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Published in:
Proceedings of the 3rd International Conference on Accelerated Pavement Testing

Publication date:
2008

Document Version
Peer reviewed version

Citation (APA):
Backcalculation of Layer Moduli using Time History of Embedded Gauge Readings

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ABSTRACT

A prototype-scale asphalt pavement was constructed in the Purdue/INDOT accelerated pavement testing facility. The test section was loaded under constant temperature conditions with a half-axle dual-wheel assembly. During construction, the pavement was instrumented with stress and strain gauges located along the wheel path and outside the wheel path at different depths. As the loading assembly traversed the test section, the response of the pavement system was monitored by the embedded gauges. In this paper, the time history of the readings is presented and subsequently utilized to backcalculate the resilient mechanical properties of the different layers. For this purpose the pavement system is modeled as a linear-elastic layered medium. Inertial (dynamic) effects are disregarded, and the quasi-static approach is applied for simulating the moving load. The analysis is performed for the pavement after it had incurred 5,000 load applications and separately after 80,000 load applications. Material properties in both cases are presented and compared. Results are also discussed in light of complex modulus testing and Falling Weight Deflectometer data.

Keywords: backcalculation, linear elasticity, resilient response, time history.

Conference Topic: Modeling and analysis of pavement systems.

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INTRODUCTION

The National Center for Asphalt Technology (NCAT) operates a full-scale test road for studying the response and performance of asphalt pavements [1]. During the 2003-2005 testing phase, NCAT instrumented eight of their test sections with stress and strain gauges [2, 3, 4]. Two of the test sections (denoted by N1 and N2), were later replicated, along with embedded instrumentation, in the Purdue/INDOT accelerated pavement testing (APT) facility [5]. The availability of instrumented pavements having similar structures but loaded in completely different conditions offered a unique research opportunity to develop and validate a mechanistic pavement model.

Subsequent research efforts were focused on resilient (recoverable) response and included the following tasks: (i) development and calibration of a mechanistic pavement model based on APT data only; (ii) analysis of laboratory tests done on the individual materials such as complex modulus and resilient modulus; (iii) formulation of a methodology to extend the capabilities of the APT model to apply to other loading configurations, other loading speeds, and different environmental conditions; and (iv) application of the derived model to simulate the loading and environment at NCAT in order to assess its forecastability.

This paper deals mainly with task (i) and is limited to the analysis of section N1 only. First, the APT experiment is presented, providing an overview of the facility and description of the pavement structure. The instrumentation plan is also presented and discussed along with the raw data collected at two different stages during the test. Next, the development and calibration procedure of a mechanistic pavement model is discussed. The pavement system is modeled using layered elastic theory (LET), and the material properties (layer moduli) are obtained through backcalculation. The analysis is performed twice, separately for each of the two test stages, using the time history of the gauge readings collected during one pass of the APT carriage. Finally, pavement properties in both cases are presented and compared. Results are also discussed in light of complex modulus tests performed in task (ii) and backcalculated moduli obtained from analysis of Falling Weight Deflectometer (FWD) data.
THE APT EXPERIMENT

General Overview
The Purdue/INDOT APT facility was fabricated in late 1990 by Purdue University for the Indiana Department of Transportation [6, 7, 8]. The APT is housed in a temperature controlled hangar. The testing area consists of a pit for accommodating prototype-scale pavements. The pit is embedded in the concrete floor; it is 6 ft (1.83 m) deep and shaped as a square with 20 ft (6.1 m) long sides. Typically, different structures are constructed in the pit, each 20 ft (6.1 m) long. The width of each section depends on the experimental configuration. The N1 section addressed herein was 10 ft (3.05 m) wide. The APT system is capable of loading either a dual-wheel assembly or a super single wheel up to 20,000 lb (9,080 kg) with a maximum speed of 5 mph (2.235 m/s). Passes can be applied in the same wheel path repeatedly or with wander. They can also be applied in either one or two directions. For this study passes were applied in unidirectional mode without wander via a dual-wheel assembly loaded to 15,000 lb (6,810 kg) and traveling at maximum speed. The temperature of the pavement layers was set to 60ºF (15.5ºC).

