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ABOUT GAP 2019

Geostructural Aspects of Pavements, Airfields, and Railways is an interdisciplinary transportation engineering forum designed to foster communication between all three primary transportation sectors. The collaborative environment is supported by public agencies and private sector companies, which together advance pavement, railway, and embankment engineering technologies.

The main objectives of GAP 2019 are to:

- Foster communication between the main transportation sectors
- Generate cooperation between the pavement and geotechnical disciplines
- Advance pavement and embankment engineering technologies

The event is an invaluable opportunity to develop better technical exchange of solutions to shared issues in our aging transportation infrastructure. A special emphasis has been placed on including technical sessions, panel discussions, and short courses that encourage contribution and learning for the participants from three modes of transportation.
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Permanent Deformation and Stiffness of Fouled Ballast Based on Static and Impact Load Tests

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Tensar International Corporation

Soheil Nazarian
Center for Transportation Infrastructure Systems, The University of Texas at El Paso

ABSTRACT

Fouling due to contamination with fines results in the degradation of mechanical properties of ballast, and in advance stages to an unstable railway track. Seasonal dry-wet cycles increase the severity of the situation. This paper aims at understanding the effects of ballast fouling on the permanent deformation and stiffness of railway track. To that end, about a dozen large-scale specimens were constructed in the laboratory to simulate railway track sections. Two types of fouling agents – rock dust with a low plasticity index (PI) and clay with a high PI – were used to contaminate the ballast specimens. To simulate the dry-wet-dry cycles, the dry contaminated ballast specimens were saturated and then left to dry for a few days. The behavior of ballast was measured with the static load tests using a Material Testing and Simulation (MTS) system and with the impact load test using a Portable Seismic Property Analyzer (PSPA) and Light Weight Deflectometer (LWD). The ballast specimens fouled with clay were highly susceptible to permanent deformation and loss of stiffness. The permanent deformation of clay fouled specimens increased and the stiffness decreased with the increase in the moisture content and the degree of fouling.

1. INTRODUCTION

Several well-documented railway incidents (Bailey et al. 2011; NTSB 2013) have been attributed to unstable tracks caused by the fouled ballast. The migration of subgrade soil into the ballast due to the dynamic train loads, crushing of ballast aggregates with time and climatic fluctuation contribute to ballast fouling (Anderson and Rose 2008; Parsons et al. 2012). Ballast fouling has a decremental effect on the mechanical properties of ballast as well (Huang et al. 2009; Duong et al. 2013; Koohmishi and Palassi 2017).

Huang et al. (2009) discussed different phases of ballast fouling and their relationships with the void spaces within the ballast. The structural stability of a clean ballast depends on the degree of contact between the aggregates. The void space within the aggregate cluster of the clean ballast also contributes to free-draining feature of ballast. With the increase in fouling, the filled voids expand and result in the loss of contact among aggregates. The major consequence of a fouled ballast is track...
deformation due to train loads. An increase in the moisture content of the fouled ballast causes a significant reduction in the strength and stiffness since the fouled material acts as a lubricant.

Selig and Waters (1994) reported that 76% of fouling was caused by ballast breakdown, 13% by infiltration from sub-ballast, 7% by infiltration from the ballast surface, 3% from subgrade intrusion, and 1% is related to tie wear. They proposed the following two terminologies for classifying fouled ballast:

- Fouling percentage = ratio of dry weight of material passing 3/8 in. sieve to dry weight of total sample, and
- Fouling index = sum of the percentage of materials passing through No. 4 and No. 200 sieves

Several researchers (e.g., Han and Selig 1997; Huang et al. 2009; Parsons et al. 2012; Tamrakar et al. 2017b) have studied the characteristics of fouled ballast using laboratory tests; yet others (e.g., Roberts et al. 2006; De Bold et al. 2015; Sadeghi et al. 2018) focused on field tests for that purpose. Huang et al. (2009) used a shear box to determine the shear strengths of clean ballast and ballast fouled with coal dust, clay and mineral filler. Those authors reported the highest strength for the clean ballast. The increase in the degree of fouling resulted in a decrease in strength. The coal fouled ballast showed a significant loss of strength. For all fouled ballasts, the loss in shear strength accelerated with the increase in the moisture content.

This paper aims to understand the effect of ballast fouling on the permanent deformation and stiffness of railway ballast. Two types of fouling agents, rock dust and clay, having different plastic indices were considered. The ballast specimens were tested with a Material Testing and Simulation (MTS) system to obtain the stiffness properties under monotonic loading, a Portable Seismic Property Analyzer (PSPA) to obtain the low-strain modulus, and a Light Weight Deflectometer (LWD) for high-strain modulus.

2. SPECIMEN PREPARATION AND TESTING

A railway track section consisting of a ballast layer and subgrade was simulated in a container as shown in Figure 1. A detail description of specimen preparation is discussed in Tamrakar (2017).
The container, which was made from a polyethylene pipe, had an inner diameter of 900 mm, height of 700 mm, and thickness of 25 mm. The material profile for each specimen consisted of 100 mm of pea gravel at the bottom, 300 mm of subgrade in the middle, and 300 mm of ballast on the top. The bottom and inner walls of the container were lined with a 150 μm thick polyethylene sheet to minimize the interaction between the geomaterials and the container walls. Based on extensive finite element analyses, Amiri (2004) found the dimensions of this specimen was appropriate for the type of tests carried out in this study. They further validated the deformation responses computed through the numerical analysis with the experimental results. The interaction between the soil and container wall based on both numerical and experimental data were negligible.

The subgrade, which was common to all specimens, was designated as SM as per Unified Soil Classification System (USCS) with maximum dry density (MDD) and the optimum moisture content (OMC) of 1794 kg/m³ and 15.2%, respectively. The limestone clean ballast, designated as AREMA 4, was obtained from a local quarry. The dry density of the clean ballast as per ASTM C29 was 1730 kg/m³. Two types of fouling agents, rock dust and clay, were used. The properties of the fouling agents, including fines content (particles passing No. 200 sieve), are tabulated in Table 1. The rock dust was obtained from the same quarry that produced the ballast. The clay was a high-plasticity clay obtained from Minnesota.

<table>
<thead>
<tr>
<th>Fouling Agent</th>
<th>Plasticity Index</th>
<th>Fines %</th>
<th>Soil Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>53</td>
<td>97</td>
<td>CH</td>
</tr>
<tr>
<td>Rock Dust</td>
<td>10</td>
<td>1.5</td>
<td>SW</td>
</tr>
</tbody>
</table>

Bailey et al. (2011) indicated that the fouling percentage could reach as high as 50%. The fouled ballast specimens for this study were prepared by mixing the dry clean ballast with the appropriate fouling agents in the proportion of 20% (moderately-fouled) or 50% (heavily-fouled) by weight of clean ballast. The specimens were compacted to the nominal densities reported in Table 2. The gradations for the clean and fouled ballast specimens are shown in Figure 2. The classifications of the fouled ballast specimens as per Selig and Waters (1994) are also tabulated in Table 2.

<table>
<thead>
<tr>
<th>Ballast Specimens</th>
<th>Degree of Fouling</th>
<th>Fouling Percentage (%)</th>
<th>Fouling Index (%)</th>
<th>Dry Density [kg/m³]</th>
<th>Measured Moisture Content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Dry</td>
</tr>
<tr>
<td>Clean</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>1,730</td>
<td>--</td>
</tr>
<tr>
<td>Clay Fouled</td>
<td>Moderately</td>
<td>20</td>
<td>39</td>
<td>1,858</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Heavily</td>
<td>50</td>
<td>99</td>
<td>2,066</td>
<td>0</td>
</tr>
<tr>
<td>Rock Dust Fouled</td>
<td>Moderately</td>
<td>18</td>
<td>12</td>
<td>1,954</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Heavily</td>
<td>46</td>
<td>30</td>
<td>2,194</td>
<td>0</td>
</tr>
</tbody>
</table>
Each specimen was tested under three moisture conditions: dry, saturated and partially dry (i.e., partially dried after saturation) conditions. The tests on the fouled ballast specimens prepared with the appropriate amount of oven-dried fouling materials represented the test under the dry condition. After the completion of the tests in the dry condition, the fouled ballast was saturated by introducing water from soaker hoses placed on top of the specimen. The tests were subsequently repeated in the saturated condition. The saturated ballast was left to dry for three days after the second set of tests, and the tests were repeated in the partially dry condition. After the completion of each test, moist samples were extracted to measure moisture content using the oven dry method. Table 2 shows the measured moisture content of the fouled ballast specimens.

A 1.3 MN MTS system (see Figure 3a) was used for conducting load-deformation test through a down-scaled railway tie using a steel bracket. A monotonic load, up to 16 kN, was applied at a loading rate of 2.2 kN/min (see Figure 3b).
After reaching the peak load, the specimen was unloaded in one minute. The seating load of 0.9 kN was considered to have proper contact between the tie and the ballast surface. Figure 3c typical loading and unloading responses of a typical load-deformation test.

PSPA is a hand-held device for measuring stiffness parameters of pavement layers (Nazarian et al. 2003). The PSPA consists of two accelerometers and a source packaged into a hand-portable system (see Figure 4a). The source produces an impulsive impact on the material surface that generates stress waves. The signals of the stress waves are captured by two accelerometers. Using the fast Fourier analysis, the time histories captured by the accelerometers are converted into frequency-domain signals. A phase plot is developed from the frequency-domain signals (see Figure 4b). An average shear wave velocity (VS) of the material is computed by unwrapping the phase plot (Tamrakar et al. 2017a). The low-strain or linear elastic modulus ($E_{PSPA}$) (termed as a PSPA modulus) is derived with Poisson’s ratio ($\nu$) and mass-density ($\rho$) using:

$$E_{PSPA} = 2\rho \, V S^2 (1 + \nu)$$  \hspace{1cm} (1)
An LWD manufactured by Zorn Instruments was adopted (Figure 5) to perform tests as per ASTM E2583. For each specimen, three spots were chosen to conduct the LWD tests. The effective modulus (termed as a LWD modulus) was computed using the following equation.

\[
E_{eff} = \left[ \frac{(1-\nu^2)F}{\pi a d_{LWD}} \right] f
\]  

(2)

where \( \nu \) = Poisson’s ratio of geomaterial (assumed as 0.4), \( a \) = radius of load plate (100 mm), \( F \) = LWD load (7.5 kN), \( d_{LWD} \) = LWD surface deflection, and \( f \) = shape factor (assumed as 2) which is a function of the plate rigidity and soil type.

3. PRESENTATION OF RESULTS

The load-deformation behavior of the clay-fouled ballast specimens at dry, saturated and partially dry conditions is presented in Figure 6. Figure 6a represents the load-deformation responses for the moderately fouled specimens whereas Figure 6b represents for the heavily fouled specimens. The black dotted curves in those figures represent the loading response of the clean ballast specimen. Unfortunately, the unloading response was not available for the specimen with the clean ballast. Compared to the clean specimen, the moderately fouled specimens deformed less, and the heavily fouled specimen deformed more. The deformation pattern of the heavily fouled specimens was exaggerated by the change in the moisture content.
Similar load-deformation behavior of the rock dust-fouled specimens is presented in Figure 7. Both moderately and heavily fouled specimens deformed less than the clean specimen. The reduction in the deformation may be due to the presence of low-PI rock dust. The load deformation curves for the rock dust-fouled specimens is essentially independent of the moisture condition (see Figure 7).

Table 3 compares the permanent deformations measured at the end of the monotonic load-deformation tests on the fouled specimens. The highest permanent deformation of 20 mm was observed for the saturated heavily fouled specimens prepared with clay. Except for the saturated and partially dry clay-fouled specimens, the permanent deformations of specimens decreased with the increase in the degree of fouling. These facts indicate that the moisture and high PI fouling agents contribute to increasing permanent deformations.
Table 3. Permanent deformation of clay- and rock dust-fouled ballast specimens

<table>
<thead>
<tr>
<th>Degree of Fouling</th>
<th>Clay</th>
<th>Rock Dust</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dry</td>
<td>Sat.</td>
</tr>
<tr>
<td>Moderately-Fouled</td>
<td>1.7</td>
<td>2.1</td>
</tr>
<tr>
<td>Heavily-Fouled</td>
<td>1.1</td>
<td>20.1</td>
</tr>
</tbody>
</table>

The PSPA moduli of the clean, and clay- and rock dust-fouled ballast specimens are presented in Figure 8. The coefficients of variation in PSPA measurements are in the range 10% to 26% which demonstrates the heterogeneous nature of the ballast and points out to the need for more than one measurement for characterizing such materials.

Figure 8. PSPA Modulus of Clean and Fouled Ballast

The PSPA tests of the clean ballast were only conducted under the dry condition because the free-draining nature of the material did not lend itself to saturation. For the moderately fouled specimens, irrespective of the fouling agent, the saturated moduli were less than the corresponding dry conditions. However, as the materials dried out to the partially dry condition, the moduli increased significantly. For the heavily fouled conditions, the trend is somewhat different. Even though the saturated moduli are less than the dry ones, the moduli after subsequent drying do not increase significantly. Comparing the moduli under the dry conditions, the dry moduli increase as the degree of fouling increases. This indicates that under the dry condition, fouling might help in stiffening the track foundation. On the other hand, as soon as the fouled ballast becomes saturated it will lose its bearing capacity to some extent.

Figure 9 presents the variation in the LWD moduli of the clean and fouled ballast specimens with moisture content and degree of fouling. The average LWD modulus of the clean ballast was 41 MPa, which was significantly less than the corresponding PSPA modulus (225 MPa). The differences can be
attributed to the fact that the PSPA moduli are the small-strain moduli of the ballast layer, whereas the LWD moduli are the high-strain moduli of the combination of the ballast and the softer subgrade layer below it.

Figure 9. LWD Modulus of Clean and Fouled Ballast

The LWD moduli for the specimens moderately fouled with clay demonstrated a pattern that is somewhat similar to the PSPA. The moduli of the specimens heavily fouled with rock dust were almost independent of the moisture condition. On the contrary, the moduli of the specimens heavily fouled with clay demonstrated sensitivity to moisture condition. Even though the moduli of the specimens moderately fouled with rock dust and clay were similar under similar moisture conditions, the moduli of the specimens heavily fouled were significantly different. Unlike the PSPA moduli from the dry conditions, a consistent pattern between the modulus and degree of fouling was not apparent.

4. SUMMARY AND CONCLUSION

To understand the mechanical behaviors of fouled ballast, several specimens representing a railway track section consisting of a 300-mm-thick ballast over subgrade were constructed in the laboratory. The specimens were prepared in a container being 0.9 m in diameter and 0.7 m in height. A limestone based clean aggregate, designated as AREMA 4, was selected for preparing the ballast layer. Rock dust with a low PI and clay with a high PI, were used as the fouling agents. The ballast specimens were moderately and heavily contaminated with either the rock dust or clay. To simulate the dry-wet-dry cycles, the dry contaminated ballast specimens were saturated and then left to dry for a few days. The behavior of ballast was measured under monotonic load using an MTS system and impact load tests using PSPA and LWD.

Based on this study, the following conclusions can be drawn:

1) The ballast specimen fouled with clay (i.e., high PI fouling agent) is highly susceptible to permanent deformation. The permanent deformation is significantly accelerated by the increase in the degree of fouling.

2) The ballast specimen fouled with rock dust (i.e., low PI fouling agent) have minimal or no impact on permanent deformation. Moisture has small effects on the permanent deformation.
3) The effect of fouling on ballast stiffness is observed more from the LWD moduli than the PSPA moduli.
4) The major factor contributing to increasing railway track deformation is the combination of increasing moisture content and high PI fouling agents.

REFERENCES


Laboratory Investigation of the Cyclic Behavior of Cementitiously Stabilized Granular Materials Using Dissipated Strain Energy Method

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University of Texas at El Paso, Civil Engineering Department

ABSTRACT

Dissipated strain energy method as a novel approach can be utilized to better understand the cyclic behavior of stabilized granular materials subjected to repeated loading. The pseudo strain concept has been used as an indication of the damage propagation in pavement surface layers. This study focuses on the utilization of energy methods to identify the onset fatigue damage in lightly and heavily stabilized cement treated granular soils. This objective was achieved by the execution of a comprehensive experiment design consisted of 128 stabilized cylindrical specimens subjected to the UCS and submaximal modulus tests at three strength ratio levels. Four aggregate sources with distinct lithologies were incorporated in the experiment design to account for the role of aggregate mineralogy on the resilient properties of tested specimen. Two curing/conditioning procedures were also incorporated in this effort to quantify the role of moisture in the hysteresis behavior of cement treated aggregate systems. The analysis of the dissipated energy in repeated load tests at different strength ratio levels provided valuable insight on the synergistic influence of stress path, moisture state, and stabilizer content on the laboratory performance of treated granular soils. The hysteresis behavior of lightly stabilized specimen in the initial load cycles confirmed the post-compaction anomalies associated with sample disturbance. The heavily stabilized systems however showed negligible dissipated energy in the initial cycles; however, the formation of micro-cracks in overly rigid systems manifested itself in growth and translation of the hysteresis loops, and consequently progression of damage in the system. The proposed protocol here can be beneficial for the pavement design industry to identify a threshold below which the cement treated layer acts as a flexbase with limited potential for reflective cracking.

1. INTRODUCTION

Cementitious stabilization has been widely used to improve the strength and durability of granular materials for the construction and rehabilitation of pavement structures. Improved mechanical properties and durability of the stabilized materials combined with relatively low cost makes it an attractive method. These materials are generally characterized on the basis of physical properties such as gradation, density, durability, strength, and stiffness from monotonic laboratory test. Many of these properties are determined either empirically or from testing procedures without properly considering the relevance to the actual performance of pavement materials under traffic loading. Dissipated strain energy method as a novel approach can be utilized to better understand the cyclic behavior of cement stabilized granular materials subjected to repeated loading. Recent research supports the use of this technique as a more fundamental approach to a material’s damage potential when subjected to an external stimuli such as repeated traffic loads (Shen et al. 2006, Ghuzlan and Carpenter 2006).
Efforts have been made to propose various representations and applications of the dissipated strain energy approach for the performance analysis in pavement engineering over the last few decades. Sobhan and Krizek (1998) in a pioneer study evaluated a fatigue damage index in terms of cumulative dissipated energy for stabilized base course materials composed primarily of recycled aggregate and fly ash. The authors reported that the total energy dissipation capacity can be construed as a fundamental characteristic to identify the onset of failure in stabilized systems. In this regard, they developed relationship between the cumulative dissipated energy and the fatigue life presented by the following equation:

\[ \ln(E_t) = 0.9798 \times \ln(N_f) - 8.32 \]  
(1)

Where, \( E_t \) is the total cumulative dissipated energy, and \( N_f \) is the number of cycles to failure.

Ghuzlan and Carpenter (2000) reported that the rate of change in dissipated energy by itself does not provide a single unified method to examine failure in different test modes. To overcome this difficulty, the authors examined a ratio of the change in dissipated energy between two cycles divided by the dissipated energy of the first cycle as evidenced in Equation 2.

\[ DER = \frac{(DE_{n+1} - DE_n)}{DE_n} \]  
(2)

Where \( DER \) is dissipated energy ratio, \( DE_n \) is dissipated energy produced in load cycle \( n \), and \( DE_{n+1} \) is dissipated energy produced in load cycle \( n+1 \).

Shen and Carpenter (2005) reported that this ratio provides a true indication of the damage imparted on the mixture in two consecutive cycles. The authors also presented typical damage curve represented by dissipated energy ratio versus loading cycles in Figure 1 which distinctively divided into three stages. The portion of interest in this plot is Stage II (plateau stage), in which the dissipated energy ratio is almost constant until the significant increase in Stage III due to onset of failure. This constant value of dissipated energy ratio, the plateau value, characterizes a period in which there is a constant percent of input energy being turned into damage. Shen et al. (2007) revealed that this value appears to be completely related to the mixture and material types. Ghuzlan (2001) also concluded that this value provides a unique relationship with fatigue life for different mixtures, and loading levels for the asphalt materials.
Although the dissipated strain energy concept has been widely used for hot-mix asphalt pavement performance, there are limited studies for the use of this approach for unbound and chemically stabilized materials. Ashtiani and Arteaga (2017) investigated the cyclic behavior of geomaterials using dissipated energy concept for unbound limestone aggregates with variable fines content, degrees of saturation, and confining pressures. The authors showed that the analysis of the hysteresis loops underscored the capability of energy methods to identify the onset of incremental collapse behavior in geomaterials. They provided valuable information for the practitioners and the pavement design industry to mitigate the distresses associated with field rutting. In a relevant study, Tao et al. (2010) used the dissipated energy concept to characterize the behavior of virgin and recycled base materials based on laboratory repeated load triaxial tests. The authors demonstrated how the dissipated energy evolves during the process of permanent deformation tests and the manner in which this energy relates to permanent deformation damage in tested materials. They reported that different responses obtained from permanent deformation tests can be reasonably well interpreted from the dissipated energy approach. Ultimately, they concluded that the use of the dissipated energy concept may bridge the gap between micromechanical processes and macro-behavior energy concept.

