from the laboratory creep test. Fig. 16b shows strain rate values projected for the three different evaluation periods for the subgrade soils of Babcock St. The laboratory creep values were 19% and 20% lower than MPMT values measured at 10 and 20 years, respectively. After 20 years, the average predicted strain for MPMT and creep was 0.025 and 0.020, respectively.

Heritage Parkway-Phase I
The values predicted using the MPMT data were larger than those estimated from the laboratory creep test. Fig. 17a reports
strain rate values projected for the three different evaluation periods for the base course of Parkway-Phase I. The laboratory creep values were 19% and 25% lower than MPMT values measured at 10 and 20 years, respectively. After 20 years, the average predicted strain for MPMT and creep was 0.025 and 0.019, respectively.

U.S. Highway 1

The $\dot{e}$ values estimated using the MPMT data were larger than those predicted from the laboratory creep test. Fig. 17b presents strain rate values projected for the three different evaluation periods for the base course of U.S. Highway 1. The laboratory creep $\dot{e}$ values were 18% and 21% lower than MPMT $\dot{e}$ values measured at 10 and 20 years, respectively. After 20 years, the average predicted strain for MPMT and creep was 0.021 and 0.017, respectively.

### TABLE 3 Summary of creep exponent values for base course materials.

<table>
<thead>
<tr>
<th>No.</th>
<th>Station</th>
<th>$m_{\text{MPMT}}$</th>
<th>$m_{\text{Creep}}$</th>
<th>$m_{\text{MPMT}}/m_{\text{Creep}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>104 + 00</td>
<td>0.851</td>
<td>0.917</td>
<td>0.93</td>
</tr>
<tr>
<td>2</td>
<td>105 + 50</td>
<td>0.849</td>
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U.S. Highway 1

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<th>No.</th>
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### TABLE 4 Maximum stresses induced in base course and subgrade soil.

<table>
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<tr>
<th>Axle Configuration</th>
<th>Compressive Stress (kPa)</th>
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<td>Top of Base Course</td>
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<td>Tandem Axle</td>
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### TABLE 5 Time-dependent soil response model.

<table>
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<tr>
<th>Field Site</th>
<th>MPMT Model</th>
<th>Creep Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parkway-Phase II</td>
<td>$\dot{e} = 4 \times 10^{-4} \frac{\text{e}^{0.010\text{D}_t}}{(1+0.001\text{D}_t)^{0.840}}$</td>
<td>$\dot{e} = 3 \times 10^{-4} \frac{\text{e}^{0.005\text{D}_t}}{(1+0.001\text{D}_t)^{0.870}}$</td>
</tr>
<tr>
<td>Babcock St.</td>
<td>$\dot{e} = 3 \times 10^{-4} \frac{\text{e}^{0.008\text{D}_t}}{(1+0.001\text{D}_t)^{0.956}}$</td>
<td>$\dot{e} = 2 \times 10^{-4} \frac{\text{e}^{0.006\text{D}_t}}{(1+0.001\text{D}_t)^{0.956}}$</td>
</tr>
<tr>
<td>Parkway-Phase I</td>
<td>$\dot{e} = 2 \times 10^{-4} \frac{\text{e}^{0.005\text{D}_t}}{(1+0.001\text{D}_t)^{0.870}}$</td>
<td>$\dot{e} = 1 \times 10^{-4} \frac{\text{e}^{0.003\text{D}_t}}{(1+0.001\text{D}_t)^{0.956}}$</td>
</tr>
<tr>
<td>U.S. Highway 1</td>
<td>$\dot{e} = 1 \times 10^{-4} \frac{\text{e}^{0.003\text{D}_t}}{(1+0.001\text{D}_t)^{0.956}}$</td>
<td>$\dot{e} = 1 \times 10^{-4} \frac{\text{e}^{0.003\text{D}_t}}{(1+0.001\text{D}_t)^{0.956}}$</td>
</tr>
</tbody>
</table>

FIG. 17b Predicted strain rates for subgrade soils.

### FIG. 16 Predicted strain rates for subgrade soils.
Discussion and Conclusions

The 5-min MPMT creep test was performed on different unbound pavement materials to assess time effects on their in situ creep behavior. The creep deformations of unbound granular materials are clearly identifiable, even though they are less than those that occurred in fine-grained soils. Generally, unbound pavement materials exhibit classical creep behavior defined by primary and secondary stages of creep deformations. Therefore, studying the creep response of unbound granular materials is essential to understanding the long-term performance of pavement systems.

The creep data obtained from MPMT tests were compared with those determined from laboratory one-dimensional creep tests. The time-dependent deformation model developed by Singh and Mitchell (1968) was used to describe creep data of these tests. The results showed that initial strain rates generated during field MPMT tests are similar to those observed during laboratory creep tests. However, the slopes (m) of the MPMT strain rate curves are lower than those from the creep tests. The lower m values indicate a higher creep potential. For subgrade soils, the m values obtained MPMT data ranged from 0.829 to 0.858, whereas those determined from creep data varied from 0.859 to 0.959. For base course materials, the m values from MPMT tests range from 0.831 to 0.869, whereas those from creep tests ranged from 0.840 to 0.986.

The creep stress parameters (A and z) were obtained by performing laboratory creep tests at three different stress levels (83, 124, 172 kPa). At the selected reference time of \( t_1 = 1 \) day, the average values of A and z were \( 3.0 \times 10^{-4} \) (%/min) and \( 55 \times 10^{-4} \) (1/kPa) for subgrade soils, and \( 2.0 \times 10^{-4} \) (%/min) and \( 51 \times 10^{-4} \) (1/kPa) for base course materials, respectively. The MPMT creep stress data indicate that the average values of A and z for subgrade soils are \( 4.0 \times 10^{-4} \) (%/min) and \( 17 \times 10^{-4} \) (1/kPa), respectively. Whereas the average values of A and z for base course layers are \( 3.5 \times 10^{-4} \) and \( 7 \times 10^{-4} \) (1/kPa), respectively. It can be concluded that both MPMT and creep tests yield the same initial strain rate (A); however, these tests produce different slopes of creep stress curves (z). However, it was found that the magnitude of z does not significantly affect the strain rate measurements.

Using the time-dependent models developed based on MPMT and laboratory creep data, the strain rates were projected for three planning pavement lives (1, 10, and 20 years). For subgrade soils, the average ratios of predicted to measured strain rate (MPMT/Creep) are 1.25 and 1.30 for 10, and 20 years, respectively. For base course materials, the average ratios of predicted to measured strain rate (MPMT/Creep) are 1.23, and 1.30 for 10 and 20 years, respectively. Even though that strain rates predicted from MPMT tests are larger than those estimated from standard laboratory one-dimensional creep tests, they would produce conservative creep estimates. The conservative results indicate that it would be a safe choice to use MPMT tests for determining long-term deformation of unbound pavement materials. The difference between MPMT and laboratory creep measurements is attributed to the differences in soil testing conditions. The creep behavior of unbound pavement materials is affected by many factors including density, moisture content, mass and size of soil samples, stress levels, and duration of applied load. In addition, the soils are allowed to deform freely during the MPMT creep test, whereas laboratory creep specimens are confined in the horizontal direction, and allowed to displace only in the vertical direction.

ACKNOWLEDGMENTS

The writers acknowledge the Florida Department of Transportation (FDOT) for their support with this research. The writers also thank Target Engineering Group for their cooperation during the field testing.

References

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