Pavement Structure and Properties
The structural section is shown in Figure 1. It is comprised of five inches (127 mm) of hot mix asphalt (HMA) overlaying six inches (152.4 mm) of crushed granite aggregate base course, placed on top of an A-4(0) soil serving as subgrade. The HMA course was made of two mixes and constructed in three lifts. The surface lift (Mix 1) was one inch (25.4 mm) thick while the intermediate and bottom lifts (Mix 2) were each two inches (50.8 mm) thick. Both HMA mix types were designed according to the Superpave system with 80 gyrations using PG 76-22 (SBS modified) binder. Their nominal maximum aggregate size differed: 9.5 and 19.0 mm for mixes 1 and 2 respectively. The average as-constructed air void content for the three lifts was as follows (top to bottom): 7.5%, 10.2% and 9.1%. The aggregate base was compacted to 97% in a single lift. The subgrade soil was compacted in lifts, each up to six inches thick (152 mm). The top 19 inches (0.48 m) were compacted to an average of 93% while the lower subgrade portion was compacted to 90%. The aforementioned compaction degrees are based on modified Proctor energies and dry densities.
Additional details on the overall project planning and pavement construction process can be found in [5, 9].

An FWD test was conducted in the APT before loading began. The FWD loading plate was 11.81 inches (300 mm) in diameter. A set of nine geophones was used, located at the following offset distances from the center of the plate: 0, 8, 12, 18, 24, 36, 48, 60 and 72 inches (0, 0.20, 0.30, 0.46, 0.61, 0.91, 1.22, 1.52 and 1.83 m). For a peak stress level of 105 psi (0.72 MPa), i.e., peak load level of 11,500 lb (5,221 kg), the corresponding peak FWD deflections were as follows: 33.73, 24.92, 18.76, 7.96, 3.61, 2.10, 1.41, 1.07 and 0.88 mils (857, 633, 477, 202, 92, 53, 36, 27 and 22 microns). The pavement surface temperature during the test was 86.5°F (30.3°C).

Additionally, laboratory complex modulus tests were performed on the two HMA types used in the experiment [3, 10]. Specimens were fabricated from reheated loose mix samples which were compacted using Gyratory equipment to an air void content of about 7%. Dynamic modulus master curves were constructed for the reference temperature prevailing during the APT experiment, i.e., 60°F (15.5°C). Their method of construction was slightly different than usual because the time-temperature shifting was performed (separately for each mix) considering both phase angle and dynamic modulus data - simultaneously [11]. The two resulting dynamic modulus master curves are shown in Figure 2 for reduced frequencies in the range of $10^{-7}$ to $10^{+7}$ Hz. The modulus ratio between the two mixes within this frequency range is shown on the right ordinate. It may be seen that Mix 2 is consistently stiffer than Mix 1 by about 25 to 50%.

**Embedded Instrumentation**

A plan showing the embedded instrumentation in the test section is provided in Figure 3. The loading centerline is denoted in the figure by the Y-axis and the transverse direction by the X-axis. The loading direction was from left to right along the Y-axis as indicated by the arrow. The entire gauge array is seen to be located in an eight foot (2.44 m) long strip, 2 ft (0.61 m) wide, at the central part of the test section. The first and last six feet (1.83 m) of the test section were not instrumented because the loading speed in these areas is not constant, with carriage either accelerating or decelerating. In the central strip the loads are applied at a constant speed. In correspondence to Figure 3, Table 1 lists the location of each gauge.
As can be seen in Figure 3 (and Table 1), the pavement system was instrumented with a total of 12 gauges consisting of four pressure gauges and eight strain gauges. The two pressure gauges (#1178 and #1185) were measuring vertical stresses on top of the base course or bottom of the HMA course. These were installed at a depth of five inches (127 mm) from the surface along the centerline of the loading path (i.e., Y-axis). Two additional pressure gauges (#1179 and #1184) were measuring vertical stresses on top of the subgrade or bottom of the base course. These were installed at a depth of 11.0 inches (279.4 mm) from the surface, also along the centerline. Pressure cells were manufactured by Geokon (model 3500).

All eight strain gauges were attached to the bottom of the HMA course, i.e., at a depth of five inches (127 mm) from the surface. Gauges G-1, G-2, G-3 and G-4 were located along the centerline of the loading path. Strain gauges G-5, G-6, G-7 and G-8 were located along a parallel line positioned two feet (0.61 m) from the loading centerline. Gauges G-2, G-4, G-5 and G-7 were measuring the horizontal strains in the loading direction (i.e., strain in Y) while gauges G-1, G-3, G-6 and G-8 were measuring the horizontal strains in the transverse direction (i.e., strain in X). Strain gauges were manufactured by CTL Group (model ASG-152).