Apart from these studies, the dissipated strain energy theory has proven very promising in discerning different responses of geomaterials under repeated loading (Werkmeister et al. 2004). Therefore, the primary objective of this research was to investigate the feasibility of applying the dissipated strain energy approach to better understand the deformation behavior of cement stabilized base materials subjected to repeated loading. This objective was achieved by establishing a multi-dimensional aggregate database, consisted of approximately 128 specimens subjected to several mechanical and durability tests in the laboratory.

2. CYCLIC BEHAVIOR OF CEMENT STABILIZED MATERIALS

Cyclic loading of geomaterials in the laboratory can be characterized under two distinct regions which correspond to the plastic deformation and the elastic recovery. The plastic region is the non-recoverable
permanent deformation that results in the densification of the aggregate strata by means of stored energy. The elastic region, also recognized as resilient strain, is the non-plastic strain energy that is not capable of performing work in the material. Both the plastic and elastic strain can be observed within the generation of the hysteresis loop as shown in Figure 2. Conceptually, the area constrained by the hysteresis loop represents the dissipated energy, and the area formed by the peak strain and the elastic strain represents the elastic energy shown in Figure 1.

![Figure 2. Schematic of Stress-strain Hysteresis Loop](image)

For the stabilized materials, the hysteresis loop area or dissipated strain energy per cycle generally decreases with the number of load cycles. Figure 3 shows the schematic behavior of cement stabilized material under cyclic loading. The plot illustrates that the most energy is dissipated during the first few cycles, reaching a steady state that the dissipated strain energy in each cycle remains relatively constant. The final dissipated energy per cycle is dependent on loading levels and it takes longer for the sample to reach a steady state under a higher loading level.

![Figure 3. Cement Stabilized Material Behavior under Cyclic Loading](image)
A polynomial approximation was utilized to calculate the area constrained by the hysteresis loop in cyclic loading of geomaterials in the laboratory. The constrained area between two polynomials formed a hysteresis loop is determined according to Equation 3. This equation presents the dissipated strain energy as the constrained area of the hysteresis loop.

\[
E_s = \left[ \int_{\varepsilon_{\text{min}}}^{\varepsilon_{\text{max}}} \sigma_t(\varepsilon) d\varepsilon - \int_{\varepsilon_{\text{p}}}^{\varepsilon_{\text{max}}} \sigma_b(\varepsilon) d\varepsilon \right] V_s
\]

(3)

Where, \( E_s \) is stored energy; \( V_s \) is volume of specimen; \( \varepsilon_{\text{min}}, \varepsilon_{\text{max}}, \varepsilon_p \) are critical strain points; and \( \sigma_t(\varepsilon), \sigma_b(\varepsilon) \) can be calculated according to Equation 4:

\[
\sigma_t(\varepsilon), \sigma_b(\varepsilon) = a\varepsilon^n + a\varepsilon^{n-1} + \cdots + y\varepsilon^1 + z\varepsilon^0
\]

(4)

3. MATERIALS AND METHODS

Four aggregate sources with distinct lithologies were incorporated in this research to account for the role of mineralogy and surface properties on the cyclic behavior of cement treated aggregates. Four increments of cement ranging from 2% to 5% were added to each permutation of the experiment design to cover the lightly stabilized to heavily stabilized spectrum. Subsequently, the variants of the experiment matrix were subjected to stress-controlled submaximal tests (strength ratio tests) performed on the 6 in × 12 in cylindrical specimen as shown in Figure 4. The strength values obtained from the UCS test in this project were the basis for the selection of the stress amplitudes applied to the permutations of the experiment design. Pre-determined fractions of the UCS-value, namely 20%, 40% and 60% were cycled for 5,000 repetitions for each variant to characterize the cyclic behavior of stabilized systems. Table 1 presents the experiment design developed in this research effort.

Two different methods of conditioning/curing of the stabilized materials were incorporated in this research to study the influence of moisture ingress on the mechanical properties of the stabilized materials.

I. 7-Day Moist Cured: Stabilized specimens were deposited in a chamber with at least 95% relative humidity for seven days

II. 10-Day Capillary Soak: Stabilized specimens were placed on porous stones in a tub of water and were subjected to capillary soak for 10 days prior to laboratory testing
The main motivation for the inclusion of the capillary soak procedure in the experiment was to identify the sensitivity of the aggregates to hold and transport unbound moisture. The rationale is that unreacted moisture trapped in the pore structure can potentially degrade the stiffness properties of the stabilized layers and consequently jeopardize the longevity of the pavement structure. The moisture susceptibility tests provided valuable information on the deleterious (or beneficial) influence of provided moisture through capillary action on the cyclic behavior of stabilized materials in the laboratory (Ashtiani et al. 2018).

Table 1. Experiment Design

<table>
<thead>
<tr>
<th>Curing Conditioning Procedures</th>
<th>Test</th>
<th>Aggregate Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Limestone El Paso</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2 3 4 5</td>
</tr>
<tr>
<td>7-Day Moist Cured</td>
<td>Submaximal Test @ 20% UCS Strength</td>
<td>✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td></td>
<td>Submaximal Test @ 40% UCS Strength</td>
<td>✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td></td>
<td>Submaximal Test @ 60% UCS Strength</td>
<td>✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td></td>
<td>Unconfined Compressive Strength Test</td>
<td>✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td>10-Day Capillary Soak</td>
<td>Submaximal Test @ 20% UCS Strength</td>
<td>✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td></td>
<td>Submaximal Test @ 40% UCS Strength</td>
<td>✓ ✓ ✓ ✓</td>
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<tr>
<td></td>
<td>Submaximal Test @ 60% UCS Strength</td>
<td>✓ ✓ ✓ ✓</td>
</tr>
<tr>
<td></td>
<td>Unconfined Compressive Strength Test</td>
<td>✓ ✓ ✓ ✓</td>
</tr>
</tbody>
</table>
The particle size distributions for four aggregate sources in this study is presented in Figure 5. The selection of the particle size distributions was based on Item 247 of TxDOT standard construction specifications for Grade 4 materials. The moisture-density tests were performed on untreated materials to identify the Optimum Moisture Content (OMC) and Maximum Dry Density (MDD) following Tex-113-E specification (2016). Table 2 provides the OMC and MDD results. Subsequently, the molding moisture content was adjusted for each increment of cement added to the mix based on Tex-120-E procedure (2013).

4. RESULTS AND DISCUSSION

Hysteresis loops for sandstone, gravel and two types of limestone sourced from San Antonio and El Paso were obtained through submaximal test at the first and 5,000th load cycles as observed in Figure 6. The results show that the area constrained by the hysteresis loop becomes larger with increasing stress ratio of load application. This indicates that materials subjected to the repeated loading at higher stress ratio (60%) had significantly higher dissipated strain energy when compared to materials at lower stress ratio. For instance, increasing the stress ratio from 20% to 60% for all material types resulted in more than 10 times increase in the dissipated strain energy. The shifting and re-adjustment of the particles, manifested by the increase in the strain energy, can potentially alter the mechanical properties of the continuum.
Figures 7 illustrates the hysteresis behavior of materials stabilized with 2% and 5% cement content under the repeated loading level for comparison the lightly stabilized systems with rigid aggregate matrices. The results show that lightly stabilized systems produced the biggest loop area, and exhibited higher dissipated strain energy when compared to the heavily stabilized aggregates at the same stress level. This is a direct consequence of stiffening behavior of stabilized materials with high calcium-stabilizer contents.

Figure 6. Hysteresis Loops of the 1st and 5,000th Load Application with Different Stress Ratio

Figure 8 provides the hysteresis loops at the first, 100th and 5,000th load cycles for sandstone and limestone aggregate materials. The investigation of the size of the hysteresis loops, as well as the translation and the inclination of loops provide valuable information regarding the damages imparted by demanding stress states on geomaterials. As observed in these plots, the radial shift in hysteresis loops are more pronounced in limestone material (especially for San Antonio limestone) compared to sandstone aggregate. The tilting of hysteresis loops in limestone base materials is a direct consequence of the development of the damage due to cyclic loading (Figure 9). In addition, limestone sourced from
San Antonio showed higher horizontal translation of the hysteresis loops, which is an indication of higher magnitude of plastic strains with increasing of load applications.

To quantify the role of moisture in the hysteresis behavior of cement treated aggregate systems, the hysteresis loops for 7 day moist cured and 10-day capillary soaked samples at the first and 5,000th load application are presented in Figure 10. The results show that the 10-day soaked specimens for limestone materials exhibited lower dissipated strain energy compared to the 7-day moist cured specimen. This could be attributed to the consumption of the available moisture in strength gain reactions, which resulted in improvements in the mechanical properties of limestone specimen. Conversely, siliceous gravel specimens subjected to the 10-day capillary soak (TST condition) had higher dissipated strain energy than specimens subjected to 7-day moist cure condition. This could be due to high moisture retention capacity of gravel aggregate sourced from Pharr district. The results provide valuable insight on the impact of the aggregate mineralogy and surface properties on the strength gain in presence of pozzolanic materials.

Figure 7. Hysteresis Loops of the 1st, 100th and 5,000th Load Application with Different Cement Content
Figure 8. Hysteresis Loops of the 1st, 100th and 5,000th Load Application for Sandstone and Limestone Sourced from El Paso and San Antonio with 2% Cement Content

Figure 9. Hysteresis Loops of the 1st Load Application for Sandstone and Limestone Sourced from El Paso and San Antonio with 4% Cement Content
Figure 10. Hysteresis Loops of the 1st and 5,000th Load Application Aggregate Base Materials Stabilized by 5% Cement Content for 7 day Moist Cured and 10-Day Capillary Soak (TST) Samples

Figure 11 shows the normalized dissipated strain energy per cycle versus number of loading cycles for limestone (El Paso) and sandstone base materials. Since the dissipated energy is directly related to the stress amplitudes, it is imperative to normalize the measured energy by the magnitude of loading cycles for proper comparison of dissipated strain energy. The results clearly demonstrate that the dissipated unit energy per cycle generally decrease with the number of load cycles. It was observed that the largest dissipated unit energy per cycle occurs within the first 10 loading cycles, after which the dissipated unit energy per cycle decreases to a relatively constant value at a material-specific rate. The plot also shows that lightly stabilized systems had significantly higher dissipated strain energy when compared to the heavily stabilized aggregates. The final dissipated energy for limestone specimen produced with 2% cement content was around 0.06 per cycle, while this value for specimen with 4% cement content (heavily stabilized aggregates) was 0.006.
Figure 11. Variation of the Dissipated Energy with the Number of Load Cycles for Limestone (El Paso) and Sandstone Base

Figures 12 illustrates the variation of the normalized dissipated energy with the number of load cycles at 20% and 60% strength ratio to demonstrate the role of stress path and aggregate mineralogy on the resilient properties of virgin aggregate base layers. The results showed that the final dissipated energy per cycle is dependent on loading levels and it takes longer for the specimen to reach a steady state under a higher loading levels. For instance, the final dissipated strain energy for limestone specimen (El Paso) produced at 60% stress ratio (higher loading level) was around 0.01 per cycle, while this value for the same specimen produced at 20% stress ratio (lower loading level) was 0.006. Larger dissipated energy generally implies a larger irreversible deformation incurred within aggregate via rolling and sliding of its particles to accommodate the external loading.

Figure 12. Variation of the Dissipated Energy with the Number of Load Cycles at 20% and 60% Strength Ratio for All Permutations of the Experiment Design with 3% Cement Content
The results also clearly indicate that siliceous gravel aggregate sourced from Pharr significantly underperformed in terms of dissipated energy compared to the counterparts. The higher magnitude of dissipated strain energy in stabilized gravel materials is a direct consequence of the development of the damage due to cyclic loading.

5. CONCLUSION

The focus of this research effort was to investigate the feasibility of applying the dissipated strain energy approach to interpret the test materials responses under repeated loading. This objective was achieved by the execution of a comprehensive experiment design consisted of four aggregate sources with distinct lithologies to account for the influence of aggregate mineralogy on the strength of cement stabilized materials. Four levels of cement content, ranging from 2% up to 5% were added to the mixes to cover a wide spectrum of cement treatment from light stabilization to heavily stabilized systems. The stabilized aggregate specimen was subjected to two curing/conditioning procedures prior to the mechanical tests, to study the affinity of the aggregates to adsorb and transport moisture. Two mechanical tests including UCS and submaximal modulus tests at 20%, 40%, and 60% strength ratios were performed on the permutations of the experiment design to study the behavior of the systems at small, intermediate, and high strain levels. The results of the study showed that lightly stabilized systems retained much more strain energy when compared to the heavily stabilized aggregates. The nonlinear and relatively soft response of crushed limestone and sandstone aggregates changed to a stiffer response with the increase of cement content. Comparatively, stabilized siliceous gravel materials benefited less from the incremental increase of cement content in terms of hardening behavior of materials under repeated loading. This showed the role of aggregate mineralogy in the process of hydration and pozzolanic reactions. In addition, stabilized base materials subjected to the repeated loading at higher strength level (60%) retained much more strain energy when compared to materials at lower strength ratio. It was shown that increasing the stress ratio from 20% to 60% for all material types resulted in more than 10 times increase in the dissipated strain energy. This energy brings changes to the aggregate’s properties through microstructural adjustment and leads to damaging effect to the aggregate depending on loading levels. Consequently, these results can be potentially beneficial for the pavement design engineers to better understand the cyclic behavior of cement stabilized granular materials subjected to repeated loading.

REFERENCES


Texas Department of Transportation, (TxDOT), (2016), Test Procedure for Laboratory Compaction Characteristics and Moisture Density Relationship of base Materials, (Tex-113-E).


Implementation of Geotechnical Asset Management

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BGC Engineering, Inc.

ABSTRACT

While bridge and pavement conditions receive much of the media attention and legislative directives for transportation agencies, the value and performance of other assets are also important to the life-cycle operation of the transportation system. One such category are geotechnical assets, which are the walls, slopes, embankments and subgrades that contribute to the ability of a transportation agency to perform the strategic mission. Implementing asset management practices for assets such as walls and embankments enables an owner to measure and manage the life-cycle investment considering performance expectations and tolerance for risk. While not mandated through legislative processes, the reasons for geotechnical asset management (GAM) are comparable to any other business practice that is directed at making smart investments with limited funds. Without geotechnical asset management, organizations are accepting unknown magnitudes undue risk to traveler safety, mobility, and economic vitality, while also potentially making unfavorable life-cycle investments. Where implemented, studies indicate performance and financial benefits from GAM, including life-cycle savings of 60 to 80% per unit length of embankment (United Kingdom). To assist infrastructure owners with implementation of asset management for features such as slopes, walls, and embankments, the National Cooperative Highway Research Program (NCHRP) funded research project 24-46: “Development of an Implementation Manual for Geotechnical Asset Management for Transportation Agencies.” The purpose of this study is to deliver an implementation manual for transportation executives, asset managers, and geotechnical practitioners to initiate asset management for walls, slopes, and embankments. The outcome from the study, NCHRP Research Report 903, was issued in May 2019. The report includes an implementation manual that will be of immediate use to practitioners who own retaining walls, slopes, embankments and subgrades. Complementary Microsoft Excel files include powerful planning worksheet tools to begin a simple asset management plan with a low level of effort by asset managers, geotechnical, bridge management, or other professionals.

1. INTRODUCTION

Above all, asset management is a business process for asset owners to maximize the value of their assets and make risk informed decisions that align with the objectives and goals of the organization. Thus, through asset management private and public organizations can demonstrate both financial stewardship to investors or the enabling taxpayers and increase the likelihood services provided to customers and users of the system are acceptable over time. Legislative authorization or regulations do not need to be the primary initiator of asset management as evidenced by the British Standards Institute (BSI) work on standardization of asset management, which was first published in 2004 with more than 50 public and private entities spanning 10 countries and 15 sectors contributing to development (BSI,
According to the Federal Highway Administration (2018), “transportation asset management plans are an essential management tool which bring together all related business processes and stakeholders, internal and external, to achieve a common understanding and commitment to improve performance.” The current obligatory requirements of Federal highway authorization require the continued management of bridge and pavement assets and encourage inclusion of other assets into asset management plans. One such asset class that can be considered is geotechnical assets.

Geotechnical assets are the retaining walls, embankments, slopes, and constructed subgrades within a transportation system right-of-way (ROW) or easement. Like other asset categories, geotechnical assets are features that are designed, constructed, and maintained by a transportation agency. Their performance—good or poor—contributes to the continuous operation of a transportation network. Geotechnical assets are also subject to deterioration and exposed to natural hazards.

Implementing asset management practices for geotechnical assets enables an agency to measure and manage the life-cycle investment, considering performance expectations and tolerance for risk. The reasons for geotechnical asset management (GAM) are comparable to any other business practice that is directed at making smart investments with limited funds. Without geotechnical asset management, transportation owners are accepting unknown levels of risk to objectives related to safety, service, and economic vitality, and may also be making unfavorable life-cycle investments.

The value of geotechnical asset management was realized by the Transportation Research Board and National Highway Cooperative Research (NCHRP) program through the commissioning of NCHRP Study 24-26: “Development of an Implementation Manual for Geotechnical Asset Management for Transportation Agencies.” The outcome of this study, NCHRP Research Report 903 includes an implementation manual with supporting software tools for transportation executives, asset managers, and geotechnical practitioners to initiate asset management for walls, slopes, and embankments.

2. DEFINING GEOTECHNICAL ASSETS

The ISO 55000 definition of an asset is: an item, thing or entity that has potential or actual value to an organization; value can be tangible or intangible, financial or non-financial, and includes consideration of risks and liabilities (ISO, 2018).

Separately, the AASHTO transportation asset management guide: a focus on implementation (2011) defines an “asset” as the physical transportation infrastructure (e.g., travel way, structures, other features and appurtenances, operations systems, and major elements thereof); more generally, can include the full range of resources capable of producing value-added for an agency: e.g., human resources, financial capacity, real estate, corporate information, equipment and materials, etc.; an individual, separately-managed component of the infrastructure, e.g., bridge deck, road section, streetlight. Thus, in both definitions transportation assets consist of the many physical assets that add value to the transportation system and can impact the ability of the organization to satisfy executive level performance objectives.
One such category of assets is those comprised of earth (soil and rock), or a geotechnical asset, with the adjective ‘geotechnical’ describing constructed earth materials. Examples of geotechnical assets include the retaining walls, embankments, slopes, or constructed subgrades that contribute to the performance of a transportation system and are located within the right-of-way or boundary. Geotechnical assets also contribute to the performance of culverts, stormwater drainage systems, and utilities that are often contained within the asset.

The presented definition of geotechnical assets is not unique to the U.S., as slopes and embankments are defined as geotechnical assets for highway and railway systems in the United Kingdom (U.K.) and have been managed as part of asset management programs for over a decade. Additionally, there often are situations where multiple geotechnical assets exist along the same unit length of road or railway, such as a retaining wall on the downslope side of a corridor and slopes on the uphill side. Thus, agencies can have substantial inventories of geotechnical assets, and in many cases may have more retaining walls than bridges.

Additional discussion on the definition of geotechnical assets is presented in the following subsections.

2.1 Retaining Walls

Retaining walls, or earth retaining structures, are structures that hold back soil and/or rock materials to prevent sliding of material onto a transportation corridor or retain material that supports other transportation assets. Retaining walls can include many types of walls, including gravity walls, soil nail walls, concrete cantilever structures, or mechanically stabilized earth (MSE) walls. Retaining walls generally have vertical or near vertical faces. The recommended height for incorporation into an asset management inventory is 4 feet, which is similar to the height that defines a retaining wall in the engineering design process.

In many cases, a retaining wall is associated with a bridge structure or approach to a bridge. If a wall is also a bridge abutment that is integral to the bridge structure, the wall likely will be incorporated into a separate bridge inspection and asset management program. However, all other walls associated with
the bridge approaches should be incorporated into a geotechnical asset management program if not already managed in an asset management program, which is often the situation.

2.2 Embankments

An embankment asset consists of a constructed fill comprised of rock, soil, or other engineered materials that enable a roadway, railway, or other transportation facility to maintain a required design elevation above lower lying ground. A threshold embankment height of 10 feet (3 m) is recommended as delineation between a minor earthwork and an embankment asset, unless there are site conditions or criticality may merit a lower height. The 10-foot recommended height for an embankment is based on the implementation experience for 240,000 geotechnical assets on highways and railways throughout the United Kingdom (NASEM, 2019).