**Measured Resilient Response**

Loading in the APT began on July 19th, 2004. By August 11th 2004, 90,000 passes had been applied to the pavement. The experiment was stopped due to bond failure that occurred at the interface between the surface and intermediate HMA lifts. At the onset of failure, the surface lift in the wheel path area was sheared-off in the direction of loading, exposing the intermediate HMA lift. Figures 4, 5 and 6 show the resilient strains and stresses measured during APT passes 5,000 and 80,000 vs. the APT carriage location which corresponds to the Y-axis in Figure 3. In each of these figures, a solid line represents pass #5,000 and a dashed line represents pass #80,000. As is customary for geomaterials, a positive sign indicates compression and a negative sign indicates tension of either stress or strain.

Figure 4 presents the vertical stresses measured on top of the subgrade and on top of the base course by the four pressure gauges. As can be seen, the resulting curves are bell-shaped and nearly symmetric. For loading pass #5,000, peak vertical stresses on top of the base course were 30 and 35 psi (0.21 and 0.24 MPa). On top of the subgrade, the measured peak stresses were 16 and 20 psi (0.11 and 0.14 MPa). In
theory the readings of each gauge pair should be identical. Furthermore, it may be seen that peak vertical stresses during pass #80,000 are slightly higher compared to pass #5,000. The difference is more significant in both absolute and relative terms for the gauges located on top of the subgrade compared to those located on top of the base course. It should be noted that the stress peaks occur slightly after the APT carriage had passed over the gauges and moved further along by about 2 to 5 inches (51 to 127 mm).

Figure 5 presents the measured horizontal strains at the bottom of the HMA course in the direction of loading. Four gauge readings are shown, two of which were located along the loading centerline (G-2 and G-4), and two were located along a parallel line (G-5 and G-7) that is offset by two feet (0.61 m) - see Figure 3 and Table 1. In all four cases it can be seen that as the load approaches a gauge, the bottom of the HMA goes into compression. Then, the strain direction is reversed and the gauges go into tension. The point of maximum tension occurs when the APT carriage had passed the gauge positions along the Y-axis by about 1 to 3 inches (25 to 76 mm). Finally, when the load is receding (APT carriage moves further along), the tensile strains are reversed and compression is induced once more at the bottom of the HMA. This pattern is more pronounced for the gauges aligned along the centerline (G-2 and G-4).

It can be graphically seen that the approaching branch of the strain response is different from the receding branch, resulting in a non-symmetrical time history curve. The two most noticeable differences are: (a) peak compressive strain is usually higher in the approaching branch compared to the receding branch; and (b) the spacing along the Y-axis between the tension and compression strain peaks is larger in the receding branch compared to the approaching curve.

Referring to the approaching branch of pass #5,000 for gauges G-2 and G-4 (both centerline gauges), peak strains were 84 and 119 microstrains in compression and 431 and 314 microstrains in tension (respectively). For gauges G-5 and G-7 (both off-center gauges) the peak compressive strains were 2 and 15 microstrains while the peak tensile strains were 50 and 100 microstrains (respectively). In theory, the readings from each gauge pair should be identical. When comparing the response between pass #5,000 and pass #80,000 the most noticeable difference is seen in the peak tensile strain magnitudes for gauges G-2 and G-4 (the centerline gauges). For the G-2 strain gauge, peak strain in tension during pass #80,000 is 380 microstrains.
(compared to 431 microstrains during pass #5,000). For G-4 gauge the peak strain in tension during pass #80,000 is 163 microstrains (compared to 314 microstrains during pass #5,000).

Figure 6 presents the measured horizontal strains at the bottom of the HMA course in the transverse direction relative to the loading centerline. In this case the centerline gauges behave differently compared to the off-center gauges. Referring to pass #5,000 data, it can be seen that gauges G-1 and G-3 (centerline gauges) go into tension as the load is approaching, with peak strains of 108 and 154 microstrains respectively. Contrary to the previous two figures, these peaks occur 4 to 6 inches (102 to 152 mm) before the APT carriage reaches the gauge. As the APT carriage passes the gauges and moves further along, the strain direction is reversed until a small level of compression is induced. This compressive strain slowly recovers during the time period (not shown in the figure) in which the APT load is lifted from pavement and moved back to the startup position. Gauges G-6 and G-8 (off-center gauges) go into compression as the APT carriage approaches, with peak strains of 103 and 152 microstrains respectively. These peaks, however, occur 3 to 5 inches (76 to 127 mm) after the load had passed each gauge. The receding branch of the response shows that the strain direction is reversed until a small level of tension is induced in the gauges.