![Figure 2. Example Embankment Asset (from NASEM, 2019).](image)

2.3 Slopes

Slopes are a type of geotechnical asset involving cut excavations that enable a roadway or railway to traverse through surrounding ground, as shown in Figure 3. Slopes also may include natural slopes adjacent to a roadway for some agencies. Slopes differ from embankments in that slopes are excavated into terrain rather than a constructed fill feature or may consist of a natural slope. Slopes can consist of soil, rock, and mixtures of soil and rock as illustrated in Figure 3. Similar to embankment assets, a 10-foot threshold for cut slope height is recommended, unless the asset is judged to create an unacceptable hazard to the safety of users and maintenance personnel.
2.4 Subgrades

Subgrade assets consist of earth materials below the engineered pavement or railway that creates a lifecycle management need. Examples of subgrade assets include constructed earthworks and ground improvements to address weak foundation soil, swelling, frozen or thawing ground, and collapsible soil or threats from karst (sinkholes) and underground mining. Examples of subgrade assets are illustrated in Figure 4.

2.5 Taxonomy for Geotechnical Assets

The taxonomy for geotechnical assets was introduced by Anderson et al. (2016) and further developed in the NCHRP Research Report 903 (NASEM, 2019). The geotechnical asset taxonomy is based on common definitions used in professional geotechnical practice and with the intent of aligning with the state of practice with transportation asset management. This taxonomy is similar in structure to the general GAM taxonomy used by Highways England and Network Rail, allowing U.S. practice to connect with international practice. As noted by Anderson et al. (2016), the purpose of this taxonomy is to clarify
language and ideas so that geotechnical engineers, other disciplines, and asset managers can communicate effectively within one organization and between different ones.

2.5.1 Role of Right-of-Way
Established transportation assets, such as bridges and pavements, are easily recognized as being within the ROW or easement boundaries of an organization and often with ample buffer space from boundaries. For these assets, the agency has control over how they are built, maintained, and managed, in addition to full access rights. Geotechnical assets also exist within the agency boundaries and many geotechnical assets often extend right to the ROW boundary, or beyond. It is not uncommon that the limits of disturbed area associated with a slope or embankment asset will define the ROW or easement boundary for the agency and retaining walls often have the function of minimizing ROW disturbance and thus are constructed at the boundary. In such conditions, there is potential for adverse consequences to external property stakeholders beyond the ROW from an agency geotechnical asset, which is different performance threat when compared to a pavement asset.

There also can be geotechnical features outside of the right-of-way or boundary that are not owned by the agency. These features often consist of location specific hazards associated with a natural slope that threatens other transportation assets or the agency performance. Examples of these features may include rockfall from geologic outcrops, landslides that originate beyond the boundary, or private retaining walls, often common in urban areas, that can impact agency assets. Historically, many agencies have assumed the responsibility for the response to events originating from geotechnical features beyond the ROW boundary. However, the access and ownership constraints limit the ability of an agency to manage these sites using the same design, maintenance, rehabilitation, or replacement treatment concepts applied to geotechnical assets in the ROW.

Given this ownership distinction, the taxonomy for geotechnical assets uses a separate category for geotechnical features or natural hazard sites that are beyond the ROW. Through this taxonomy distinction, an agency can choose whether to include geotechnical features beyond the ROW in a geotechnical asset management program or to defer to other programs, such as enterprise wide risk and resilience strategies that address other external agency hazards such as flooding, earthquake, or terror events. In such an example, having an inventory of beyond the ROW geotechnical features can be beneficial.

2.5.2 Geotechnical Elements within other Assets
Bridges, tunnels, and pavements contain geotechnical visual elements or hidden components, such as foundations and ground reinforcements, that enable the function of that specific asset. As indicated by Anderson et al. (2016), these other asset groups already have asset management practices to comply with federal requirements and it is important to recognize the contribution of the geotechnical elements to these other assets, and to manage them through the existing platforms, and not create something new.

For the geotechnical elements and components of other structure assets (e.g., bridges and tunnels), these items can be identified as such, or through using the exact terms already used within the given asset specific asset taxonomy. As an example, the foundations of a bridge comprise a portion of the bridge substructure component and are geotechnical items within an already managed asset.
2.5.3 Geotechnical Assets

Figure 5 presents the hierarchy used to classify embankments, slopes, retaining walls, and subgrades into the geotechnical asset class. For each of these assets, they have geotechnical composition and can be shown to measurably contribute to the value of an agency. Separately, there may be inventory and/or management systems that address one or a few of these geotechnical asset or feature types, particularly those identified as having progressed to an unstable condition. For example, the Rockfall Hazard Rating System (RHRS) was developed in the 1980s by the Oregon Department of Transportation with support from FHWA and other states (Pierson, 1991) to prioritize rockfall hazards from slopes within and beyond the ROW and has since been adopted or modified by transportation organizations with some including other unstable slope types. While slope and embankment assets can share characteristics those with in these legacy hazard management systems, it also is possible to integrate these sites into the broader geotechnical asset category and the risk-based asset management approach.

Figure 5. Taxonomy for Geotechnical Assets (from NASEM, 2019).

3. BENEFITS FROM GEOTECHNICAL ASSET MANAGEMENT

When geotechnical assets are managed in a proactive, whole-life asset management approach, there can be benefits that include life-cycle cost savings; the capability to measure, communicate and manage risk; reduce operational disruptions; and fewer emergency stabilization projects that draw from contingency budgets. Based on outcomes from successful programs in transportation and other
infrastructure systems, the benefits of performing asset management on walls, slopes, embankments, and subgrades can be summarized as follows.

- Financial savings across the life-cycle, with values estimated to be greater than 30% in studies by the U.S. Army Corps of Engineers (USACE, 2013), and 60 to 80% per embankment in the United Kingdom (Perry and others, 2003)
- Measure and manage safety risk exposure across the asset class and with a comparison to other assets
- Lessens traveler delay and closure times
- Reduces adverse economic impacts to connected communities
- Optimize resources, improves sustainability, and reputations
- Enables data driven decisions that support agency and executive objectives
- Understand the risk exposure across objectives and the ability to manage those risks

3.1 Scaling Implementation to Realize Benefits

Asset management is an on-going process that relies on process improvements to direct advancement where the greatest value can occur. When implementing asset management, each organization can adapt the fundamental concepts to the needs and objectives of their agency. Per the AASHTO Transportation Asset Management Guide (ex summary 2011): “There is no ‘one-size-fits-all’ TAM solution for an agency.” Thus, geotechnical asset management does not need to be funded as a substantial new program; rather, it can evolve to the needs and objectives of the organization.

When viewing the implementation history for bridge and pavement assets, each program has evolved from startup programs into more complex maturities that demonstrate measurable benefits. In the case of bridge and pavement asset management programs in the U.S., many municipal agencies have adopted the practices without being required because of the obvious benefits that result. Additionally, the benefits of asset management can be realized before an inventory is complete as evidenced by several years of implementation experience for both Network Rail and Highways England in the United Kingdom (NASEM, 2019). Through these examples, owners of geotechnical assets are encouraged to begin asset management as a complete process that locates early value gains, rather than focusing individually on each step in asset management.

Data collection for inventory and condition can become a time and cost intensive process and the investment in data collection should be compared against the required level of detail for decision support. As stated in the International Infrastructure Management Manual (IIMM) “a rule of thumb is often 80% of the data can be collected for half the cost of 100%. Seeking 100% coverage and accuracy may not be justified, except for the most critical assets” (IIMM, 2015). Following this guidance when starting geotechnical asset management, agencies can benefit from an approach that relies on different levels of detail and collection tools for inventory and condition data.

Per the IIMM (2015), a staged approach is the most practical process for data collection and begins with identification of minimum data for compliance and reporting requirements, next moves to data for prioritizing operations and maintenance decisions, and then concludes with optimizing life-cycle decisions. As discussed in Power et al. (2012), a similar progression to data collection occurred with geotechnical asset management for the U.K. Highway Agency. Within this staged data workflow, not all
assets will necessary go to the final data collection level and reaching the most detailed data state occurs only where justified.

3.2 Measuring Performance in the Context of Asset Management

The goal of any asset management system is to logically align asset design, operation, maintenance, and upgrade decisions with agency goals and objectives. For asset management to succeed across an organization, the program should relate how asset performance affects customers and decisions by executives that are centered on agency goals and objectives. For this to occur, asset performance measures should relate to high-level agency objectives, such as common safety and system performance objectives.

The performance measurement for geotechnical assets can connect with objectives related to Federal Authorization and other common executive benchmarks found in agency mission and vision statements. Through such measurement, the geotechnical asset manager can track and communicate levels of risk, preservation need, or other factors that enable decision makers understand geotechnical assets in the context of agency specific goals. For communicating and measuring implementation benefits in a simple means, geotechnical asset performance measures should be generalized and high-level while still being effective at measuring performance relative to outward objectives and informing decisions.

Lessons learned from mature geotechnical asset management programs and input from agency executives indicates that successfully adopted performance measures are those that relate how the asset performance affects customers or executive decision making. As an example, Network Rail geotechnical asset are assessed with the following measures (Network Rail, 2017):

- train derailments,
- train delay minutes,
- temporary train speed reductions, and
- earthwork failures.

As presented in NASEM (2019), these approaches have been developed into a geotechnical asset level of risk (LOR) measure that connects to objectives related to asset condition, safety impacts, mobility, and economic consequences, which are common objectives across DOTs and means for connecting geotechnical asset performance to stakeholder goals and objectives.

3.3 Life-Cycle Planning and Return on Investment

Across assets types, asset management informs decisions throughout the life-cycle phases of design, construction, operation and maintenance, and decommissioning. For this to occur, asset management decisions will assess the following:

- total cost of the asset over the life, or life-cycle cost,
- risk across the life-cycle, and
- Financial and investment plans for multiple assets over a life-cycle.

To evaluate the life-cycle cost and risk from geotechnical assets and complete financial planning steps, organizations will need to optimize treatments that should be performed on an asset following initial construction. This optimization process should include the cost of each treatment, the appropriate
timing for treatment, and what effect treatment has on the condition of asset. Basic treatments for geotechnical assets include:

- Do Minimum,
- Maintenance,
- Rehabilitation, and
- Renewal or Reconstruction

Through geotechnical asset management, owners of geotechnical assets can make risk-informed treatment plans for their assets that have the benefit of minimizing life-cycle costs and managing risk to levels that are acceptable to the organization. Additionally, treatment decisions should consider the return on investment using benefit-cost analyses to estimate parameters such as net present value (NPV) or benefit cost ratio (BCR). The recently released Geotechnical Asset Management for Transportation Agencies, Volume 2: Implementation Manual (NASEM, 2019) provides guidance and includes tools for organizations to evaluate life-cycle costs and optimize treatment planning decisions for their geotechnical assets.

4. IMPLEMENTATION GUIDANCE AND TOOLS

To assist transportation organizations with geotechnical asset management, the Transportation Research Board commissioned NCHRP Study 24-46 which developed the recently released NCHRP Research Report 903: Geotechnical Asset Management for Transportation Agencies (NCHRP Report 903). NCHRP Report 903 provides an introduction to geotechnical asset management and adaptable guidance for to implementing risk-based geotechnical asset management into the broader practice of asset management. Volume 2 of the report contains an implementation manual with recommended processes and Microsoft Excel spreadsheet tools that can be of immediate use.

The geotechnical asset management process in NCHRP Report 903 aligns with established transportation asset management practices, including the following steps

- Objectives and Measures
- Inventory and Condition
- Performance Gap Identification
- Life-Cycle Cost and Risk Management Analysis
- Financial Plan
- Investment Strategies

To facilitate implementation, the processes and tools are purposely simple and can be used by engineering or asset management staff, or other agency professionals such as maintenance managers and bridge inspectors. As part of starting geotechnical asset management in a resource limited organization, agencies are encouraged to use and adapt data from existing inventories and begin with desktop approaches for an inexpensive inventory development. These data can be input into an inventory and assessment spreadsheet that accompanies NCHRP Report 903 and will provide initial recommendations for asset treatment types, enabling an agency to evaluate 10-year financial planning horizons.
Similar to existing asset management programs for pavements and bridges, the estimated program level investment needs for geotechnical assets across an agency will likely exceed the available funds. Therefore, the implementation manual contains additional processes for considering risk management, risk prioritization, and life-cycle cost investment prioritization approaches at the asset level. These different prioritization approaches are intended to address critical investment needs while also being flexible to the range in objectives that exist among executive and planning staff in different organizations.

5. CONCLUSION

Owners of geotechnical assets – the retaining walls, embankments, slopes, and subgrades that support or protect other transportation assets – are encouraged to incorporate these assets into the broader agency wide asset management planning strategy. Evidence from other industries and countries suggests sustaining benefits are possible based on information from organizations that have been performing geotechnical asset management for well over a decade. When the examples of life-cycle and risk management improvements are extrapolated nationwide to all state DOTs, railway and transit organizations, and local jurisdictions, the benefits from geotechnical asset management can be measurable and substantial. As there is a cost to delaying implementation, simply beginning asset management for a few geotechnical assets can be beneficial and is recommended over postponing implementation because of the absence of legislative or regulatory requirements.

To address the need and enable transportation organizations recognize the potential value, NCHRP Report 903 provides implementation guidance and tools that can be used now by organizations to manage geotechnical assets in a flexible and adaptable process.

REFERENCES


Assessment of Historical Pavement Condition Data from Army Airfield Pavements

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ABSTRACT

Pavement condition surveys and evaluations of Army airfields have been conducted since the late 1940s and early 1950s by local agencies using local funding mechanisms. In May 1982, the Department of the Army initiated a formal program to determine and evaluate the physical properties, load-carrying capacity, and general condition of the pavements at all major U.S. Army airfields. This procedure detailed specific pavement distresses, defined means for physical measurement of each distress, and defined the severity of each identified distress. Implementation of a standardized evaluation approach allowed for repeated testing and inspection of the same portion of pavement, thus creating a history of pavement condition. This paper presents the results of an effort to compile historical data from selected Army airfields for the purpose of assessing global pavement deterioration trends and identifying types of common pavement distress. The initial data collection effort consisted of gathering pavement condition index (PCI) data for 91 asphalt-surfaced pavement sections and 59 portland cement concrete (PCC) pavement sections. Six asphalt concrete (AC) pavement sections and seven PCC pavement sections were down-selected from the initial data collection effort for further analysis and evaluation. It was observed that climate-related distresses were the most prevalent distress type occurring in the first 10 to 15 years of pavement life. Spalling and joint seal damage were found to be some of the first distresses identified in the life of a PCC pavement. Longitudinal/transverse cracking and weathering were found to be common distresses observed in asphalt pavements. Early-age occurrence of climatic distress types suggests that preventative maintenance techniques or fundamental material changes should be investigated.

1. INTRODUCTION

Both airfield and highway pavements are a significant financial investment that, without periodic maintenance, will not achieve anticipated service life. The American Society of Civil Engineers (ASCE) publishes a national infrastructure report card every four years that uses a letter grade format to evaluate the overall condition of the nation’s infrastructure. In 2017, ASCE reported that the nation’s aviation system (i.e., airport facilities in general) was assigned an overall grade of D and that $157 billion was needed to address funding needs (ASCE, 2017). The ability to forecast required maintenance and delay or mitigate pavement deterioration through improved material selection techniques increases the effective use of limited budgets. In order to achieve this goal, a standardized, repeatable pavement evaluation approach should be employed; and historical data should be reviewed to determine prevalent distress types and mechanisms. Further, these data should be used to direct research initiatives geared at addressing global issues that, if successful, motivate innovative design and maintenance techniques. This paper presents the results of an effort to compile historical pavement
condition data spanning up to 50 years from selected Army airfields for the primary purpose of assessing global pavement deterioration trends and identifying common pavement distress types. The data provided a means to recommend potential preventative maintenance solutions and fundamental material changes that should be investigated.

2. BACKGROUND

A review of available historical reports conducted for this paper indicates that pavement condition surveys of Army airfields and structural evaluations were initiated in the late 1940s and early 1950s. Condition evaluations were performed by local field personnel identifying pavement distresses and providing an overall descriptive classification of pavement condition. In early reports, general descriptions such as “good, fair, poor, failed” were used to describe pavement condition; and few detailed measurements were performed. Structural evaluations consisted of extensive field investigation generally completed through direct-sampling techniques such as test pits and subsequent laboratory characterization and evaluation. These general procedures were followed through the 1970s.

In May 1982, the Department of the Army initiated a program to determine and evaluate the physical properties, load-carrying capacity, and general condition of the pavements at major U.S. Army airfields. This procedure detailed specific pavement distresses, defined methods for physical measurement of each distress, and defined severity levels of each identified distress. The resulting findings are used to calculate an overall Pavement Condition Index (PCI). PCI is a numerical indicator of pavement condition assigned to the following ratings: Good (100-86), Satisfactory (85-71), Fair (70-56), Poor (55-41), Very Poor (40-26), Serious (25-11), and Failed (10-0). Additionally, structural evaluations of airfield pavements became less labor intensive through the use of nondestructive falling and heavy-weight deflectometer (FWD and HWD) devices, thus providing for more data to be collected in less time with greater pavement area coverage. The standard method for performing airfield condition surveys to identify and quantify pavement distress (type, severity, and magnitude) is outlined in ASTM D5340 (ASTM, 2018), and the Department of Defense guidance for conducting a structural evaluation is detailed in UFC 3-260-03 (Headquarters, 2001). To provide repeatability in subsequent pavement evaluations, the entire airfield pavement facility is subdivided into smaller sample sections that are grouped by unique properties such as pavement surface type, pavement thickness, anticipated traffic, and/or repair and construction history. Delineation of unique pavement areas allows for repeated testing and inspection of the same portion of pavement, thus creating a history of pavement condition and deterioration.

Initiation of a specific and standardized evaluation method allowed for generally repeatable and reliable assessment of airfield pavements. The development of the PAVER software package in the late 1970s provided an electronic means of storing and summarizing all the field-collected data.

3. OBJECTIVE AND APPROACH

The main objective of the study was to compile historical condition data (e.g., PCI) and evaluate the general relationship between airfield pavement condition and pavement age. The data were used to observe prevalent trends and identify predominant distress mechanisms to make recommendations for maintenance techniques or potential material changes to extend pavement performance.
Seventeen Army airfields were selected for initial review (Table 1). These were located within the continental United States and represented a cross section of airfield pavements throughout four climatic regions: (1) wet-no freeze, (2) wet-freeze, (3) dry-no freeze, and (4) dry-freeze. Data were selected from multiple climatic regions to limit potential bias introduced by selecting a majority of airfield pavements from a single region. It should be noted that it was not the intent of this study to delineate deterioration based on climatic region but rather to observe global deterioration trends and failure mechanisms.

**TABLE 1. Summary of Airfield Climatic Regions.**

<table>
<thead>
<tr>
<th>Climatic Region</th>
<th>Airfield Designation</th>
<th>State</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet-No Freeze</td>
<td>AAF1</td>
<td>Alabama</td>
</tr>
<tr>
<td>Wet-No Freeze</td>
<td>AAF2</td>
<td>Georgia</td>
</tr>
<tr>
<td>Wet-No Freeze</td>
<td>AAF3</td>
<td>Georgia</td>
</tr>
<tr>
<td>Wet-No Freeze</td>
<td>AAF4</td>
<td>Alabama</td>
</tr>
<tr>
<td>Wet-No Freeze</td>
<td>AAF5</td>
<td>North Carolina</td>
</tr>
<tr>
<td>Dry-No Freeze</td>
<td>AAF6</td>
<td>Texas</td>
</tr>
<tr>
<td>Dry-No Freeze</td>
<td>AAF7</td>
<td>Texas</td>
</tr>
<tr>
<td>Dry-No Freeze</td>
<td>AAF8</td>
<td>Arizona</td>
</tr>
<tr>
<td>Dry-No Freeze</td>
<td>AAF9</td>
<td>California</td>
</tr>
<tr>
<td>Wet-Freeze</td>
<td>AAF10</td>
<td>Missouri</td>
</tr>
<tr>
<td>Wet-Freeze</td>
<td>AAF11</td>
<td>Wisconsin</td>
</tr>
<tr>
<td>Wet-Freeze</td>
<td>AAF12</td>
<td>Maryland</td>
</tr>
<tr>
<td>Wet-Freeze</td>
<td>AAF13</td>
<td>New York</td>
</tr>
<tr>
<td>Wet-Freeze</td>
<td>AAF14</td>
<td>Kentucky</td>
</tr>
<tr>
<td>Dry-Freeze</td>
<td>AAF15</td>
<td>Colorado</td>
</tr>
<tr>
<td>Dry-Freeze</td>
<td>AAF16</td>
<td>Kansas</td>
</tr>
<tr>
<td>Dry-Freeze</td>
<td>AAF17</td>
<td>Washington</td>
</tr>
</tbody>
</table>

AAF = Army Airfield

From each airfield, multiple sections were selected, representing both asphalt and portland cement concrete pavements. PCI and construction history data were collected for each section. Pavement age was based on available construction history and plotted with a PCI number to observe general trends.