The response of the gauges, as seen in Figure 6, is very confusing. First, the G-3 gauge shows two peaks instead of one similar to the rest of the gauges. Next, when comparing the response between pass #5,000 and pass #80,000 the trends are not uniform: (a) it seems that the response of the G-3 gauge has shifted (delayed), as if the gauge was physically moved a few inches along the Y-axis during the experiment; and (b) the peak strains decreased during the test for gauges G-1, G-3 and G-6 but not for the G-8 gauge. At this point we do not have a good explanation for these obscure behaviors.
MODEL DEVELOPMENT AND CALIBRATION

Layered Elastic Program
Since its introduction by Burmister [12, 13], LET has been used by engineers and researchers for representing the resilient response of asphalt pavement systems. LET also serves as the main “engine” for the Mechanistic Empirical Pavement Design Guide (MEPDG) through the JULEA computer code [14, 15]. As a first approximation, the APT test section is modeled herein using LET. Accordingly, materials are assumed linear elastic, homogeneous, isotropic and weightless. Each layer is characterized by an elastic (Young’s) modulus \( E_i \) and a Poisson’s ratio \( \nu_i \). The subscript \( i \) identifies the specific layer with \( i = 1 \) at the top.

Using the non-dimensional derivation given by Huang [16], the governing equations for a five layered system, assuming fully bonded interfaces, were programmed into an Excel worksheet*. The numerical integration required within the calculation procedure was carried out between the first 200 zeros of the Bessel functions involved. The Gauss integration scheme was used for this purpose whereby the first interval was integrated using 30-point Gaussian formula, the second interval was integrated using 20-point formula, the third interval was integrated using ten-point formula and the remaining intervals were integrated using a five-point formula. In order to speed the computational time, the number of matrix inversions was limited to 96, corresponding to 96 predetermined values of the Hankel transform variable (or integration variable) in the range of 0 to 50,000. A cubic spline interpolation scheme was used to derive intermediate results within this range. Furthermore, in order to improve the convergence of the integration, especially for points residing close to the surface, one step of Richardson extrapolation was employed [17].

Backcalculation using Gauge Readings
It is well recognized that pavement materials do not comply with LET assumptions. The resilient response of asphalt mixes is known to be nonlinear viscoelastic [18, 19, 20]. The resilient response of unbound layers is nonlinear elastic and stress-state sensitive [21, 22]. Consequently, a systematic error is introduced into the analysis when using LET. In an effort to minimize this error, the model parameters (i.e., elastic

* program available upon request from the corresponding author.
moduli) are derived through a process of backcalculation using the time history of embedded gauge readings. Using this approach, subsequent stresses, strains and deflections calculated with the calibrated model will resemble measured responses even though the model assumptions are over-simplified. In this connection it should be noted that LET cannot inherently simulate certain features that were seen in the experiment [23, 24]. One example refers to the offset observed between peak responses and load location (in LET they must coincide). Another example is the non-symmetric response relative to the load location, i.e., the differences between approaching and receding curves as recorded by the gauges (in LET the response is symmetric).

For performing the backcalculation, the pavement system was represented using four layers. The three HMA lifts were combined into one (top) layer, five inches (127 mm) thick with $\nu_1 = 0.3$. The second layer from the top represents the crushed aggregate base course, with a thickness of six inches (152.4 mm) and $\nu_2 = 0.35$. Because no instrumentation was embedded in the subgrade (only on top), there is no available data to support its sub-layering. Hence, the upper and lower subgrade layers were combined into one layer (third layer from the top) having a total thickness of 61 inches (1.55 m) and $\nu_3 = 0.40$. The fourth and final layer, with semi-infinite thickness, represents the concrete floor of the test pit. The elastic properties of this layer are fixed to the following values: $E_4 = 4,000,000$ psi (27,560 MPa) and $\nu_4 = 0.2$. The dual-wheel loading was simulated by two circular areas, each eight inches (203 mm) in diameter, transferring uniform vertical stresses of 150 psi (1.03 MPa) to the pavement surface. The spacing between the centers of the loads is 13.5 inches (343 mm). For simulating the moving APT carriage, the quasi-static approach was applied in which dynamic (inertial) effects are disregarded. This assumption seems reasonable because of the relatively slow loading speeds in the APT.

Generated model responses were compared to measured responses and a nonlinear optimization algorithm (generalized reduced gradient [25]) was applied to manipulate the material properties until best fit was achieved. This process was repeated twice to separately analyze the structure during pass #5,000 and during pass #80,000. Due to the non-symmetric strain response of the pavement (refer to previous discussion), only data from the approaching branch was used for the comparison. Subsequently, 25 data points were pre-selected from each time history, corresponding
to 25 different APT carriage positions relative to the gauge location (with denser spacing closer to the gauge). These “offset” distances ranged between 70 inches (1.78 m), for which readings were negligible, and zero, in which the APT carriage was exactly in line with the gauge along the Y-axis.

Regardless of the number of data points used for the comparison between model and experiment there were only three moduli that needed to be backcalculated (for a given pass level), namely the HMA modulus ($E_1$), the aggregate base modulus ($E_2$), and the subgrade modulus ($E_3$). In order to derive the numerical values of these moduli, an objective (scalar) function describing the agreement between the model and test data was formulated. First, for each gauge separately (out of the total twelve gauges available) an error term was defined as follows:

$$ ERR_g = \frac{1}{N} \sqrt{\sum_{n=1}^{N} \left[ R_{n}^{\text{APT}} - R_{n}^{\text{model}} \right]^2} $$

(1)

in which $N$ is the number of data points used for the comparison for the $g^{th}$ gauge (i.e., $N = 25$). $R^{\text{APT}}$ represents the measured APT response of either stress or strain and $R^{\text{model}}$ is the corresponding LET response. Note that $ERR_g$ has the same units as $R^{\text{APT}}$ (or equivalently $R^{\text{model}}$) and is always positive. Next, these individual errors were combined to formulate a unitless global error term, defined as follows:

$$ \text{Global Error} = \frac{1}{G} \cdot \sum_{g=1}^{G} \left[ \frac{ERR_g}{\min(\text{ERR}_g)} - 1 \right] $$

(2)

where $G$ is the total number of gauges considered in the analysis (i.e., $G = 12$), and $\min(\text{ERR}_g)$ represents the lowest achievable error between the model and the test data for the $g^{th}$ gauge. The numerical value of $\min(\text{ERR}_g)$ was obtained by employing an over-fitting technique, i.e., the layer moduli were first manipulated using the optimization algorithm in an effort to separately minimize each of the individual errors (equation 1). Note that $\min(\text{ERR}_g)$ is always greater than zero, because even if the model was perfect, all test data contains some random “noise”. However, the global error term can, in principal, equal zero. This situation occurs mathematically when all individual errors are minimal. Therefore, equation 2 serves as a weighted average of the individual errors, making sure that neither of the gauge
readings is underweighted or overweighted in the backcalculation process. In order to enable a direct comparison between the global error for pass #5,000 and pass #80,000, the list of $\text{min}(ERR_y)$ values obtained for pass #5,000 were also used for the backcalculation of pass #80,000.
RESULTS AND DISCUSSION

Table 2 presents the backcalculated layer moduli for pass #5,000 and pass #80,000. The global error term (equation 2) was 4.89% for pass #5,000 and 6.27% for pass #80,000. In both cases it can be seen that the stiffness of the pavement structure is decreasing from top to bottom. During pass #5,000 the HMA is 14.6 times stiffer than the underlying aggregate base. The aggregate base is seen to be twice as stiff as the subgrade. By comparing these results with pass #80,000, it is clear that during the APT experiment the individual layer moduli increased: (a) the HMA experienced a slight stiffness increase of about 8.5%; (b) the stiffness of the base increased significantly by about 54%; and (c) the subgrade increased in stiffness by about 16.5%. Subsequently, the relative stiffness within the structure also changed with the HMA ending up 10.3 times stiffer than the underlying base, and the base becoming 2.6 times stiffer than the subgrade. In lieu of direct test data, these changes are believed to be the result of further densification under load, especially of the unbound materials.

Figures 7 and 8 show both the measured and calibrated model responses for pass #5,000 and #80,000 (respectively) vs. offset (or distance) from the gauge. Each figure contains six charts. The two topmost charts show horizontal strains in X (left) and in Y (right) for gauges located along the loading centerline. The charts in the middle show horizontal strains in X (left) and in Y (right) for gauges positioned outside the loading path. The bottom charts show vertical stresses as measured by pressure cells located on top of the base (left) and on top of the subgrade (right). In each chart the measured gauge data is represented by solid markers. Because the pavement was instrumented with pairs of gauges measuring the same response, two types of markers are used. The calibrated model responses are shown using a solid line.