4. **RESULTS**

4.1 **Overall Trends**

Figure 1 presents all data points collected for AC pavements. The data shown in this plot represent 91 distinct AC pavement sections. While there is significant scatter in the data, it is observed that most AC pavements experience a generally steep decline in condition from initial construction to age 20, which is the current design life with most AC pavements at or older than 30 years having PCI values that are considered poor to serious (PCI ranging from 55 to 10).

Data values collected for 59 distinct PCC pavement sections are shown in Figure 2. In general, the trend between PCI and pavement age is not as clear as that for asphalt pavements; however, a more gradual decrease in PCI over time can be observed. Reported PCI values become much more variable as the
pavement age increases, a trend which could be a function of maintenance (or the lack thereof) or drastic changes in airfield operations. Condition values of the older pavements ranged from good to failed, and it is noted that some PCC pavements have been in-service for well over 40 years with satisfactory pavement condition ratings.

Review of the plotted PCI for each pavement type generally follows expected trends. Asphalt pavements were found to experience fairly rapid deterioration in condition, and PCC pavements experienced a more gradual decrease in condition. Although the compiled data include maintenance activities, which can increase PCI, these figures provide an indication of the overall deterioration trends.

![Figure 1. Asphalt pavement condition index with pavement age.](image1)

![Figure 2. PCC pavement condition index with pavement age.](image2)
After review of the overall data collection, thirteen pavement sections, summarized in Table 2, were selected for further data collection and analysis. Seven PCC pavement sections were selected with ages ranging from 15 to 56 years, and six AC pavement sections were selected with ages ranging from 4 to 45 years. Note that some sections are repeated in the table of selected asphalt pavements with varying ages. This is due to some major rehabilitation (mill/overlay) and/or reconstruction activity, which was considered to reset the age (in terms of surface condition) of the pavement.

Distress mechanisms were identified by type (climate/durability, load, and other) to evaluate trends over time. Most distress types have three levels of severity: low, medium, and high. Severity level is based on the size of the distress and/or its potential to produce Foreign Object Debris (FOD). A detailed explanation of distress classification and severity level can be found in ASTM D5340 (ASTM, 2018).

**TABLE 2. Pavement Sections Considered for Further Evaluation.**

<table>
<thead>
<tr>
<th>Airfield</th>
<th>Section Type</th>
<th>Age (years)</th>
<th>Geographic Region</th>
<th>Pavement Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>AAF8</td>
<td>Runway</td>
<td>29</td>
<td>Dry-No Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF8</td>
<td>Taxiway</td>
<td>29</td>
<td>Dry-No Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF6</td>
<td>Runway</td>
<td>52</td>
<td>Dry-No Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF13</td>
<td>Runway</td>
<td>56</td>
<td>Wet-Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF13</td>
<td>Runway</td>
<td>15</td>
<td>Wet-Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF3</td>
<td>Runway</td>
<td>21</td>
<td>Wet-No Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF2</td>
<td>Taxiway</td>
<td>55</td>
<td>Wet-No Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF14</td>
<td>Taxiway</td>
<td>21</td>
<td>Wet-Freeze</td>
<td>PCC</td>
</tr>
<tr>
<td>AAF8</td>
<td>Runway</td>
<td>27</td>
<td>Dry-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF7</td>
<td>Taxiway</td>
<td>17</td>
<td>Dry-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF7</td>
<td>Taxiway</td>
<td>15</td>
<td>Dry-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF7</td>
<td>Taxiway</td>
<td>13</td>
<td>Dry-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF7</td>
<td>Taxiway</td>
<td>45</td>
<td>Dry-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF13</td>
<td>Taxiway</td>
<td>21</td>
<td>Wet-Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF14</td>
<td>Runway</td>
<td>21</td>
<td>Wet-Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF2</td>
<td>Runway</td>
<td>19</td>
<td>Wet-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF2</td>
<td>Runway</td>
<td>9</td>
<td>Wet-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF2</td>
<td>Runway</td>
<td>28</td>
<td>Wet-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF4</td>
<td>Taxiway</td>
<td>21</td>
<td>Wet-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF4</td>
<td>Taxiway</td>
<td>4</td>
<td>Wet-No Freeze</td>
<td>AC</td>
</tr>
<tr>
<td>AAF4</td>
<td>Taxiway</td>
<td>25</td>
<td>Wet-No Freeze</td>
<td>AC</td>
</tr>
</tbody>
</table>

PCC = portland cement concrete; AC = asphalt concrete

4.2 PCC Pavement Condition Index

Each PCC distress type, as defined in ASTM D5340 (ASTM, 2018), is organized in Table 3 by distress mechanism. Figure 3 displays the contribution of each distress mechanism associated with various ages for the PCC pavements. The data presented represent a compilation of all selected PCC pavements and the associated distress contribution observed for each evaluation. The data indicate that the initial types of distress occurring in PCC pavements are identified as climate/durability and other.
distresses generally occur early in the pavement life (from 0 to approximately 15 years). Load-related distresses generally start to occur around 16 years and trend upward to peak in the 40-year range. Note that one pavement section showed load-related distress one year after construction, an apparent exception to the general trend. After 40 years of pavement age, the load-related distress mechanism experiences a downward trend, which is likely due to maintenance operations that typically consist of total slab replacement if distresses are considered medium to high severity.

TABLE 3. PCC Distress Type by Distress Mechanism.

<table>
<thead>
<tr>
<th>Climate/Durability</th>
<th>Load</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blow Up</td>
<td>Corner Break</td>
<td>Small Patch</td>
</tr>
<tr>
<td>Durability Cracking</td>
<td>Linear Cracking</td>
<td>Large Patch/Utility</td>
</tr>
<tr>
<td>Joint Seal Damage</td>
<td>Shattered Slab</td>
<td>Popouts</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Pumping</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Scaling</td>
</tr>
</tbody>
</table>

Table 4 shows an example PCI deterioration graph with pavement age and typical distress mechanisms for a PCC pavement section (AAF8 Runway is shown). Other PCC pavements displayed similar trends and distress types. It was found that joint and corner spalling were recurring distress types and were first documented from 2 to 6 years after initial construction. In general, joint and corner spall severity was found to progress to a higher severity level (i.e., low to medium or medium to high) over a period of approximately 4 years. Joint seal damage was also identified as a common distress across all PCC
pavements and was first observed 2 to 4 years after construction. Distress severity level was found to increase after a time span of approximately 9 to 10 years.

For PCC pavements, note that repair activities (i.e., spall repair or crack sealing) are measured as pavement defects. This results in the PCC pavements’ never returning to a PCI of 100. However, review of the data indicates that the rate of deterioration of most PCC pavements was gradual in nature.

**TABLE 4. PCC Typical Deterioration and Distress Types with Time.**

<table>
<thead>
<tr>
<th>Age (years)</th>
<th>Load</th>
<th>Climate</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>New Construction</td>
<td>New Construction</td>
<td>New Construction</td>
</tr>
<tr>
<td>2</td>
<td>Linear Cracking (L)</td>
<td>None</td>
<td>Joint Spalling (L), Corner Spalling (L)</td>
</tr>
<tr>
<td>10</td>
<td>Not reported</td>
<td>Not reported</td>
<td>Not reported</td>
</tr>
<tr>
<td>17</td>
<td>Linear Cracking (L)</td>
<td>Joint Seal Damage (L)</td>
<td>Small Patch (L), Shrinkage Cracking (NA), Joint Spalling (L)</td>
</tr>
<tr>
<td>21</td>
<td>Linear Cracking (L)</td>
<td>Joint Seal Damage (L)</td>
<td>Small Patch (L), Small Patch (M), Shrinkage Cracking (NA)</td>
</tr>
<tr>
<td>25</td>
<td>Linear Cracking (L)</td>
<td>Joint Seal Damage (L)</td>
<td>Small Patch (L), Small Patch (M), Shrinkage Cracking (NA), Joint Spalling (L), Corner Spalling (L)</td>
</tr>
<tr>
<td>29</td>
<td>Linear Cracking (L)</td>
<td>Joint Seal Damage (L)</td>
<td>Small Patch (L), Small Patch (M), Shrinkage Cracking (NA), Joint Spalling (L)</td>
</tr>
</tbody>
</table>

L = low; M = medium, NA = not applicable

4.3 Asphalt Pavement Condition Index

Each asphalt distress type, as defined in ASTM D5340 (ASTM, 2018), is organized in Table 5 by distress mechanism. Distress mechanism with pavement age is presented graphically in Figure 4. The data presented represent a compilation of all selected AC pavements and the associated distress contribution.
observed for each evaluation. Review of the selected data indicates that climate-related distresses are the primary contributor to PCI reduction for the first 10 to 12 years of service life and remain a major contributor for most of the identified pavement age. Load-induced distress mechanisms were found to generally begin in the 14- to 15-year age range and peak at the 20- to 21-year age range. A decrease in load-related distress was noted after approximately 20 years, likely due to some form of major rehabilitation.

A review of historical data indicates relationships between asphalt pavement age and surface deterioration (PCI) are more difficult to define. It was found that repair and/or maintenance activities tend to occur frequently, in turn causing multiple increases in condition index values. In general, the data indicate asphalt-surfaced pavements do not approach a 20-year life with regards to PCI without major maintenance.

Table 5. Asphalt Distress Type by Distress Mechanism.

<table>
<thead>
<tr>
<th>Climate/Durability</th>
<th>Load</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block Cracking</td>
<td>Alligator Cracking</td>
<td>Bleeding</td>
</tr>
<tr>
<td>Joint Reflection Cracking</td>
<td>Rutting</td>
<td>Corrugation</td>
</tr>
<tr>
<td>Longitudinal/Transverse</td>
<td></td>
<td>Depression</td>
</tr>
<tr>
<td>Patching</td>
<td></td>
<td>Jet Blast</td>
</tr>
<tr>
<td>Raveling</td>
<td></td>
<td>Oil Spillage</td>
</tr>
<tr>
<td>Weathering</td>
<td></td>
<td>Polished</td>
</tr>
</tbody>
</table>

Table 6 shows an example PCI deterioration graph with pavement age and typical distress mechanisms for an asphalt pavement section (AAF7 Taxiway is shown).
Asphalt overlays will return the PCI to a value of near 100, essentially resetting the deterioration curve, as can be seen at year 17 and 32 in Table 6. However, underlying distresses (e.g., fatigue cracking, rutting, or loss of subgrade support due to rapid changes in subgrade moisture content from poor drainage or utility leaks) may increase the rate of distress seen after maintenance activities.

Longitudinal and transverse cracking were found to be the most common and recurring AC pavement distresses, typically first observed in the 2- to 4-year time frame after construction or rehabilitation. An increase in severity level was commonly observed over an 8- to 10-year time period. Weathering was also a common distress type, generally identified 9 to 10 years after construction.

5. OBSERVATIONS FROM PCI TRENDS

It was observed from both PCC and AC datasets that non-load related distresses are the primary contributor to pavement deterioration, particularly early in pavement life. Current maintenance practices are generally reactive in nature; however, the results suggest that preventative techniques or material changes should be pursued to mitigate or reduce these distresses on airfield pavements.

One example is the application of a pavement sealer to reduce or mitigate the potential for weathering on asphalt pavements. Research (Rushing et al, 2015) has shown that pavement preservation techniques, such as surface treatments, can be used in low-speed airfield applications such as aprons, taxiways, or helipads. Further, Rushing et al. (Rushing et al, 2015) concluded that some surface treatments are effective in retarding longitudinal/transverse cracking.

Another option to mitigate climate-related distress on asphalt pavements, particular cracking, takes a more fundamental approach. Early-age cracking can indicate that current airfield asphalt mix design procedures do not supply enough asphalt binder to the mix to maintain flexibility and resist cracking. The highway paving industry is investigating balanced mix design procedures, in which optimum asphalt binder content is selected based on a balance between rutting performance and cracking performance, although a definitive laboratory cracking test is still needed. Al-Qadi et al. (Al-Qadi et al, 2015) developed the semi-circular bend (SCB) Illinois Flexibility Index Test (I-FIT), which is a laboratory cracking test, to evaluate asphalt mixture brittleness. The research determined that mixtures with a flexibility index (FI) less than 2 were the worst performers, and mixtures with an FI value greater than 6 were the best performers. Bennert et al. (Bennert et al, 2017) applied similar methodology to perform a forensic investigation of pavements on the JFK and Newark International airport runways. Pavements with FI values greater than 6 were found to have little to no observed cracking, whereas poor cracking performance was observed in pavements with FI values less than 4. A similar approach could be used for military airfield asphalt pavements, since the data do not indicate that load-related distresses are a widespread problem.

A similar approach could be pursued for PCC pavements. Poole and Rollings (Poole and Rollings, 2009) suggested that in-place air content and air content variability could be a contributor to joint spalling and that research was needed to investigate differences between air content of fresh concrete and hardened concrete. It was suggested that higher air contents in fresh concrete may be needed to account for lower in-place air content values. Further, Harrington et al. (Harrington et al, 2018)
suggested that the air void spacing factor (i.e., the distance water must travel to reach an air void) impacts durability. Additionally, Poole and Rollings (Poole and Rollings, 2009) recommended that a detailed investigation into the role of deicing salts on aggregate durability be conducted.

Table 6. Asphalt Typical Deterioration and Distress Types with Time.

<table>
<thead>
<tr>
<th>Age</th>
<th>Load</th>
<th>Climate</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>15</td>
<td>None</td>
<td>Longitudinal/Transverse Cracking (L, M)</td>
<td>None</td>
</tr>
<tr>
<td>17</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>19</td>
<td>None</td>
<td>Longitudinal/Transverse Cracking (L), Weathering (L)</td>
<td>None</td>
</tr>
<tr>
<td>24</td>
<td>None</td>
<td>Longitudinal/Transverse Cracking (L, M), Weathering/Raveling (L)</td>
<td>None</td>
</tr>
<tr>
<td>26</td>
<td>Alligator Cracking (L)</td>
<td>Longitudinal/Transverse Cracking (L, M), Block Cracking (L), Weathering/Raveling (L)</td>
<td>None</td>
</tr>
<tr>
<td>31</td>
<td>Alligator Cracking (L)</td>
<td>Longitudinal/Transverse Cracking (L, M), Block Cracking (L), Weathering/Raveling (L)</td>
<td>None</td>
</tr>
<tr>
<td>32</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>36</td>
<td>None</td>
<td>Longitudinal/Transverse Cracking (L)</td>
<td>None</td>
</tr>
<tr>
<td>40</td>
<td>None</td>
<td>Longitudinal/Transverse Cracking (L)</td>
<td>Oil Spillage (NA)</td>
</tr>
<tr>
<td>45</td>
<td>None</td>
<td>Longitudinal/Transverse Cracking (L), Weathering (L, M)</td>
<td>Oil Spillage (NA), Slippage</td>
</tr>
</tbody>
</table>

L = low; M = medium, NA = not applicable
6. CONCLUSIONS

This paper presented the results of an effort to compile historical data from selected Army airfields and populate a summary database. The initial data collection effort consisted of collecting PCI data for 91 AC pavement sections and 59 PCC sections. Six AC pavement sections and seven PCC pavement sections were selected from the initial data collection effort for further analysis and evaluation. The following are the main findings from this study.

PCC Pavement Findings

- The dataset indicates that the selected PCC pavements generally should meet or exceed a 40-year PCI performance expectation.
- Non-load classified distresses were found to be the primary distress mechanism identified during the first 15 years. Load-related distresses were found to occur first around year 16 and trend upward to peak at year 40. A decrease in load-related distresses was observed after year 40, likely due to maintenance operations.
- Joint and corner spalling was found to be a recurring distress identified and first documented from 2 to 6 years after initial construction. In general, the distress severity increased to a higher level over a period of approximately 4 years.
- Joint seal damage was found to be a common distress across the selected PCC pavements and was generally identified 2 to 4 years after construction. The distress severity level generally increased after 9 to 10 years.

Asphalt Pavement Findings

- Observed PCI trends indicate that the selected AC pavements do not approach a 20-year PCI age without some form of major rehabilitation (overlay or mill/overlay).
- Climate-related distresses were found to be the primary distress mechanism for the first 10 to 12 years and remained a major contributor for most of the pavement age.
- Load-related distresses in AC pavements were found to begin generally in the 14- to 15-year pavement age range and peak around 20 years. A decrease in load distresses was observed after 20 years, likely due to rehabilitation.
- Longitudinal/transverse cracking was found to be a recurring AC pavement distress, generally first observed in the 2- to 4-year time frame. An increase in severity level was observed over an 8- to 10-year time period.
- Weathering was also a common climate distress type, generally identified 9 to 10 years after construction.

ACKNOWLEDGEMENTS

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REFERENCES


Implementing Pavement Preservation into Pavement Management Systems

J.F. Rushing and W.J. Robinson
US Army Engineer Research and Development Center

ABSTRACT

Many agencies are now focusing on pavement preservation methods as a more economical means of extending the life of transportation infrastructure. However, pavement management systems are often designed with condition assessment reports that highlight failure according to the agency's minimum rating criteria. The Department of Defense (DoD) has such minimum criteria for airfield pavements that constitutes policy for which runways, taxiways, or aprons must be maintained. The DoD uses the PAVER pavement management software to input inspection data, to calculate pavement condition index, and to develop work plans that address failing pavements. After several years of researching pavement preservation techniques for airfield asphalt pavements, the DoD decided to incorporate pavement preservation strategies into PAVER software to produce work plans that address pavements in good condition and that are excellent candidates for low-cost treatments that can extend the periods between major rehabilitation. The work plans are generated by eliminating pavement inventory that are poor candidates for preservation activities. For example, asphalt pavements experiencing medium-severity block cracking would generally not be a candidate for preservation, as any type of preventive maintenance treatment would be ineffective at reversing long-term effects of aging. This paper details the process of selecting the candidate pavements for preservation treatments and discusses the range of values for each asphalt distress used in selection process, along with elements of the generated work plan that create a comprehensive approach to implementing pavement preservation into pavement management.

1. INTRODUCTION

Environmental deterioration is a major factor leading to required rehabilitation of asphalt concrete pavements on airfields. Current design procedures used for airfields rarely lead to structural failures. In fact, Rushing et. al (2014) reviewed historical data from Army airfields and reported that only 5% of distresses existing in pavements ranging from 7 to 15 years old were load-related based upon pavement condition index (PCI) surveys. Meanwhile, the distresses related to environment-related degradation can cause large deductions in the PCI values. For example, the presence of medium-severity weathering over an area of airfield pavement can cause a PCI deduction of over 20 points (on the 0-100 point scale). The Department of Defense (DoD) mandates minimum threshold values for pavement condition, often leading to required maintenance resulting from environmental damage alone. Pavement preservation techniques are needed that can combat these types of distresses and maintain pavements in adequate condition for longer periods of time to reduce the overall life-cycle cost.

1.1 Background pavement preservation studies

Pavement preservation products have been commercially available for many years. Numerous reports document benefits and drawbacks of these products, including some by U.S. Army Corps of Engineer
researchers studying methods to preserve airfield asphalt concrete pavements. Brown and Johnson (1976) studied five different asphalt rejuvenators during a period from 1971 to 1975. This study included field applications on three different Air Force bases in the southeastern, southwestern, and northern U.S. Target pavements were approximately 10 years old that had been free from maintenance. Test methods included skid resistance, penetration and viscosity tests on extracted asphalt binder, crack count, and surface texture analysis. Three of the five products decreased viscosity and increased penetration of the binder compared to untreated areas after three years, indicating softening of the binder was achieved. The same products reduced the number of cracks that were greater than 0.25 in. wide observed in the pavement. Further, all products reduced the loss of fines from the pavement surface, supporting the claims of reduction in environmental weathering. However, all products caused a decrease in skid resistance for a prolonged period of time (up to 2 years) which create the need to balance safety concerns with pavement life extension.

Shoenberger (2003) performed a similar laboratory and field study on 16 rejuvenators, rejuvenator/sealers, and seal coats at two Air Force bases during a period of more than one year. Materials were characterized to determine the effects of treatment on extracted and recovered asphalt binder as well as the impacts on surface texture and skid resistance. These results supported those concluded by Brown and Johnson that some products did reduce asphalt binder viscosity from recovered field samples. However, most of these products reduced the friction of the pavement surface which continues to be the general consensus of researchers today.

Rushing (2009) provide information on usage and effectiveness of multiple types of pavement preservation products placed at DoD installations in the U.S. and Europe. While visual in nature, these inspections revealed inconsistent use of pavement sealer types at different installations and high dependency on the locally-available products. Navy researchers conducted a detailed field study from 2007 to 2008 to evaluate a gilsonite-based asphalt sealer at multiple locations throughout the U.S. (Cline, 2011). Results indicated improved life-cycle cost by using preventive maintenance treatments. Further, updates were recommended to Unified Facilities Criteria that would recommend or require the use of pavement preservation in pavement management.

In 2009 the U.S. Army Engineer Research and Development Center (ERDC) began a 5-year study on asphalt surface treatments for the Army to identify the best types of products for preserving pavements in good conditions. Field sites were selected on airfields at Fort Hood (Texas), Fort Benning (Georgia), and Fort Drum (New York) where fourteen different products were installed and monitored. Two years later the ERDC expanded on this study for the Air Force, applying nine materials on airfields at Davis-Monthan (Arizona), Dover (Delaware), and Hurlburt Field (Florida) Air Force bases for long-term observation. A comprehensive friction study during the initial curing period provided data for addressing safety concerns with these types of products. Since these controlled field studies, several DoD installations have performed large-scale treatment projects using different sealers. Runways at Edwards Air Force Base (California), Barking Sands Pacific Missile Range (Hawaii), the U.S. Air Force Academy (Colorado), and Naval Air Station Grotagglie (Italy) all have received asphalt surface treatments. Continued monitoring of these test sites will provide data for adjusting policy guidance in the future.
1.2 Objective and scope

The objective of this paper is to present the methodology used to establish preliminary criteria for using preservation products on asphalt concrete airfield pavements and to implement this criteria into currently-used pavement management software. The criteria selected for identifying candidate pavements for preservation was created through consensus of pavement engineers within the DoD (Air Force, Army, Navy). The criteria was based upon identifying appropriate pavement condition, time between treatments, and required maintenance prerequisites. The scope of the project included any airfield pavements surfaced with asphalt concrete that are included in the routine pavement condition inspections performed by each respective agency. The PAVER software was used as the framework for implementation based upon its mandated use for managing the airfield pavement infrastructure. Criteria implementation was achieved by creating a module within the PAVER software that would review pavement inspection and construction history data and then screen these data according to the criteria. Multiple options for preservation treatment types may be identified in the process, requiring the user to balance cost and effectiveness given the local funding availability and pavement maintenance needs. Figure 1 illustrates the process by which the criteria is used within the pavement management software.

![Figure 1. Process by which pavement preservation treatments are suggested based upon condition.](image)

2. CATEGORIES OF PRESERVATION TREATMENTS

For the purpose of this paper, there are three categories of preservation treatments that are based upon their application method. These include liquid only sprays, liquid and sand sprays, and slurry-applied materials. The liquid only sprays are generally fog seal or rejuvenator products applied at rates commonly ranging from 0.05 to 0.1 gallons per square yard. Some are intended to penetrate the surface of the asphalt concrete and soften the binder while others serve to act as a surface barrier and binding agent. Liquid and sand sprays are those surface applications that include a mineral filler, that are applied at heavier rates (typically between 0.1 and 0.2 gallons per square yard), and that create a surface barrier
over the pavement substrate. These materials may include fuel-resistant sealers as well. Finally, slurry-applied products provide a new thin wearing surface based on the gradation and shape of the aggregate in the slurry and the designed mixture.

3. DISTRESS EXCLUSION POLICY AND PRE-TREATMENT MAINTENANCE REQUIREMENTS

The overall procedure for selecting the appropriate type of surface treatment is based on a distress exclusion policy. This principle eliminates undesirable candidates from consideration based upon the presence of certain types, severity, and quantity of identified distresses. For example, alligator cracking is a common pavement distress that is caused by inadequate structural capacity. Preservation treatments are not effective at mitigating this type of distress, so the presence of alligator cracking above threshold values may result in a treatment type being excluded for consideration given that the pavement needs different rehabilitation strategies to improve the numerical PCI rating.

Each of the asphalt concrete distresses listed in ASTM D5340, Standard Test Method for Airport Pavement Condition Index Surveys, was assigned an exclusion value if applicable. Additionally, the exclusion values were based upon the severity of the distress. Using this method allows for the direct importing of field data into the algorithms that screen the data for appropriate preservation treatments. This enables current software to provide recommendations. For pavement databases absent of certain distresses, the exclusion table can be bypassed.

Asphalt concrete distresses that did not influence the decision on using a liquid spray product included bleeding, low-severity block cracking, jet blast erosion, low-severity longitudinal and transverse cracking, patching, polished aggregate, raveling, low-severity swells, and weathering. For liquid and sand sprays as well as slurry-applied products, limitations were included on the presence of low-severity block or longitudinal and transverse cracking. These types of products were restricted because they would be less likely to fill the cracks and would be prone to having the cracks reflect to the surface. The remaining distresses contained an exclusion criteria for all product types.

Alligator cracking was limited to 5 percent for the total of all severity levels (low, medium, and high) and 2 percent for the area having medium and high severity. Being a load-related distress, alligator cracking is likely confined to the wheel path and only affects a portion of the pavement. Any persistent occurrence of this distress indicates structural capacity limitation and does not warrant the use of surface treatments, because rehabilitation is likely required.

Block cracking as well as longitudinal and transverse cracking were given an exclusion value for all product types if it occurred as medium or high severity. The threshold value for these types of cracking changed based upon treatment type. Because block cracking and longitudinal and transverse cracking are indicative of significant environmental deterioration, their presence greatly restricted the use of surface treatments. If the pavement binder has aged to the point where this type of cracking is occurring, it is unlikely that surface treatments will extend the usable life. Further, slurry-applied treatments may flake around reflected cracks and cause the risk of foreign object damage (FOD) to aircraft.
Corrugations and depressions were limited for all treatment types. It is unlikely that an airfield pavement section would experience large quantities of either of these distresses. However, if present, it would be difficult for either liquid sprays or slurries to be applied evenly. Ponding in the low spots of corrugated or depressed pavement could cause incomplete product curing that could lead to tracking of material.

Joint reflection cracking was limited to 10 percent total and 5 percent medium and high severity for all treatment types. Once this type of distress is observed, surface treatments are unlikely to limit its increase in quantity or severity. Cracks will propagate to the surface of the pavement and have potential to produce FOD. Asphalt concrete overlaying Portland cement concrete is often a poor candidate for pavement preservation because of the propensity for this distress to occur.

Oil spillage was given an exclusion value of 20 percent. Pavements experiencing greater than this quantity of oil spillage have likely endured significant binder damage. Additionally, surface treatments may trap unwanted oils in the pavement. Fuel-resistant sealers should be placed on airfield pavements prior to spillage occurring above this limit.

Rutting was restricted to prevent ponding of the surface treatments in the low spots of the pavement. Rutting is not differentiated by its relation to asphalt mixture problems or subsurface structural issues. If it were asphalt mixture related, a surface treatment would not be desirable because it could keep the mixture tender for longer periods of time. The restriction for slurry-applied products was lower than for spray seals because a slurry type product can help to level the pavement surface.

Shoving was limited to 10 percent for all treatment types. It is unlikely that this distress would occur in greater quantities over a broad area, but if it did, the pavement could potential suffer from mixture issues that could be exacerbated by using a surface treatment.

Slippage cracking was limited to 10 percent of the total area. Except in the cases of an unusual event, the presence of slippage cracking is often indicative of horizontal shear failure due to lack of bond between pavement layers. Pavements exhibiting significant quantities of this distress should be slated for rehabilitation efforts.

Swells were limited to 20 percent for medium and high severity. While some surface deviations would not cause great concern, the presence of greater than low severity swells typically indicates significant frost action or swelling soils that need corrective maintenance.

Pavements having distresses below threshold exclusion values were identified as candidates for preservation treatments. However, some maintenance is required prior to applying the surface treatment in many cases. Surface treatments are inherently limited by their ability only to address certain types of pavement distresses. Therefore, it is necessary to perform additional maintenance prior to their application to enhance effectiveness. Typical maintenance includes sealing medium- and high-severity cracking as well as patching. Each of the asphalt concrete distresses was reviewed to determine if maintenance should be performed prior to applying a surface treatment. For those requiring
maintenance prior to applying surface treatments, the distress database from field surveys can be used to generate work plans based upon the quantity of the subject distress reported.

Table 1 provides the distress exclusion table and pre-treatment maintenance requirements for liquid only spray seals. Table 2 provides the requirements for liquid and sand spray seals, and Table 3 provides the requirements for slurry-applied products.

Table 1. Distress exclusions and pre-treatment maintenance requirements for liquid only spray seals.

<table>
<thead>
<tr>
<th>Distress</th>
<th>Severity</th>
<th>Exclude if Density &gt; percent</th>
<th>Maintenance Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alligator Cracking</td>
<td>L+M+H</td>
<td>5</td>
<td>Surface crack seal low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>2</td>
<td>Full depth patching</td>
</tr>
<tr>
<td>Bleeding</td>
<td>n/a</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Block Cracking</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>10</td>
<td>Seal cracks</td>
</tr>
<tr>
<td>Corrugation</td>
<td>L+M+H</td>
<td>20</td>
<td>Grind or patch</td>
</tr>
<tr>
<td>Depression</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>5</td>
<td>Patch</td>
</tr>
<tr>
<td>Jet Blast Erosion</td>
<td>n/a</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Joint Reflection Cracking</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>5</td>
<td>Seal cracks</td>
</tr>
<tr>
<td>L&amp;T Cracking</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>25</td>
<td>Seal cracks</td>
</tr>
<tr>
<td>Oil Spillage</td>
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<td>20</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Patching</td>
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<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Polished Aggregate</td>
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<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Raveling</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Rutting</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>5</td>
<td>Patch</td>
</tr>
<tr>
<td>Shoving</td>
<td>L+M+H</td>
<td>10</td>
<td>Grind or patch</td>
</tr>
<tr>
<td>Slippage Cracking</td>
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<td>10</td>
<td>Patch</td>
</tr>
<tr>
<td>Swell</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>20</td>
<td>Patch</td>
</tr>
<tr>
<td>Weathering</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing</td>
</tr>
</tbody>
</table>

* L = low; M = medium; H = high; n/a = not applicable (no severity levels associated with distress)
Table 2. Distress exclusions and pre-treatment maintenance requirements for liquid and sand spray seals.

<table>
<thead>
<tr>
<th>Distress</th>
<th>Severity</th>
<th>Exclude if Density &gt; percent</th>
<th>Maintenance Requirement</th>
</tr>
</thead>
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<tr>
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<td>Surface crack seal low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>2</td>
<td>Full depth patching</td>
</tr>
<tr>
<td>Bleeding</td>
<td>n/a</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Block Cracking</td>
<td>L+M+H</td>
<td>25</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>10</td>
<td>Seal cracks</td>
</tr>
<tr>
<td>Corrugation</td>
<td>L+M+H</td>
<td>20</td>
<td>Grind or patch</td>
</tr>
<tr>
<td>Depression</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>5</td>
<td>Patch</td>
</tr>
<tr>
<td>Jet Blast Erosion</td>
<td>n/a</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Joint Reflection Cracking</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>5</td>
<td>Seal cracks</td>
</tr>
<tr>
<td>L&amp;T Cracking</td>
<td>L+M+H</td>
<td>25</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>10</td>
<td>Seal cracks</td>
</tr>
<tr>
<td>Oil Spillage</td>
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<td>20</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Patching</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Polished Aggregate</td>
<td>n/a</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Raveling</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Rutting</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>5</td>
<td>Patch</td>
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<tr>
<td>Shoving</td>
<td>L+M+H</td>
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<td>Grind or patch</td>
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<tr>
<td>Slippage Cracking</td>
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<td>10</td>
<td>Patch</td>
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<tr>
<td>Swell</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td></td>
<td>M+H</td>
<td>20</td>
<td>Patch</td>
</tr>
<tr>
<td>Weathering</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing</td>
</tr>
</tbody>
</table>

* L = low; M = medium; H = high; n/a = not applicable (no severity levels associated with distress)
Table 3. Distress exclusions and pre-treatment maintenance requirements for slurry-applied seals.

<table>
<thead>
<tr>
<th>Distress</th>
<th>Severity</th>
<th>Exclude if Density &gt; percent</th>
<th>Maintenance Requirement</th>
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<tr>
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<td>Surface crack seal low only</td>
</tr>
<tr>
<td>Bleeding</td>
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<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Block Cracking</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td>Corrugation</td>
<td>L+M+H</td>
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<td>Grind or patch</td>
</tr>
<tr>
<td>Depression</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td>Jet Blast Erosion</td>
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<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Joint Reflection Cracking</td>
<td>L+M+H</td>
<td>10</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td>L&amp;T Cracking</td>
<td>L+M+H</td>
<td>25</td>
<td>Do nothing low only</td>
</tr>
<tr>
<td>Oil Spillage</td>
<td>n/a</td>
<td>20</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Patching</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Polished Aggregate</td>
<td>n/a</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
<td>Raveling</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing</td>
</tr>
<tr>
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<td>L+M+H</td>
<td>25</td>
<td>Do nothing low only</td>
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<td>Shoving</td>
<td>L+M+H</td>
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<td>Grind or patch</td>
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<td>Slippage Cracking</td>
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<td>10</td>
<td>Patch</td>
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<tr>
<td>Swell</td>
<td>L+M+H</td>
<td>100</td>
<td>Do nothing low only</td>
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<tr>
<td>Weathering</td>
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<td>100</td>
<td>Do nothing</td>
</tr>
</tbody>
</table>

* L = low; M = medium; H = high; n/a = not applicable (no severity levels associated with distress)

4. IMPLEMENTATION INTO PAVER

PAVER is the DoD’s software system used for pavement management. One function of PAVER is to create PCI reports based upon user input of distress types, severity, and density in accordance with ASTM D5340. Additional functionality of PAVER includes the ability to create work plans, cost estimates, and predicted future pavement condition reports. These functions allow owners to prioritize funding to
ensure pavements meet minimum policy thresholds for rating condition and to schedule maintenance and rehabilitation efforts.

PAVER offers a direct implementation of pavement preservation criteria because reported individual pavement distresses are already housed within the database from the latest inspection. DoD policy requires all airfield pavements to be inspected periodically in accordance with the ASTM procedure. In addition to pavement distress data, PAVER also stores construction history data that can be used to guide users on appropriate timing between preservation treatments. The time between preservation treatments is based upon the type of material used to promote maximum effectiveness. While preservation treatments are most effective early in a pavement life, application frequency should be limited to prevent excessive buildup of material at the pavement surface. When available, pavement friction data can be used to ensure safety considerations are incorporated into the decision making process. Table 4 provides some overall pavement characteristics included in the criteria to guide users to the most effective implementation of a preservation strategy. These characteristics include the time since the last treatment (global) or overlay (major), the overall PCI, the percentage of distresses attributed to load or climate, and the pavement friction.

Table 4. Overall pavement characteristics for selection of preservation treatments.

<table>
<thead>
<tr>
<th></th>
<th>Liquid only spray seals</th>
<th>Liquid + sand spray seals</th>
<th>Slurry applied seals</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>minim m</td>
<td>maxim m</td>
<td>minim m</td>
</tr>
<tr>
<td>Time since last global</td>
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<td>100</td>
<td>4</td>
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<tr>
<td>Time since last major</td>
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<td>PCI</td>
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<td>50</td>
</tr>
<tr>
<td>% load</td>
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<td>20</td>
<td>0</td>
</tr>
<tr>
<td>% climate</td>
<td>50</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>mu value (40 mph)*</td>
<td>0.6</td>
<td>1</td>
<td>0.6</td>
</tr>
</tbody>
</table>

* Performed in accordance with method defined in Federal Aviation Administration Advisory Circular 150/5320-12, Measurement, Construction, and Maintenance of Skid Resistant Airport Pavement Surfaces

5. SUMMARY AND RECOMMENDATIONS

In summary, pavement preservation strategies can be effective at maintaining pavements in adequate conditions for extended periods of time. These strategies should be incorporated into overall pavement management programs to establish data-based policy for triggering the timing of such treatments. Using the PAVER software program, the DoD has established a methodology for users to evaluate an overall facility and to prioritize funding for preservation treatments. This procedure uses inspection data to
screen pavement sections, to exclude those needing corrective maintenance or rehabilitation, and to establish overall pavement characteristics leading to the selection of an appropriate treatment type. Coupled with the ability to forecast pavement condition and establish long-term work plans, pavement management using PAVER can be optimized by incorporating pavement preservation into a comprehensive pavement management program. The initial criteria reported herein should be routinely evaluated and adjusted based upon input from local installations. Selection of products used for preservation treatment should be based upon local material availability, cost, and past performance.

6. ACKNOWLEDGEMENTS

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REFERENCES

Development of a Mechanistic Approach to Quantify Pavement Damage Using Axle Load Spectra from South Texas Overload Corridors

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The University of Texas at El Paso, Civil Engineering Department

ABSTRACT

Traditionally the damages imparted by heavy vehicles on transportation infrastructures are quantified using the Axle Load Equivalency Factors (ALEFs). Due to the nature of assumptions in the development of such equivalent load factors, the observed damages in the field are vastly different from numerical predictions using the traditional Asphalt Institute ALEF values. This was the motivation to develop a mechanistic approach to properly quantify the damage factors associated with the loading characteristics, environmental conditions, and pavement profile for each specific site in the studied network. To achieve this objective, the authors installed portable Weigh-In-Motion (WIM) devices in South Texas overload corridors to develop an axle load spectra database. Later, the research team deployed Ground Penetrating Radar (GPR) and Falling Weight Deflectometer (FWD) to ten representative sites for structural evaluation of pavement sections. Subsequently, the back-calculated layer moduli and layer configurations from the field tests, and the loading characteristics from the portable WIM devices were incorporated in a 3D finite element model for response calculations of the pavements subjected to taxing stress paths imparted by over-weight truck traffic in the network. The authors proposed three different approaches for the determination of the modified damage factors in this study. The proposed mechanistic approach for the calculation of the site-specific axle load factors resulted in the significantly higher prediction of the damage factors in Farm to Market (FM) roads subjected to over-weight trucks.

1. INTRODUCTION

Texas has experienced a boom in the production of energy-related activities such as natural gas and crude oil since 2008. These energy development activities have created large volumes of traffic operations in the network that adversely impacted the transportation infrastructure systems such as pavements and bridges. Specifically, in the South Texas region, damaged local and county roads have been a major source of inconvenience for the local residents. Lack of accurate assessment of the problem coupled with unclear guidelines is among the many elements that contribute to the delay of the pavement maintenance and repair in several counties. The quantification of the energy development impacts on the highway network is the prelude to adopting proper rehabilitation strategies to meet the future growth of traffic in overload corridors. This can effectively protect the taxpayers’ money spent on transportation systems each year. Consequently, there is a pressing need to properly quantify the damages imparted by the energy-related vehicles in the affected network.

The main step in the quantification of the pavement damage induced by energy-related operations is by the determination of Axle Load Equivalency Factor (ALEF), defined as the quantification of the damage per pass to a pavement section relative to that of a standard 18-kip single axle. The ALEF values are...
primarily dependent on the pavement types, pavement layers, layer thicknesses and structural integrity of the pavement layers in the network. Current ALEFs are based on the AASHTO formulations with several simplifying assumptions for generalization across the nation. The major problem that pavement designers and other professionals face is that the analysis procedures rely on experimental information that was developed from field measurements in the late 1950s and early 1960s. Moreover, the Road Test was limited to one location and had limited traffic levels in comparison to the current traffic loading conditions. The relationships that were established from the AASHTO Road Test might be suitable for similar conditions under which the tests were conducted; however, the relationships were also extrapolated to conditions that the original tests did not account for. Therefore, considering the fact that each network has its own specific characteristics, it deems necessary to develop a new set of ALEFs that accounts for the site-specific traffic loading conditions as well as the pavement layer properties attributed to the overload corridors in the energy sector zones.

Pavement damage quantification has been previously documented in several studies. Batioja-Alvarez et al. (2018) developed a probabilistic approach to quantify the pavement damage induced by Over-Weight Over-Size (OW/OS) trucks, calculating Load Equivalency Factors (LEFs) based on two failure criteria (fatigue and rutting failure). The authors used 3-D Move software to evaluate pavement responses induced by the heavy axle load passages. Traffic loading conditions were assumed based on the previous permits issued. The results showed that fatigue cracking-based LEFs attributed to the OW vehicles were higher than rutting-based LEFs. Based on the quantified damage imparted to the pavement structure, the researchers also calculated the pavement damage associated costs attributed to the passage of OW trucks. Banerjee et al. (2015) developed a methodology to determine load equivalency factors for various axle configurations and loads with a focus on oversize and overweight vehicles. They used a mechanistic-empirical approach to estimate the deterioration of the pavement structure. The authors provided the basis for developing axle-specific equivalency factors associated with the rutting and fatigue cracking. Sadeghi et al. (2007) evaluated the influence of overloading on the operational life of flexible pavements. The authors developed a deterioration model that is used for proposing a cost recovery method. They found that in most cases the asphalt layer tensile strain was the critical parameter. Wu et al. (2017) developed a GIS-based routing assistance tool to optimize OS/OW routes in Texas based on historical data, the expected heavy traffic level, and pavement condition. To achieve this, they developed pavement performance models based on a sigmoidal and basic power function under the combined characteristics of OS/OW vehicles, origin and destination, permitted routes, the frequency of routes, pavement condition, and climatic effects. The authors found that pavements under cold and dry environments performed better in the short term period than those subjected in hot and humid environments. The other reported finding was that at the early age of the road, higher OS/OW loading would bring a faster deterioration rate.