These figures provide some intuition and information on several experimental and modeling aspects. First, the large difference between measured responses of the gauge pairs is demonstrated. Graphically, these differences seem to be smaller for the stress measurements compared to the strain readings. During pass #5,000 the maximum relative difference in the peak stress readings with respect to the average reading at the peak is 10.6% (for pressure gauges 1178 and 1185). Similarly, the
maximum relative difference in the peak strain readings with respect to their average at the peak is 35.0% (for strain gauges G5 and G7). During pass #80,000 the corresponding differences are 10.8% (again, pressure gauges 1178 and 1185) and 44.7% (strain gauges G6 and G8). These differences are believed to represent both structural heterogeneity and slight dissimilarity in gauge installation conditions.

Next, the goodness of fit of the calibrated model can be visualized. It may be graphically seen that LET captures relatively well the vertical stresses and also the horizontal strains in X and Y directions for the off-center gauges. For the centerline gauges, the horizontal strains are captured relatively well only in the direction of loading. The fit is not very good for the strains in the transverse direction. The above findings, however, should not be expected to hold in general. In other cases, the stress dependence of the unbound layers, and perhaps even anisotropy, may impair the theory’s forecastability. Also, the ability to successfully use LET is likely to diminish when the pavement structure is comprised of thicker HMA layers. In this case, the HMA’s time dependence will be more dominant and the non-symmetry in the strain and stress response within the structure will be more pronounced.

A modulus of 350 ksi (2,412 MPa) was backcalculated for the HMA course during pass #5,000 (see Table 2). Using the dynamic modulus master curves in Figure 2, this value is seen to be associated with a reduced frequency of 0.084 Hz in the case of Mix 1 and 0.018 Hz for Mix 2. As a discussion point, it is interesting to compare the HMA frequency and corresponding modulus that the MEPDG would provide for a similar structure and loading conditions. Currently, HMA master curves are used in the MEPDG to generate elastic moduli. The HMA course is sub-layered and an effective loading “frequency” is computed for each sub-layer based on the vehicle speed, radius of loaded area, depth from the surface, temperature profile, level of aging and subgrade stiffness (see appendix CC-3 in [14]). This frequency is then used to generate modulus values within the HMA course from the corresponding master curves.

Such an analysis was carried out herein for the APT conditions, i.e., no aging, uniform temperature profile, vehicle speed of 5 mph (2.235 m/s), etc. The HMA course was divided into 6 sub-layers with the following thicknesses: 0.5, 0.5, 1.0, 1.0, 1.0 and 1.0 inches (12.7, 12.7, 25.4, 25.4, 25.4 and 25.4 mm). The computed frequencies and moduli for each sub-layer are given in Table 3. As expected, different frequencies and moduli are assigned by the MEPDG to different evaluation depths.
The highest frequency (8.63 Hz) is assigned to the top sub-layer of the HMA course based on evaluation depth of 0.25 inches (6.35 mm). The lowest frequency (1.81 Hz) is assigned to the bottom sub-layer based on evaluation depth of 4.50 inches (114.3 mm). Note that these frequencies are consistently larger than those obtained from backcalculation by about two orders of magnitude. Consequently, the corresponding moduli are also larger than 350 ksi (2,412 MPa) by a factor of about 3. A similar conclusion was reached in [24] after analyzing an asphalt pavement structure using the Finite Element Method with linear viscoelastic theory.

Another point for discussion is related to analysis of FWD data. The available deflections were analyzed with the LET program developed in this study. The four-layered structural model was utilized again with similar pre-selected Poisson’s ratios and fixed elastic properties for the underlying concrete floor, as before. The FWD moduli were obtained in the usual way whereby the agreement between measured and modeled deflections was minimized using an optimization algorithm [25]. It is important to note that the objective function for minimization in this case was defined based on absolute relative errors. If \( d_{FWD}^j \) denotes measured FWD deflection at the \( j^{th} \) geophone and \( d_{model}^j \) denotes the corresponding calculated deflection (using LET), then the objective function is of the form:

\[
FWD_{\text{Error}} = \frac{1}{J} \sum_{j=1}^{J} \left| \frac{d_{FWD}^j - d_{model}^j}{d_{FWD}^j} \right|
\]

in which \( J \) is the total number of geophones used for the comparison (herein, \( J = 9 \)).

Derived FWD moduli were as follows: \( E_1(\text{HMA}) = 100,000 \) psi (689 MPa), \( E_2(\text{Base}) = 21,000 \) psi (145 MPa), and \( E_3(\text{Subgrade}) = 14,500 \) psi (100 MPa). The fitting error (equation 3) was 30.2% with a maximum (absolute) difference between measured and calculated deflection of 4.6 mils (116.8 microns) for the geophone located 12 inches (0.30 m) from the center. As can be seen, the base and subgrade FWD moduli compare relatively well with those in Table 2 for pass #5,000. The HMA modulus is 3.5 times smaller partially because the FWD test was done at a higher temperature, 86.5°F (30.3°C) instead of 60°F (15.5 °C).