Other researchers in Texas have preferred the use of the Pavement Condition Score (PCS) as a primary performance index to evaluate pavement damage caused by OS/OW truck traffic (Robinson et al., 1996). Though not intended as a mechanistic indicator of the distress progression in pavements, researchers have preferred its use as a most accurate regression possible with the available traffic and pavement data. This score takes into consideration both the Distress Score (DS) and roughness (in terms of the
International Roughness Index, IRI), defined by Texas Department of Transportation (TxDOT) in the PMIS database (Gharaibeh et al., 2012).

Chatti et al. (2009) conducted a study in Michigan to evaluate the effect of heavy multi-axle trucks with different axle configurations on induced pavement damage. This was done by determining the Axle Factors (AF) from laboratory and mechanistic analysis and then calculating Truck Factors (TF) using the AF and the AASHTO load equivalency factors. The analysis was performed considering the limited types of material properties and datasets. The permanent deformation parameters (α, µ) in VESYS rutting model, were calculated using the previous SPS-1 experiment in the LTPP program. The rutting model was further calibrated based on only one standard axle. Ultimately, the analysis showed that the imparted rutting-based damage on pavement sections was mainly attributed to the passages of heavy trucks with multiple axles; however, these multiple axles were less damaging in fatigue compared to single axles. It was also found that the calculated damage equivalency factors, were significantly higher than those from AASHTO, especially for thinner pavements. This underscored the significance of the axle type and pavement layer properties effects on the pavement damage protocol.

In this study, a new mechanistic approach was developed to accurately calculate the damage equivalency factors tailored towards the site-specific material properties of the pavement layers and the unique features of the transportation systems, directly derived from the field data collection efforts in the overload corridors of South Texas.

2. BACKGROUND

The ALEF, for flexible pavements as defined by AASHTO, is defined as:

\[
ALEF = \frac{W_{18}}{W_{Ax}}
\]  

(1)

where:

\(W_{Ax}\) = Number of \(x\)-axle load repetitions after time \(t\), and
\(W_{18}\) = Number of 18-kip axle load repetitions after time \(t\), calculated from Equation 2,

\[
\log \left( \frac{W_{Ax}}{W_{18}} \right) = 4.79 \log(18 + 1) - 4.79 \log(L_x + L_2) + \log L_2 + \frac{\beta_x}{\beta_18} - \frac{G_t}{G_{18}}
\]  

(2)

where \(G_t\) and \(\beta_x\) are defined as:

\[
G_t = \log \left( \frac{4.2 - p_4}{4.2 - 1.5} \right)
\]  

(3)

\[
\beta_x = 0.4 + \frac{0.081(L_x + L_2)^{3.23}}{(3N + 1)^{5.19}L_2^{3.23}}
\]  

(4)

where:

\(L_x\) = Load in kips on one single axle, one set on tandem axles and one set of tridem axles,
\(L_2\) = Axle code, 1 for single axle, 2 for tandem axle, and 3 for the ridem axle,
\(SN\) = Structural number,
\[ p_t \] = Terminal Serviceability,
\[ G_t \] = Function of terminal serviceability, and
\[ \beta_{18} \] = Value of \( \beta_x \) when \( L_x \) is equal to 18 kips and \( L_2 \) is one.

As is evident in Equation 1, the ALEF for each axle load group is a function of the structural number (SN) which in turn is related to stiffness properties of layers, drainage conditions and the thickness of pavement layers in the network. For generalization purposes, Asphalt Institute (AI) assumed structural number (SN) as 5 and terminal serviceability \( (p_t) \) as 2.5 to develop tables of ALEFs that are widely used by the pavement design industry to characterize the damage imparted by \( i^{th} \)-axle load group relative to the standard axle on the pavements (Huang, 1993).

There are several sources of inaccuracy and systematic errors present in such assumptions that resulted in proposing a different approach to calculate the LEFs for the determination of the relative damages based on the unique features of the energy section zones. The major shortcomings of using the AI load equivalency factors to quantify damage are as follows:

- The ALEF tables are developed for specified SN and terminal serviceability \( (p_t) \) based on equations developed and later modified in the AASHTO road test. Considering the fact that some of the pavements in the energy section zones have been subjected to heavy loads for several years and already show visible signs of distresses and deteriorations, assumption of SN = 5 will result in underestimation the damage in the system.
- The effect of terminal serviceability and structural number on the value of the ALEF in the AASHTO equation is erratic and not consistent with the theory. Super heavy wheel loads are expected to have significantly higher ALEF than unity to indicate more damage compared to the standard 18-kip axle; however, the AASHTO equations predict higher damage with lower SN values.
- SN is a function of the layer thicknesses, drainage conditions and the stiffness properties of the layers. This basically indicates that ALEF is not a single value and should be different based on the features of the pavement systems in the network. Considering the fact that the passage of an overweight truck over an un-surfaced gravel road will potentially induce more damage compared to the passage of same truck over a well-designed and well-maintained interstate highway, using the same ALEF for both cases will compromise the accuracy of damage analysis.

In addition to these limitations, issues such as general methodology for the calculations of ALEF using AASHTO, coupled with the change in the material properties and loading conditions since the last modification of AASHTO equations, are the compelling reasons that induced proposing a new mechanistic approach for the calculations of the modified ALEFs.

3. NEW MECHANISTIC APPROACH FOR PAVEMENT DAMAGE QUANTIFICATION

The proposed approach is based on the mechanistic determination of the axle load equivalency factors directly derived from the analysis of the different measures of failure criteria. This approach considers the site-specific traffic loading conditions, pavement layer configurations, as well as the stiffness properties of layers determined from the field data collection effort. Figure 1 indicates the procedure for
the determination of the modified ALEF values. Initially, traffic information collected by WIM devices, as well as the site-specific pavement structures and layer properties determined after analysis of GPR and FWD field test results, were incorporated into the finite element analysis. Subsequently, the pavement responses including tensile strain at the bottom of Asphalt Concrete (AC), compressive strain at the top of subgrade layer, and surface deflection were obtained to calculate the three measures of axle load equivalency factors based on the three proposed failure criteria (fatigue, rutting, and surface deflection criteria). Ultimately, the highest ALEF value was chosen as the modified ALEF value tailored towards the site-specific conditions of the overload corridors in the network. Detailed information associated with the field data collection, finite element modeling and the related equations for calculating the modified Axle Load Equivalency Factors is provided in the subsequent sections.

![Procedure for the Determination of the Modified Axle Load Equivalency Factor.](image)

### Figure 1. Procedure for the Determination of the Modified Axle Load Equivalency Factor.

#### 3.1 Field Data Collection

Data collection procedure was established to obtain the information associated with the traffic loading conditions and site-specific material properties (thickness and layer modulus) utilizing the GPR and FWD nondestructive tests. This information was collected for ten representative sites in the Eagle Ford Shale region of South Texas, their locations are listed in Table 1.
Table 1. Selected Roadways in Eagle Ford Shale Network

<table>
<thead>
<tr>
<th>District</th>
<th>County</th>
<th>Roadway</th>
</tr>
</thead>
<tbody>
<tr>
<td>Laredo</td>
<td>Dimmit</td>
<td>US 83</td>
</tr>
<tr>
<td></td>
<td>La Salle</td>
<td>FM 468</td>
</tr>
<tr>
<td>San Antonio</td>
<td>McMullen</td>
<td>FM 624</td>
</tr>
<tr>
<td></td>
<td>McMullen</td>
<td>FM 99</td>
</tr>
<tr>
<td></td>
<td>Atascosa</td>
<td>SH 16</td>
</tr>
<tr>
<td>Corpus Christi</td>
<td>Live Oak</td>
<td>US 281</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH 72</td>
</tr>
<tr>
<td></td>
<td>Karnes</td>
<td>SH 123</td>
</tr>
<tr>
<td>Yoakum</td>
<td>Gonzales</td>
<td>US 183</td>
</tr>
<tr>
<td></td>
<td>Dewitt</td>
<td>SH 119</td>
</tr>
</tbody>
</table>

3.1.1 Traffic Loading Information

Proper characterization of the traffic operations in the affected overload corridors is of paramount importance for accurate quantification of the damage. The most favorable approach, as adopted in the new Mechanistic-Empirical Pavement Design Guide (MEPDG), is the concept of the Axle Load Spectra. In this approach, the performance of the pavement is tied to the traffic distributions of different class vehicles, unlike the traditional Equivalent Single Axle Load (ESAL) concept. To develop the Axle Load Spectra database, the authors collected the necessary traffic information by deploying portable Weigh-In-Motion (WIM) devices across ten representative sites in heavily trafficked highways used by the oil and gas industry. The Axle Load Spectra database contained extensive information regarding:

- Truck traffic distributions
- Gross vehicle weights (GVW)
- Axle weights – Steering, Single, Tandem, Tridem, Quad
- Frequency of vehicle class in the highway network

The corresponding traffic information such as axle weight, axle type, GVW and vehicle classification, derived from the Axle Load Spectra, were incorporated into the FE simulation. This information coupled with the axle configurations, obtained from the data recorded by the portable WIM devices, were instrumental for the determination of the new modified load equivalency factors for South Texas overload corridors. Figure 2 illustrates the single axle load distributions for US 281, which is an extremely trafficked highway in the Corpus Christi District.
In this study, using the finite element analysis, OW axle loads were simulated since these heavy loads are extremely destructive to the pavement structure. Table 2 presents the overweight axle ranges considered in the numerical modeling. It also indicates the Texas permissible weight limits for different axle types. Axle weights heavier than these values are essentially considered overweight (OW) axles. Additionally, it illustrates the maximum axle weight recorded from the field data for different axle types and the load intervals defined in the finite element simulation procedure of the OW axles.

Table 2. Overweight Axle Range in Finite Element Modeling

<table>
<thead>
<tr>
<th>Axle Type</th>
<th>Texas Maximum Permissible Weight (lb)</th>
<th>Maximum Axle Weight Observed in the Field (lb)</th>
<th>Load Intervals (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single Axle</td>
<td>20,000</td>
<td>42,000</td>
<td>1,000</td>
</tr>
<tr>
<td>Tandem Axle Group</td>
<td>34,000</td>
<td>64,000</td>
<td>2,000</td>
</tr>
<tr>
<td>Tridem Axle Group</td>
<td>42,000</td>
<td>114,000</td>
<td>4,000</td>
</tr>
<tr>
<td>Quad Axle Group</td>
<td>50,000</td>
<td>140,000</td>
<td>6,000</td>
</tr>
</tbody>
</table>

3.1.2 Calculation of Tire Pressure and Tire Footprint

Due to the fact that the tire pressure has a significant impact on the pavement responses, the authors were motivated to conduct some experimental investigations in the field to calculate the actual tire pressure, instead of assuming a typical value. For this purpose, using Class 6 and Class 9 vehicles provided by the TxDOT, truck axles were weighed by static scales while their respective tire footprints were measured from prints of painted tires, as shown in Figure 3. Based on the observed measurements, a tire pressure of 120 psi was selected as representative for simulation purposes.
3.1.3 Nondestructive Testing
The authors also conducted the GPR and FWD test, as shown in Figure 4, to obtain site-specific material properties of the representative highways. Accurate layer thicknesses, layer configurations, and back-calculated layer moduli were incorporated in the FE models for selected pavement sections in the overload corridors. Pavement layers properties associated with all ten representative sites are provided in Ashtiani et al. (2019).

3.2 Finite Element Modeling
Different pavement layers consisting of surface layer, base and subgrade were simulated in the ABAQUS program as is presented in Figure 5. Based on the nondestructive test results, specific structural properties of the pavement layers such as the layer modulus and the layer configuration were assigned for each representative roadway in the finite element model. In terms of the meshing, a refined mesh was used to better address the critical pavement responses, at points directly located under the wheel path. However, to expedite the computation time and reduce the output files size, a coarser mesh in the regions far from the loading areas was used. Therefore, as shown in Figure 5, three-dimensional continuum elements C3D8 (eight-node linear brick) with finer meshing size and C3D6 (six-node linear triangular prism) with coarser meshing size, were assigned to the loading area elements and the other elements, respectively. In addition, a transition area was defined between fine and coarse mesh areas for gradual change of element size to ensure improved accuracy. Figure 6 also illustrates the locations.
where critical responses are obtained from the finite element modeling: Surface deflection at Point A, tensile strain at bottom of the AC layer (Point B) and compressive strain at top of the subgrade (Point C).

Figure 5. Model Geometry and Meshing.

Figure 6. Locations of the Pavement Response Parameters.

3.3 Modified Axle Load Equivalency Factors

The following three different classifications of axle load equivalency factors are used for accurate characterization of the damages imparted by energy-related operations in the network based on the finite element analysis:

- **ALEF based on fatigue failure criterion**: In the AI approach, the tensile strain at the bottom of the asphalt layer was selected as the critical response that controls the fatigue performance of the flexible pavements as shown in Equation 5:

  \[ N_f = f_1 (\varepsilon_T)^{f_2} (E_{AC})^{f_3} \]  

  (5)

  Where the \( N_f \) is the allowable number of load applications to fatigue failure, \( \varepsilon_T \) is the tensile strain at the bottom of the asphalt layer, \( E_{AC} \) is the modulus of the asphalt layer and \( f \)-values are the model parameters.

  The equivalent axle load factor for axle load group \( x \) compared to standard 18-kip axle based on the fatigue criteria can be calculated from Equation 6 as:
ALEF based on rutting criterion: The Al rutting failure model assumes that the asphalt layer and the base layer will not experience any permanent deformation; therefore, all rutting is associated to subgrade permanent deformation (Bahia, 2000). Hence, the compressive strain $\varepsilon_c$ at the top of the subgrade is assumed to be the controlling factor for the determination of the rutting performance of the flexible pavements. Equation 7 defines the rutting performance as:

$$N_f = f_4 (\varepsilon_c)^{f_5}$$  \hspace{1cm} (7)

Where $N_f$ is the allowable number of load applications to rutting failure. The axle load factor based on the rutting criterion can be calculated from Equation 8 as:

$$ALEF = \left( \frac{W_{110}}{W_{tx}} \right) = \left( \frac{\varepsilon_{tx}}{\varepsilon_{110}} \right)^{f_5}$$  \hspace{1cm} (8)

Coefficients $f_1$ through $f_5$ were input to the distress model equations similar to Al values, as shown in Table 3.

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_1$</td>
<td>$7.96 \times 10^{-2}$</td>
</tr>
<tr>
<td>$f_2$</td>
<td>3.29</td>
</tr>
<tr>
<td>$f_3$</td>
<td>0.85</td>
</tr>
<tr>
<td>$f_4$</td>
<td>$1.37 \times 10^{-9}$</td>
</tr>
<tr>
<td>$f_5$</td>
<td>4.47</td>
</tr>
</tbody>
</table>

ALEF based on the surface deflection: The Al rutting failure criterion assumed that the asphalt and base layer remains intact during the service life of the pavement, and all the deformation is due to subgrade rutting. This is often not true, as there will be rutting in the asphalt and base layers during the service life of flexible pavements. Therefore, it deems necessary to consider an additional measure to calculate the axle load equivalency for pavement sections. To achieve this objective, the Finite Element method was used to calculate the surface deflection that represents the combined deformation of all layers. This information was used to develop a new measure of an axle load equivalency factor based on surface deflection criteria for different axle loads as shown in Equations 9 through 11:

$$N = \left( \frac{1}{D} \right)^{3.8}$$  \hspace{1cm} (9)

Single Axles: $ALEF = \left( \frac{W_{110}}{W_{tx}} \right) = \left( \frac{D}{D_b} \right)^{3.8}$  \hspace{1cm} (10)

Multiple Axles: $ALEF = \left( \frac{W_{110}}{W_{tx}} \right) = \left( \frac{D}{D_b} \right)^{3.8} + \sum \left( \frac{\Delta_i}{D_b} \right)^{3.8}$  \hspace{1cm} (11)

where, $D$ is the surface deflection, $\frac{D}{D_b}$ is the ratio of pavement surface deflections caused by a single axle load to those recorded under the standard 18,000-lb single axle load ($D_b$). Furthermore, $\Delta_i$ is the difference in magnitude between the maximum deflection recorded
under each succeeding axle and the minimum residual deflection preceding the axle (Kawa et al., 1998).

4. FE NUMERICAL SIMULATION RESULTS AND ANALYSIS

Finite element analysis was conducted to properly quantify the damage imparted on the overload corridors by evaluating the influence of site-specific roadway properties on the ALEF values. The modified ALEFs values of the three roadway types in the network were contrasted with each other for comparison purposes in this research effort as shown in Figure 7. The results showed that FM roadways with the weakest layer configurations had the highest damage factors among the three roadway types. Conversely, SH and especially US roadways with more robust layer configurations (medium thickness or thick asphalt layer as well as a stiffer base layer) had the lowest axle load equivalency factors. This underscores the fact that overlooking the influence of pavement layer configurations can potentially incur systematic errors for the pavement damage analysis of the overload corridors.

In order to clarify the significance of the site-specific characterizations, modified ALEF values were also compared with the traditional Asphalt Institute ALEFs as depicted in Figure 7. It was observed that traditional Asphalt Institute approach presents lower ALEF values than the proposed mechanistic approach. As indicated in Table 4, modified ALEF values on average could be up to 78%, 41%, and 38 % higher than traditional Asphalt Institute ALEF values in FM, SH and US roadways, respectively. Hence, it should be acknowledged that the traditional AI method underestimates the damage imparted on the different pavement types since it is not capable to evaluate the effect of site-specific properties on the ALEF values.
Figure 7. ALEF Values for Different Roadways in the Network and Different Axle Types a) Single Axle, b) Dual Axle, c) Tridem Axle, and d) Quad Axle.

Table 4. Percentages of Underestimation in Traditional Asphalt Institute ALEFs

<table>
<thead>
<tr>
<th>Roadway Type</th>
<th>Axle Type</th>
<th>Single</th>
<th>Tandem</th>
<th>Tridem</th>
<th>Quad</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM</td>
<td></td>
<td>57</td>
<td>55</td>
<td>78</td>
<td>24</td>
</tr>
<tr>
<td>SH</td>
<td></td>
<td>40</td>
<td>31</td>
<td>41</td>
<td>18</td>
</tr>
<tr>
<td>US</td>
<td></td>
<td>34</td>
<td>28</td>
<td>38</td>
<td>15</td>
</tr>
</tbody>
</table>

5. SUMMARY AND CONCLUSIONS

The proposed mechanistic approach provides a useful and methodologically sound approach to properly quantify pavement damages associated with overweight vehicles in energy developing zones of South Texas. The major findings in terms of the damage factors in the selected roadways in the network are summarized as follows:

- The proposed mechanistic approach confirms that the modified ALEF values are significantly higher than the traditional Asphalt Institute axle load equivalency factors, commonly used by
the pavement design engineers. In other words, conventional AI method underestimates the damage, since it is not capable to assess the ALEF values tailored towards the site-specific characteristics.

- The detrimental effect of traffic loads is higher for pavements with lower structural capacity. This is as expected, because heavy axle loads are more destructive to less robust pavement profiles such as FM roads than in US roadways. In the studied sections, FM roadways have the highest ALEFs, followed by SH and ultimately US roadways.

- Considering the fact that the pavement profile, mixture design, and environmental factors greatly influence the mechanistic damage equivalency factors in overload corridors, it is imperative to cluster and differentiate between different types of roadways, such as FM, SH and US roadways, to better represent the damages imparted on roadways due to overweight truck operations.

- The numerical simulations of 10 representative pavement sections in this study confirmed that the Tridem and Quad axles have lower damage factors per tonnage compared to single and tandem axles. This is primarily attributed to the distribution of axle loads over multiple wheel arrangements.

6. ACKNOWLEDGMENTS

The authors thank TxDOT for their financial support and the Maintenance Division of San Antonio, Corpus Christi, Yoakum and Laredo Districts for their collaboration with the research team to collect data.

7. REFERENCES


Pavement Management Information System-Final Report,” Report FHWA/TX-12/0-6386-3, Texas A&M Transportation Institute, College Station, TX.


Geo-Structural Characterization of Foundation Materials Using Automated Plate Load Testing

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Ingios Geotechnics, Inc.

ABSTRACT

Structural design of highway or airfield pavements or railroads involves assumption of key mechanistic parameter values such as resilient modulus, modulus of subgrade reaction, dynamic modulus, and track modulus. There is growing interest in mechanistic-based construction testing methods to field verify design-modulus assumptions. Many testing methods used today used for this purpose (e.g., light/falling weight deflectometers, low-strain moduli measuring devices, dynamic cone penetrometer), are indirect measurements with empirical relationships to design assumed values. Although they are rapid tests, key factors such as stress-dependency, inconsistencies in measurement influence depths between field testing and what is assumed in design, need for proper seating loads, and loading rate are often ignored with the indirect measurements. Further, empirical relationships are often associated with significant uncertainty with future predictions. Automated Plate Load Testing (APLT) is an advanced testing method that has been developed to specifically overcome those issues and provide a direct measure of the design-modulus values used in design of highways, airfields, and railroads. APLT has been used successfully on over 80 projects across the United States, Canada, and Latin America on research and implementation projects. In this paper, the different mechanistic design input parameter values are described and the corresponding in situ APLT testing methods along with example test results are provided. The input parameter values include stress-dependent resilient modulus for foundation geomaterials, stress- and frequency-dependent asphalt dynamic modulus, modulus of subgrade reaction, strain modulus, and permanent deformation under cyclic loading.

1. INTRODUCTION

One of the key input parameters in designing highway or airfield pavements and railroads is the design modulus value. Because geo-materials exhibit non-linear behavior, there are many definitions for moduli values – elastic modulus, reload modulus, resilient modulus, secant modulus, modulus of subgrade reaction, etc. – and there are many factors that influence the moduli values (see Briaud 2001). Design equations, particularly for pavement thickness design, are typically calibrated for moduli values measured using a certain test and interpretation procedure. Depending on the design procedure chosen, it is important for designers to understand these factors to select the appropriate testing procedure for field verification of the design modulus value.

Plate load testing (PLT) is considered the long-standing “gold standard” test measurement for assessing in situ pavement foundation support conditions and railway subgrades. From the 1930s to the 1980s, the Bureau of Public Roads, U.S. Corps of Engineers, AASHTO, and several state agencies used PLT to determine the modulus of subgrade reaction k-value for airfield and highway applications, investigate concrete pavement behavior, and verify/calibrate design equations (see Teller and Sutherland 1943, U.S. Corps of Engineers 1943, AASHTO 1962). In the 1940s Bureau of Public Roads reported extensive field testing for the Arlington Experiment Farm in Virginia, which involved repeated load-unload plate load tests (Teller and Sutherland 1943). Railway subgrade support is also characterized using the k-value (see
Selig and Lutenegger, 1991) and plays an important role in trackbed design and analysis for railways (McHenry and Rose 2012).

The pioneering efforts from the 1930s to the 1980s established PLT to determine the load-displacement relationship of foundation layers and a significant role in calibrating the pavement thickness design equations developed by the AASHTO, PCA, and Corps of Engineers. However, the manual PLT methods were time consuming because of significant setup times with heavy reaction loads often creating unsafe conditions. Also, without automation, producing reproducible results from manual plate load testing can be difficult because of operator bias, lack of control with maintaining and applying loads, etc., even for a static test. It is almost impractical to apply repeated loads at a controlled load pulse using manual methods.

Because of those limitations, the frequency at which these tests were conducted has diminished substantially. As a simplification, several agencies attempted to develop local empirical relationships between plate load test measurements from California bearing ratio, R-value, falling weight deflectometer (FWD) testing, and others. These empirical relationships, however, present significant uncertainties and poorly match the field conditions.

Realizing the very important role plate load testing plays in determining the design-moduli values and the limitations involved with the manual setups and the uncertainties associated with using empirical relationships, the modern automated plate load testing (APLT) system was developed. APLT is a state-of-the-art test device used to characterize a variety of in situ mechanistic performance parameter values for pavement and pavement foundation layers (White and Vennapusa 2017, White et al. 2019). APLT has been used successfully on over 80 projects across the United States, Canada, and Latin America on research and implementation projects.

In this paper, a summary of the various design modulus input parameters used in pavement and railway design are summarized along with the current state-of-the-practice testing methods and their limitations. Then, the APLT procedures to determine the different design moduli values are presented along with example results from field projects.

2. DESIGN INPUT PARAMETERS, CURRENT TESTING METHODS, AND LIMITATIONS

In this section, the key input parameters used in highway, airfield, and railroad design are described as background information along with the current state-of-the practice for measuring these properties.

2.1 Foundation Layer Resilient Modulus, $M_r$

For flexible pavement design, the main pavement foundation layer input parameters in the AASHTO (1972, 1986, and 1993) design guides for flexible pavement design are resilient modulus ($M_r$) of subgrade, structural layer coefficients ($a_i$) for all layers above the subgrade including the base, subbase, and surface layers, and drainage coefficient ($m_i$) for the base and subbase layers. Empirical relationships are presented therein for $a_i$ and moduli values for base and subbase layers, in addition to other properties such as California bearing ratio (CBR) and R-value.

In the mechanistic-empirical (ME) pavement design procedure (AASHTO 2015), the main pavement foundation layer input parameter for flexible pavement design is $M_r$. There are three design levels in AASHTO (2015). Level 1 includes selection of stress-dependent constitutive model parameters ($k_1$, $k_2$, $k_3$).
and $k_3$) or determining $M_r$ at the anticipated field stresses, and the model parameters are typically determined from laboratory $M_r$ testing (i.e., AASHTO T307). Level 2 includes empirical correlations to estimate $M_r$ from surrogate measurements such as CBR, dynamic cone penetrometer (DCP) measurements, and R-value measurements. Level 3 includes using default or recommended $M_r$ values based on the soil AASHTO classification.

For rigid pavement design, the key design input parameter for foundation layers in many of the rigid pavement design procedures (AASHTO 1972, 1986, 1993, 2008; PCA, 1984; FAA 2016) is the modulus of subgrade reaction ($k$-value), which is discussed in the following sub-section of this paper. One exception is the Level 1 design in the ME design procedure (AASHTO 2015), where $M_r$ constitutive model parameters are needed. For Levels 2 and 3 ME design, the selected $M_r$ values are converted to a $k$-value in the design software (AASHTO 2015).

The $M_r$ parameter is a highly stress-dependent parameter, and most soils exhibit the effects of increasing stiffness with increasing bulk stress and decreasing stiffness with increasing shear stress. The results from a test that involves applying a series of cyclic deviator and confining stresses can be used to model the behavior using the “universal” model (AASHTO 2015) shown in Eq. (1):

$$M_r = k_1 P_a \left( \frac{P_a}{P_a} \right)^{k_2} \left( \frac{\tau_{oct}}{P_a} + 1 \right)^{k_3}$$  (1)

where, $M_r$ = resilient modulus (psi); $P_a$ = atmospheric pressure (psi); $\theta$ = bulk stress (psi); $\tau_{oct}$ = octahedral shear stress (psi); and $k_1$, $k_2$, and $k_3$ = regression coefficients.

In ME design guide (AASHTO 2015), it is recommended that the $M_r$ properties be determined using laboratory testing (AASHTO T-307) for new projects and using falling weight deflectometer (FWD) on rehabilitation projects. Laboratory testing provides a controlled set of measurements at different stress combinations to develop regression parameters used in the “universal” model (Eq. 1). However, due to the complexity of the laboratory triaxial test and often non-representative boundary conditions, $M_r$ of pavement foundation materials is often obtained from empirical correlations between $M_r$ and other properties such as soil classification, California Bearing Ratio (CBR) or Hveem R-value (i.e., Levels 2 and 3 in ME Design). As an example, the different empirical relationships available in the literature between CBR (determined from different test procedures including the dynamic cone penetrometer (DCP) penetration resistance (PR) values) and elastic modulus values are shown in Figure 1, which highlights the significant uncertainties associated with the estimated moduli values.

In situ $M_r$ is also often estimated from non-destructive surrogate tests including the FWD or light weight deflectometer (LWD). In practice, the elastic moduli values calculated from these test devices based on peak deformations are often confused with $M_r$ values which are based on resilient (i.e., recoverable) deformations. Limitations of these non-destructive surrogate tests is the lack of a conditioning stage prior to testing and limited ability to maintain a defined contact stress during unloading. During pavement construction, pavement foundation materials are subjected to relatively high loads from construction traffic and compaction equipment. In response to these loads, aggregate particles rearrange themselves resulting in higher density and stiffness. For this reason, it is important to apply conditioning load cycles prior to testing to determine in situ $M_r$, which is not possible with FWD testing. Once surface paving is complete, the pavement foundation below is confined by the overlying pavement
layers. The response of a pavement foundation to subsequent repeated traffic loading is both nonlinear and stress-dependent and therefore the effect of confinement is an important condition to consider in a field based M, test.

Figure 1. Empirical relationships published in the literature between CBR and elastic or resilient modulus.

2.2 Foundation Layer Modulus of Subgrade Reaction, \(k\)-value

The subgrade \(k\)-value and the composite \(k\)-value which accounts for inclusion of base layer placed over the subgrade, are the key foundation layer input parameters in most of the rigid pavement design procedures and railway subgrades (AASHTO 1972, 1986, 1993, 2008; PCA 1984; FAA 2016; Selig and Luteneberger 1991; McHenry and Rose 2012). Selection of the \(k\)-values have significant implications on design, cost and future performance of the pavement section. Depending on the design method chosen, it is important to understand the corresponding \(k\)-values for which the design equations were originally calibrated. Darter et al. (1995) explained this history in detail. In brief, the Corps of Engineers defined \(k\)-values as the ratio of applied load corresponding to 0.05 in. of plate deformation, wherein the loads are applied incrementally like the procedure described in ASTM D1196 or AASHTO T-222 using a 30-in. diameter loading plate (Middlebrooks and Bertram 1942). This formed the basis of the PCA (1984) and the Corps of Engineers rigid pavement design procedures (U.S. Corps of Engineers, 2001).

On the other hand, the AASHTO road tests (Highway Research Board 1962) conducted in the late 1950s followed a test procedure that involved performing three loading/unloading cycles each at three stress levels (5, 10, and 15 psi) using a 30-in. diameter loading plate (9 loading cycles total). The \(k\)-values were then determined using two procedures. The first procedure involved determining the elastic \(k\)-value (\(k_E\))
based on the rebound deformations for each loading cycle (excluding the permanent deformation) and then averaging the data for the nine cycles. The second procedure involved determining the gross $k$-value ($k_G$) based on the total deformation produced for each load level (at the end of the three loading cycles), and then averaging the data for the three load levels. The AASHTO (1972) design guide states that only one value was used to represent AASHTO Road Test sections in developing the rigid pavement design equation and is based on the $k_G$ value. The later versions of the AASHTO design guide (AASHTO 1986, 1993) does not indicate whether to use $k_E$ or $k_G$.

From the different procedures described above, the $k_E$ value results in a higher value compared to both the $k_G$ value and the $k$-value determined using the 0.05 in. deflection criteria per Corps of Engineers. The magnitude of difference between these, however, will depend on the stiffness of the material, degree of saturation, level of compaction, and stresses applied.

The $k$-value is typically determined using AASHTO T-222 or ASTM D1196 non-repetitive static PLT or AASHTO T-221 or ASTM D115 repetitive static PLT. As stated earlier, traditional manual methods of conducting PLT is time-consuming. Therefore, as a simplification, several agencies attempted to develop local empirical relationships between plate load test measurements from CBR, R-value, FWD testing, and others. These relationships also show significant variability depending on the relationship chosen (see Zhang et al. 2019).

### 2.3 Strain Modulus, $E_v$

Strain modulus (or also referred to as the deformation modulus) is a quality assurance parameter used in Europe to field evaluate the pavement foundation layers (ISSMGE 2005, ATB Vag 2005). The DIN 18134 (2001) standard for plate load test describes the procedure to calculate strain moduli ($E_v$) using different plate sizes ranging from 12 to 30 in. diameter. The test involves applying two loading cycles, where the load is applied in equal increments until either a target deformation criterion (0.20 in.) or a maximum stress of 72.5 psi, whichever occurs first, is achieved. The strain modulus ($E_v$) is calculated for the two loading cycles (referred to as $E_{v1}$ and $E_{v2}$), using the load-deformation curve and assuming a Poisson’s ratio and shape factor for the anticipated stress distribution beneath the plate. High-speed rail design and field quality assurance specifications also use the $E_v$ value (see Zicha 1989; Rulens et al. 2009).

The manual setup for this test suffers the same limitations as the traditional static PLT methods described above for $k$-value tests, with time-consuming setup and need for heavy reaction loads creating unsafe working conditions.

### 2.4 Asphalt Concrete Layer Dynamic Modulus

Dynamic modulus of the AC layer is one of the key asphalt layer property that has a substantial impact on the designed pavement layer thickness using regression based methods such as AASHTO (1972, 1986, and 1993) or the ME based method (AASHTO 2008). For linear visco-elastic materials such as hot mix asphalt (HMA) mixtures, the stress-strain relationship under a continuous sinusoidal loading in the frequency domain is defined by its complex dynamic modulus ($E^*$) (Dougan et al. 2003). In laboratory testing, the dynamic modulus is defined as the ratio of the amplitude of the sinusoidal stress at a given time ($t$) and the amplitude of the sinusoidal strain at the same time and frequency that results in a steady state response. Laboratory testing to determine dynamic modulus (AASHTO TP62) is repeated at different temperatures and frequencies, and the results are then used to develop a complex modulus master curve, which is one of the design inputs in the ME Design (AASHTO 2008). Master curves are
constructed using the principle of time-temperature superposition, where data at various temperatures are shifted with respect to log of time until the curves merge into a single smooth function. The amount of shift required at each temperature required to form the master curve describes the temperature dependency of the material (Dougan et al. 2003).

In AASHTO (1972, 1986, and 1993) design procedures, the design expression for flexible pavement design to calculate the design equivalent single axle loads (ESAL) is developed based on a tire inflation pressure of 70 psi and a travel speed of 55 mph with a bias-ply design (AASHTO 1962). It must be noted that the tire pressures today are much higher than this (> 100 psi) and is not accounted for in that ESAL calculations (Timm et al. 2014). The asphalt layer property is considered using a structural layer coefficient \(a_1\) to represent the relative contribution of each pavement layer in the design. The following empirical expression is used to estimate \(a_1\) using the asphalt layer modulus \(E_{AC}\) in units of psi:

\[
a_1 = 0.171 \ln(E_{AC}) - 1.784
\]  

(2)

In the AASHTO (1993) design guide, Eq. 2 is intended to use for use with a maximum EAC of 450ksi, which corresponds to \(a_1 = 0.44\). According to Timm et al. (2014), 45% of the states in the U.S. use 0.44 for at least one paving layer, 28% of the states use less than 0.44, two states (Alabama and Washington) recently revised the coefficients to 0.54 and 0.50. The higher coefficients used by Alabama and Washington reflect modern advances in the materials and the construction procedures (Timm et al. 2014).

In the ME design guide (AASHTO 2008), it is recommended that the dynamic modulus is determined using laboratory testing for new projects and using falling weight deflectometer (FWD) on rehabilitation projects. Laboratory testing provides a controlled set of data at different frequencies and temperatures to develop the master curves required in design. The FWD field testing is a convenient and rapid test which involves dropping a series of dynamic loads (typically 3 to 4), obtaining a deflection basin, and back calculating the individual layer properties. However, it only provides a single value and is typically obtained at one stress, and the loading applied is not sinusoidal as performed in the laboratory. Further, FWD testing is only performed using a few loading cycles which is not the same as the laboratory testing where the sample is conditioned prior to testing using hundreds of loading cycles. Because the loading is not sinusoidal, a frequency-dependent dynamic modulus cannot be estimated and directly used in the ME design. For these reasons, FWD testing results present limitations when using the data as part of the design process.

3. DIRECT MEASUREMENT OF INPUT PARAMETERS USING AUTOMATED PLATE LOAD TESTING (APLT)

The APLT technology uses modern control and data collection systems combined with advanced stress control capability to simultaneously measure stress-dependent elastic and permanent deformation, stress-dependent elastic and resilient modulus, and load-pulse and frequency-dependent responses. The APLT is used to perform static PLTs (e.g., AASHTO T222) which takes about 30 minutes to 4 hours depending on subgrade stiffness and cyclic/repetitive plate load tests (e.g. ASTM 1195) with up to 100 cycles (5 minutes), 1000 cycles (20 minutes), and 10,000+ cycles test (2+ hours) per test location. The cyclic test process uses a controlled load pulse duration and dwell time (e.g., as required in the laboratory AASHTO T307 M, test methods) for selected cycle times depending on the field conditions and measurement requirements. The advantage of cyclic tests is that the modulus measurements better
represent the true field modulus value. This finding is well documented in the literature and is considered a major short-coming of other testing methods that only apply a few cycles/dynamic load pulses on the foundation materials.

The APLT system has the capability to measure inputs to develop in-situ confining and deviator stress-dependent constitutive models used in the AASHTOWare® Pavement ME Design (AASHTO 2015). The result of this test is a direct field measure of the mechanistic response of the pavement foundation. This is the only such in-situ test to directly measure the stress-deflection response with confinement control. Confinement control can be applied to precisely duplicate the pavement-induced stress conditions. Because the APLT test system is automated, the test methods are highly repeatable and reproducible (i.e., no operator bias). Operators only need to input the desired loading conditions (cyclic stress levels, load pulse duration and dwell time, and number of cycles) which are then tightly controlled by the machine. An advanced fluid-power control system was designed to perform the test operations and meets or exceeds the applicable testing standards.

Figure 2 shows the operator station, controls, monitor, and plate load test setup with sensors and reference beam for down core hole and on surface (foundation layer) measurements with different plate size configurations. The results of cyclic deformation, permanent deformation, elastic modulus, stiffness, resilient modulus, cyclic stresses, and number of cycles are calculated in real-time and are available for reporting immediately.

Figure 2. (a) APLT equipment setup for testing over a core hole location on pavement, (b) downhole plate and reference beam setup, (c) plate setup with deformation measurements of plate and at 2r, 3r, and 4r from plate center axis, and (d) 30 in. diameter plate load test setup.
3.1 Modulus of Subgrade Reaction (k-value)

APLT can be used to perform automated static PLTs in accordance with the applicable AASTHO, ASTM, Army Corps of Engineers, and European test standards. APLT is configured with 6 in., 12 in., 18 in., 24 in., and 30 in. diameter loading plates. A 30-in. diameter plate setup is shown in Figure 1d. The test is automated to meet the applicable test methods and therefore can produce highly repeatable and reproducible test results with no operator bias in testing or interpretation of results.

An example of test results with a 30-in. diameter loading plate with two loading cycles is shown in Figure 3, with calculations shown to calculate k values (uncorrected and corrected for plate bending), per AASHTO T222. The graph shows stress versus deformation values for the two loading cycles along with plate rotation measurements. The stress versus deformation readings from each loading cycle are fit with a second order polynomial relationship, which shows a coefficient of determination (R2) of close to 1, demonstrating the quality of the data produced from the automated static PLT.

Figure 3. Example results of 30 in. diameter static plate load test with two loading cycles, per AASHTO T222.

3.2 Composite Resilient Modulus (M\textsubscript{r-comp})

Composite uncorrected resilient moduli values from APLT can be calculated using the modified Boussinesq’s elastic half space solution equation shown in Eq. (3):

\[
M_{r-comp} = \frac{(1-v^2)\delta_{r,0}}{\delta_{r,0} \times f}
\]  

where, \(M_{r-comp}\) is in situ composite resilient modulus, \(\delta\) is the resilient deflection of plate during the unloading portion of the cycle (determined as the average of three measurements along the plate edge, i.e., at a radial distance \(r' = r\)), \(v\) is the Poisson ratio (often assumed as 0.40), \(\sigma_0\) is the cyclic stress, \(r\) is...
the radius of the plate, \( f \) is the shape factor selected based on the anticipated stress distribution beneath the plate (\( \pi/2 \) to \( 8/3 \)).

### 3.3 Layered Analysis for Individual Layer Resilient Modulus

A layered analysis sensor kit (Figure 1c) setup with APLT measures the resilient deflections at radii of 12 in. (2\( r \)), 18 in. (3\( r \)), and 24 in. (4\( r \)) away from the plate center. The layered analysis measurement sensor kit provides average resilient deflections measured over one-third of the circumference of a circle at the selected radii. This method was designed to improve upon practices that use point measurements, which are often variable from point-to-point for unbound aggregate materials. Like the loading plate representing an integrated response of the material under the plate, the deflection basin circumference bars were designed to represent an integrated deflection basis response over a length of one-third the circumference.