When a different objective function was used to represent the level of agreement between measured and calculated deflection basin, based on squared errors or absolute errors, the derived moduli were completely different with the subgrade
layer being stiffer than the crushed aggregate base and the HMA modulus 2.5 times higher. For instance, using an objective function based of squared errors, the FWD moduli would have been: $E_1$(HMA) = 250,000 psi (1,722 MPa), $E_2$(Base) = 3,400 psi (23.4 MPa), and $E_3$(Subgrade) = 27,000 psi (186 MPa). Hence, the availability of a calibrated LET model based on gauge readings supports the use of absolute relative errors for FWD data analysis.
SUMMARY

The resilient response to a moving load under constant temperature conditions was studied in the Purdue/INDOT APT facility. The paper focused on the pavement in the initial phase of the experiment, after 5,000 load applications, and also after 80,000 load applications. Measured responses were presented and discussed for the two different experimental stages. It was found that: (a) large differences were recorded between pairs of gauges that should have provided identical readings. Maximum relative differences at peak responses with respect to the average reading at the peak was about 11% for the stress gauges and as high as 45% for the strain gauges; (b) resilient response is non-symmetric with respect to the load location, i.e., the approaching curves where different in shape and shorter in duration compared to the corresponding receding curves; (c) peak stresses and strains were usually seen only after the load had passed over the gauge and moved further along; and (d) the resilient response of the pavement changed during the experiment whereby peak levels of stress or strain measured by a given gauge differed between the initial and final testing phases. In most cases, there was an increase in peak vertical stress levels measured on top of the subgrade and base, and a decrease in peak horizontal strain levels measured at the bottom of the HMA.

The pavement was modeled using LET with material properties derived through a process of backcalculation. The analysis was performed twice, separately for each experimental stage, using the time history of the all gauge readings collected during one pass of the APT carriage. Due to the non-symmetric response of the pavement, only data from the approaching curves was used for the comparison. In general, the calibrated model captured relatively well the trends in the test data. Moreover, backcalculated moduli showed that during the experiment the pavement increased its stiffness. Most significant was the increase in base stiffness, which was more than 50% higher after 80,000 load applications compared to the initial conditions. Further densification under load is believed to be the main cause for these changes.

Using the calibrated LET model in conjunction with dynamic modulus master curves it was found that the MEPDG methodology would have overestimated the modulus of the HMA course by a factor of 3. Additionally, it was found that
backcalculated moduli based on FWD deflections could provide similar results provided that the level of agreement between measured and computed deflections is expressed in terms of absolute relative errors.
REFERENCES


TABLE 1 Location of APT instrumentation (relate to Figure 3).

<table>
<thead>
<tr>
<th>Gauge ID</th>
<th>Gauge Type</th>
<th>Location in X, in. (m)</th>
<th>Location in Y, in. (m)</th>
<th>Depth in Z, in. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1178</td>
<td>Pressure Cell</td>
<td>0.0</td>
<td>72 (1.83)</td>
<td>5 (127)</td>
</tr>
<tr>
<td># 1179</td>
<td>Pressure Cell</td>
<td>0.0</td>
<td>96 (2.44)</td>
<td>11 (279)</td>
</tr>
<tr>
<td>#1184</td>
<td>Pressure Cell</td>
<td>0.0</td>
<td>144 (3.66)</td>
<td></td>
</tr>
<tr>
<td>#1185</td>
<td>Pressure Cell</td>
<td>0.0</td>
<td>168 (4.27)</td>
<td></td>
</tr>
<tr>
<td>G-1</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>132 (3.35)</td>
<td>5 (127)</td>
</tr>
<tr>
<td>G-2</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>156 (3.96)</td>
<td></td>
</tr>
<tr>
<td>G-3</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>108 (2.74)</td>
<td></td>
</tr>
<tr>
<td>G-4</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>84 (2.13)</td>
<td></td>
</tr>
<tr>
<td>G-5</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>156 (3.96)</td>
<td></td>
</tr>
<tr>
<td>G-6</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>108 (2.74)</td>
<td></td>
</tr>
<tr>
<td>G-7</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>132 (3.35)</td>
<td></td>
</tr>
<tr>
<td>G-8</td>
<td>Strain Gauge</td>
<td>24 (6.1)</td>
<td>156 (3.96)</td>
<td></td>
</tr>
<tr>
<td>#</td>
<td>Layer</td>
<td>Thickness, in. (mm)</td>
<td>Poisson’s Ratio</td>
<td>Pass #5,000</td>
</tr>
<tr>
<td>----</td>
<td>---------------</td>
<td>---------------------</td>
<td>-----------------</td>
<td>-------------------</td>
</tr>
<tr>
<td>1</td>
<td>HMA</td>
<td>5 (127)</td>
<td>0.30</td>
<td>350,000 (2,412)</td>
</tr>
<tr>
<td>2</td>
<td>Base</td>
<td>6 (152)</td>
<td>0.35</td>
<td>24,000 (165)</td>
</tr>
<tr>
<td>3</td>
<td>Subgrade</td>
<td>61 (1,549)</td>
<td>0.40</td>
<td>12,000 (83)</td>
</tr>
<tr>
<td>4</td>
<td>Concrete</td>
<td>Semi-infinite</td>
<td>0.20</td>
<td>4,000,000 (27,560)</td>
</tr>
</tbody>
</table>

TABLE 2 Backcalculated layer moduli for pass #5,000 and pass #80,000.
<table>
<thead>
<tr>
<th>HMA Type</th>
<th>Sub-layer Thickness, in. (mm)</th>
<th>Evaluation Depth, in. (mm)</th>
<th>Effective Frequency [Hz]</th>
<th>HMA Modulus, ksi (MPa) @ 60°F (15.5°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix 1</td>
<td>0.5 (12.7)</td>
<td>0.25 (6.35)</td>
<td>8.63</td>
<td>1,014 (6,985)</td>
</tr>
<tr>
<td></td>
<td>0.5 (12.7)</td>
<td>0.75 (19.05)</td>
<td>6.06</td>
<td>943 (6,498)</td>
</tr>
<tr>
<td>Mix 2</td>
<td>1.0 (25.4)</td>
<td>1.50 (38.10)</td>
<td>4.12</td>
<td>1,221 (8,416)</td>
</tr>
<tr>
<td></td>
<td>1.0 (25.4)</td>
<td>2.50 (63.50)</td>
<td>2.88</td>
<td>1,136 (7,825)</td>
</tr>
<tr>
<td></td>
<td>1.0 (25.4)</td>
<td>3.50 (88.90)</td>
<td>2.22</td>
<td>1,076 (7,417)</td>
</tr>
<tr>
<td></td>
<td>1.0 (25.4)</td>
<td>4.50 (114.3)</td>
<td>1.81</td>
<td>1,032 (7,108)</td>
</tr>
</tbody>
</table>

TABLE 3 Effective frequencies and corresponding moduli calculated for the APT experimental conditions using MEPDG methodology.
### FIGURES

<table>
<thead>
<tr>
<th>FIGURE 1 Pavement structure in APT.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>5 in.</strong></td>
</tr>
</tbody>
</table>
| HMA | Mix 1: NMAS 9.5 mm; PG 76-22, AV-C 7.5%  
Mix 2: NMAS 19 mm; PG 76-22, AV-C 10.2%  
Mix 2: NMAS 19 mm; PG 78-22, AV-C 9.1%  |
| **6 in.** |
| Aggregate Base | Crushed granite material: 97% compaction; Water content: 3.5% |
| **19 in.** |
| Upper Subgrade | A-4(0) soil: 93% compaction; w=14.7% |
| **Lower Subgrade** | 90% compaction; w=14.7% |
FIGURE 2 Master curves @ 60°F (15.5°C) for Mix 1 (square markers) and Mix 2 (circular markers) and modulus ratio $E_2/E_1$ (dashed line).
FIGURE 3 Plan of embedded instrumentation in APT.
FIGURE 4 Measured vertical stresses in the APT on top of the base and on top of the subgrade during pass #5,000 (solid line) and pass #80,000 (dashed line).
FIGURE 5 Measured horizontal strains at the bottom of the HMA in the direction of loading during pass #5,000 (solid line) and pass #80,000 (dashed line).
FIGURE 6 Measured horizontal strains at the bottom of the HMA in the transverse direction to the loading during pass #5,000 (solid line) and pass #80,000 (dashed line).
FIGURE 7 Resilient responses during pass #5,000. Both measured (solid markers) and model generated (solid line) are shown.
FIGURE 8 Resilient responses during pass #80,000. Both measured (solid markers) and model generated (solid line) are shown.