Using the deflection basin measurements, two and three-layered analysis can be performed to develop stress-dependent \( M_r \) values. The two-layered analysis is performed using the Odemark method of equivalent layer thickness approach (AASHTO 1993), while the three-layered analysis is performed using a proprietary back calculation analysis recently developed by Ingios (APLT-BACK). The program was developed through a numerical algorithm to solve an extended formulation of the linear-elastic analysis theory and details are provided in White et al. (2019b). The most significant advantage of the APLT-BACK program over the many back calculation programs that are currently available is that the program allows modeling the analysis for different stress distributions beneath the loading plate (i.e., uniform, parabolic, and inverse parabolic). The different stress distributions can be easily accounted for in the \( M_r \) \( \text{Comp} \) calculations using the appropriate stress distribution factor \( (f) \) in Eq. 3, but most of the current back calculation programs typically are only designed to solve a uniform stress distribution problem. The uniform stress distribution is true only for a flexible plate on cohesionless soil, but the assumption is not accurate because of the rigidity of the plate and the soil type can be either cohesive or cohesionless.

### 3.4 Determination of "Universal" Model Regression Parameters

The “universal” model regression parameters required for the ME design can be obtained in situ using APLT at different cyclic stresses like the AASHTO T-307 lab testing. The applied cyclic and contact stresses and the number of loading cycles are customized per project needs. Cyclic stresses can be varied between 2 psi and 150 psi using a 12-in. diameter loading plate. The data can then be analyzed to fit the model shown in Eq. (4), which is like the laboratory test based model shown in Eq. (1), except that the regression parameters are identified with an asterisk (*) to differentiate with the regression parameters obtained from laboratory testing:

\[
\ln(\text{in-situ } M_r) = k_1 P_a \left( \frac{\sigma}{P_a} \right)^{k_2} \left( \frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \tag{4}
\]

where, \( \text{in-situ } M_r \) = resilient modulus determined in-situ (psi); \( P_a \) = atmospheric pressure (psi); \( \theta \) = bulk stress (psi) = \( \sigma_2 + \sigma_3 = \) applied cyclic stress (\( \sigma_{\text{cyclic}} \)) plus confining stress due to pavement layer based on the thickness of the pavement above the unbound foundation layer (confining stress = 1 to 2 psi if performed down in a core hole and 0 if performed at the surface with no pavement confinement); \( \sigma_2 = K_0 \sigma_1; \sigma_3 = \sigma_2; K_0 = \) coefficient of lateral earth pressure at rest = \( v/(1-v) \); \( v \) = Poisson’s ratio; \( \tau_{oct} = \)
octahedral shear stress (psi) = \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} / 3; \text{ and; and } k_1^*, k_2^*, \text{ and } k_3^* = \text{ regression coefficients.}

An example of the universal model fit curves obtained from the IL Tri State Tollway project near O’Hare airport are shown in Figure 4 and the model parameters are summarized in Table 1. Results showed that the in situ M_{r\text{-comp}} values are sensitive to the applied cyclic stress and showed a “break-point stress (\sigma_{cyclic-BP})” at which point further increase in stress showed a decrease in M_{r\text{-comp}} values. Identification of this break-point stress is critical to pavement designers to model future pavement designs to limit permanent deformation and premature distress problems. Using the deflection basin measurements and layered analysis calculations performed on APLT measurements obtained at different cyclic stresses, “universal” model parameters can be obtained for both the top and bottom layers in a two-layered structure. Example test results of such a case with testing on a crushed aggregate base over natural subgrade is shown in Figure 5 along with the “universal” model parameters reported separately for each layer. Results showed a generally increasing trend with cyclic stress for the top base layer (granular material) and a generally decreasing trend with cyclic stress for the bottom subgrade layer (cohesive material).

![Figure 6. Cyclic stress versus in situ composite M_r and universal model fit curves.](image)

Table 1. Summary of universal model regression parameters from IL Tri-State Tollway project.

<table>
<thead>
<tr>
<th>Test point</th>
<th>k_1^*</th>
<th>k_2^*</th>
<th>k_3^*</th>
<th>M_{r\text{-comp-BP}} (psi)</th>
<th>\sigma_{cyclic-BP} (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>APLT_033, SB, Lane 1</td>
<td>926</td>
<td>0.444</td>
<td>-3.11</td>
<td>12,603</td>
<td>13.6</td>
</tr>
<tr>
<td>APLT_037, SB, Lane 2</td>
<td>638</td>
<td>0.561</td>
<td>-3.05</td>
<td>9,936</td>
<td>19.1</td>
</tr>
<tr>
<td>APLT_032, SB, Lane 4</td>
<td>1072</td>
<td>0.523</td>
<td>-1.62</td>
<td>23,239</td>
<td>39.0</td>
</tr>
</tbody>
</table>
4. CONCLUDING REMARKS

Although convenient, with often much less upfront costs compared to direct measurement options, using the indirect methods and empiricism, introduce substantial risks because the estimated values may not match the actual field conditions. Adoption of more rigorous in situ field measurements has been limited by the ability to perform reliable and repeatable measurements. With development of modern APLT as described in this paper, it is not possible to obtain reliable and repeatable test measurements to field verify design moduli values such as stress-dependent resilient modulus (M_r) and modulus of subgrade reaction (k-value) for foundation layer materials, and stress and frequency dependent dynamic moduli values for asphalt pavement layer materials. Direct measurements reduce risk to both the owner and the contractor allowing for confidence in optimizing designs, materials, and construction practices and represent the current and future industry direction for implementation of broader direct measurement programs.

Figure 5. Cyclic stress versus in situ M_r of base and subgrade layers, and universal model parameters separately for each layer.

REFERENCES


Resilient Modulus Mapping Using Validated Intelligent Compaction: Case Study on Colorado I-25 North Express Lanes

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ABSTRACT

The I-25 North Express Lanes Project (Johnstown to Fort Collins, CO) involves rehabilitation and reconstruction of existing pavement, interchanges, and adding express lanes. The project design involved optimizing the pavement system to meet the project design requirements using AASHTOWare® Pavement mechanistic empirical (ME) design. The pavement foundation layers were designed based on target in situ resilient modulus values for subgrade, subbase, and a mechanically stabilized aggregate base layer. Traditional quality inspection of moisture-density control and modern automated plate loading testing (APLT) were used at point locations to verify achievement of the minimum design requirements. Given the size of the project and the desire to achieve uniform pavement foundation modulus values, validated intelligent compaction (VIC) testing was implemented on a portion of the project. VIC involves outfitting a smooth drum roller with a set of sensors and a data acquisition system with on-board display to provide real-time geospatial mapping of the compaction layer. The VIC system was field calibrated to output stress-dependent composite resilient modulus values, which enabled the contractor to monitor compaction quality in real-time and make immediate improvements in “weaker” areas potentially unforeseen through random point testing. By improving compaction and monitoring modulus in real-time, better uniformity in support conditions were achieved. This paper describes the VIC calibration process, shows results of in situ resilient modulus maps, and details of how the roller operator used the VIC roller to improve compaction quality. Results demonstrate how the VIC technology can be used for achieving high quality compaction and verifying design values in situ.

1. INTRODUCTION

The new AASHTOWare™ Pavement mechanistic empirical (ME) design guide allows the use of stress-dependent resilient modulus ($M_r$) as the key design input parameter for foundation layer. For the ongoing Interstate 25 (I-25) North Express Lanes Project between Johnstown and Fort Collins, CO, which consists of nearly 15-mile-long rehabilitation and reconstruction of existing pavements and adding new express lanes, pavement design optimization was performed using the ME design method. The design optimization was performed by selecting in situ measured $M_r$ values for the foundation layers on similar materials at a nearby project. The optimized pavement foundation layer profiles used on this project included a nominal 6 in. of mechanically stabilized base (MSB) underlain by 24 in. of granular subbase
and natural subgrade and nominal 6 in. of MSB directly over the compacted subgrade. The MSB layer consisted of a multi-axial geogrid with hexagonal structure and triangular apertures placed at the MSB/subbase interface.

The project quality assurance (QA) specifications required the use of in situ cyclic plate load testing (PLT) for field verification of the assumed \( M_r \) values, beyond achieving the traditional moisture-relative compaction QA criteria. In situ \( M_r \) field verification testing was conducted using the Automated Plate Load Testing (APLT), by performing a 1,100 cycle multi-stress sequence cyclic PLT. Considering the size of the project and the desire to achieve uniform \( M_r \) values, validated intelligent compaction (VIC) testing was implemented on a portion of the project. VIC differs from the traditional intelligent compaction (IC), where in VIC outputs include a design moduli value (stress-dependent \( M_r \) for this project) based on field calibration testing.

In this paper, the results from the VIC field calibration testing, example results from production operations where the contractor used the VIC map results to make real-time improvements, and analysis of VIC mapping results highlighting the concept of intelligent inspection with prioritized areas of rework are presented.

2. FIELD TESTING

2.1 Validated Intelligent Compaction (VIC)

VIC is a technique that uses advanced data analytics and requires site specific calibration of the roller sensor measurements using in situ plate load test measurements (e.g., in situ resilient modulus). The VIC calibration process utilizes the full spectrum of the drum acceleration signature rather than pre-selected frequencies as in case of compaction meter value (CMV). Recent field calibrations on subgrade and base mate materials using this approach showed coefficient of determination \( (R^2) > 0.9 \) are achievable using this technique [compared to \( R^2 \) of 0.3 to 0.6 using CMV for the same data (White et al. 2018, White et al. 2014, White et al. 2019a)]. Because of the high \( R^2 \) values and relatively low measurement errors associated with this site calibration process, the calibration relationships can be reliably used to develop the desired mechanical property maps for quality control/acceptance. With VIC, the output maps are validated mechanical property outputs (e.g., stress-dependent \( M_r \) as documented in this paper).

In this study, a Caterpillar smooth drum vibratory roller weighing about 27,450 lbs outfitted with Ingios VIC retrofit system was used (Figure 1a), which comes with an on-board customizable display for real-time viewing of mapping results (Figure 1b). A valid field calibration effort should require statistical determination of the minimum number of test measurements needed to achieve a desired level of reliability and confidence level in future predictions. The minimum sample size needed for this calibration effort was determined using a procedure recommended by Dupont and Plummer (1998) and is graphically presented in Figure 2. The inputs needed to determine the minimum sample size include the mean and coefficient of variation (COV) of the measured and the predicted values, standard error of the regression fit, the expected slope of the regression fit between the measured and the predicted values, and desired confidence level in the future estimates. These inputs were first estimated based on past experience but were later clarified based on the in situ calibration test results.
Figure 1. (a) Smooth drum vibratory roller used for VIC calibration and APLT testing; (b) VIC measurements on-board display in the roller cab; and (c) cyclic APLT setup with 12 in. diameter loading plate and layered analysis sensor kit.

Figure 2. Statistical estimation of minimum sample size required for VIC calibration based on standard error of predicted values and target confidence level in the predictions.
2.2 Automated Plate Load Testing (APLT)

APLT is a state-of-the-art test device used to characterize a variety of in situ mechanistic performance parameter values for pavement and pavement foundation layers (see White and Vennapusa 2017, White et al. 2019b). The advantages of using in situ cyclic testing to determine M, using APLT is the ability to perform a conditioning stage like a laboratory M, test (AASHTO T307-99) and obtain M, values at various cyclic stresses.

Cyclic APLT using a 12 in. diameter loading plate (Figure 1b) was conducted to determine in situ composite resilient modulus (M_{r,\text{comp}}) at six different stress levels, which involved one conditioning sequence with 500 cycles followed by six loading steps with 100 cycles each. Average of the last 5 cycles from each step were used for M_r calculations. Both resilient (rebound) and permanent deformations (\delta_p) are monitored for each loading cycle. The nominal maximum stresses in each loading step were: 15 psi (conditioning), 5 psi, 10 psi, 15 psi, 20 psi, 30 psi, and 40 psi. A contact stress of about 1.5 psi was maintained during each loading cycle. Composite resilient modulus (M_{r,\text{comp}}) was calculated using the applied cyclic stresses and the resilient deflection (during unloading) using the Boussinesq’s half-space equation:

\[ M_{r,\text{comp}} = \frac{(1-\eta^2)\Delta\sigma_{\text{cyclic}} r}{\delta_r} \times F \]  

where: \( M_{r,\text{comp}} \) = in situ composite resilient modulus (psi), \( \delta_r \) = the resilient deflection of plate (in.) during the unloading portion of the cycle (determined as the average of three measurements along the plate edge), \( \eta \) = Poisson’s ratio (assumed as 0.4), \( \Delta\sigma_{\text{cyclic}} \) = cyclic applied stress (psi), \( r \) = radius of the plate (in), \( F \) = shape factor depending on stress distribution (assumed as 8/3 for granular materials). Using the criteria of 1 to 1.5 times the plate diameter for measurement influence depth, the M_{r,\text{comp}} values have an influence depth of about 12 to 18 in.

The M_r results were used to model the behavior using the ME design guide “universal” model (AASHTO 2015) shown in Eq. (2):

\[ M_r = k_1^* P_a \left( \frac{\theta}{P_a} \right)^{k_2^*} \left( \frac{\tau_{\text{oct}} + 1}{P_a} \right)^{k_3^*} \]  

where, \( M_r \) = in situ resilient modulus (psi); \( P_a \) = atmospheric pressure (psi); \( \theta \) = bulk stress (MPa) = \( \sigma_1 + \sigma_2 + \sigma_3 \); \( \sigma_1 \) = applied cyclic stress (\Delta\sigma_{\text{cyclic}}) used in M_{r,\text{comp}} calculations because there is no confining stress at the surface; \( \sigma_2 = K_o \sigma_1 \); \( \sigma_3 = \sigma_2 \); \( K_o \) = coefficient of lateral earth pressure at rest = \( \eta/(1-\eta) \); \( \eta \) = Poisson’s ratio assumed as 0.4; \( \tau_{\text{oct}} \) = octahedral shear stress (MPa) = \( \sqrt{(\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} / 3 \); and \( k_1^*, k_2^*, \) and \( k_3^* \) = regression coefficients determined from in situ testing (these coefficients are presented herein with an asterisk (*) to differentiate with the regression coefficients traditionally developed using laboratory test results).
3. MATERIALS, DESIGN-ASSUMED VALUES, AND FIELD REFERENCE VALUES

In situ testing for VIC calibration was performed in three areas to capture a range of resilient moduli ($M_{r\text{-Comp}}$) values for different material and layer conditions (i.e., subgrade, subbase over subgrade, MSB over subbase, and MSB over subgrade). The subgrade layer material is classified as A-6(5), subbase layer material as A-2-6, and MSB layer material is classified as A-1-a material, according to the AASHTO Soil Classification System. The three calibration testing areas included:

- **Area 1**: freshly compacted nominal 24-inch-thick A-2-6 subbase layer placed over native subgrade material. APLTs were performed at 10 selected locations in this area.
- **Area 2**: compacted MSB layer. The base layer in this area was placed and compacted about month before the calibration work. The profile at the test locations consisted of 5.5 in. to 7.25 in. MSB over nominal 2 ft of A-2-6 subbase, with geogrid at the MSB/subbase interface. APLTs were performed at 6 selected locations.
- **Area 3**: freshly moisture conditioned and compacted subgrade. The subgrade in this area was prepared by scarifying, moisture-conditioning, and recompacting the material. APLTs were performed at 3 selected locations on the subgrade.

Field reference values for the different areas/pavement foundation layer profiles are summarized in Figure 3, based on values assumed in the ME design which are also shown in the figure. The target $M_{r\text{-Comp}}$ for Area 1 is 8,310 psi, Area 2 is 11,560 psi, and Area 3 is 5,355 psi.

![Figure 3. In situ $M_r$ reference values established based on design assumed moduli values.](image)

4. VIC CALIBRATION AND MAPPING RESULTS

APLT results showing applied cyclic stress ($\sigma_{\text{cyclic}}$) versus $M_{r\text{-comp}}$ values obtained from three different test areas are shown in Figure 4. A cyclic stress level of 40 psi was selected for calibration analysis to match the stress conditions anticipated during compaction. The $M_{r\text{-Comp}}$ values used in the calibration analysis varied between 2.1 ksi and 64.7 ksi and averaged about 25.1 ksi, at the 19 test locations.

The VIC calibration record is provided in Figure 5. The calibration regression analysis yielded a coefficient of determination ($R^2$) $\geq 0.9$, and a standard error in the predictions of about 5.8 ksi which represents about 23% error relative to the mean value of 25.1 ksi. As a verification of the calibration analysis,
The statistical determination of the minimum number of test measurements needed to achieve the desired level of reliability and confidence level in future predictions was analyzed. For this calibration effort, a minimum of 12 test measurements were required to maintain a 99% confidence in the predictions at 90% reliability (see Figure 2). The calibration effort for this project used results from 19 test locations, thus exceeding the minimum number of measurements required. The VIC mapping results output operator selected material type (aggregate base, subbase, subgrade, or other), machine pass count, \( M_{\text{Comp}} \), and delta \( M_{\text{Comp}} \) (= VIC \( M_{\text{Comp}} \) - Target \( M_{\text{Comp}} \)). A summary of the geospatial maps with subbase layer material is shown in Figure 6.

![Figure 4](image1)

**Figure 4.** Cyclic APLT results showing cyclic stress-dependent \( M_{\text{Comp}} \) values from one selected test point each on geogrid stabilized base layer, subbase layer, and subgrade layer.

![Figure 5](image2)

**Figure 5.** Results of calibration showing VIC-\( M_{\text{Comp}} \) versus actual measured \( M_{\text{Comp}} \) using cyclic APLT along with regression and measurement statistics.

<table>
<thead>
<tr>
<th><strong>Regression Statistics</strong></th>
<th></th>
</tr>
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<tbody>
<tr>
<td>( N )</td>
<td>19</td>
</tr>
<tr>
<td>( R^2 )</td>
<td>0.925</td>
</tr>
<tr>
<td>( R^2(\text{adj.}) )</td>
<td>0.904</td>
</tr>
<tr>
<td>( \text{RMSE} )</td>
<td>5,822 psi</td>
</tr>
<tr>
<td>( F\text{-value} )</td>
<td>105.87</td>
</tr>
<tr>
<td>( p\text{-value} )</td>
<td>&lt;0.0001</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Measurement Statistics</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>( \text{Min.} )</td>
<td>2,108 psi</td>
</tr>
<tr>
<td>( \text{Max.} )</td>
<td>64,665 psi</td>
</tr>
<tr>
<td>( \text{Mean} )</td>
<td>25,096 psi</td>
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<tr>
<td>( \text{Median} )</td>
<td>19,644 psi</td>
</tr>
<tr>
<td>( %SE^{**} )</td>
<td>23.20%</td>
</tr>
</tbody>
</table>

**Percent error in prediction relative to mean**

---

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Figure 6. VIC-Maps showing material type, pass count, resilient modulus, and delta resilient modulus (in situ $M_r$ – reference $M_r$).
Key attributes of the VIC system, as it relates to the project specification and implementation are as follows:

- The VIC system features base station-free satellite-based augmentation system (SBAS) global positioning system (GPS).
- The VIC system displays color-coded spatial map of calibrated compaction measurements (i.e., composite resilient modulus – $M_{r,\text{Comp}}$) over a georeferenced aerial map of the project area. The aerial map also includes project mainline stationing, where line drawings were made available. The real-time display shows the % passing the reference target value, the compaction quality index, and the coefficient of variation. The reference $M_r$ values for each layer are summarized in in the next section of this paper.
- The VIC system has been calibrated with 12 in. diameter loading plate cyclic APLTs measuring the $M_{r,\text{Comp}}$, which has a measurement influence depth of about 18 to 24 in. below surface (1.5 to 2 x plate diameter).
- The VIC compaction report system was setup within a secure Microsoft® Azure cloud-based dashboard system. Automated email alerts were programmed to be sent out with a link to a secure Ingios web portal when a VIC map is initiated, completed, successfully downloaded, and a report is generated. Also, users with a secure login could view the mapping activity in real-time through a secure link within the dashboard. Once the operator finishes mapping and pushes the “stop” button, the data file is submitted to the Ingios cloud server where the data is processed to auto-generate the VIC compaction reports. The VIC compaction report is in a *.pdf format and include the $M_r$ map, material map, pass count map, quality analysis metrics, a data summary table, and compaction statistics.
- Three compaction quality criteria were reported as part of the VIC compaction report:
  - **Compaction Quality Index (CQI)**, which is a relative compaction index based on the percentage of the geospatial area that meets the minimum target values and accounts for the uniformity of compaction. The default minimum target CQI is 95% using a uniformity weight factor of 50%.
  - **Percent Passing Target Values** calculated based on the number of geospatial grid points from the VIC output that meet or exceed the minimum reference $M_r$ for the selected material. The default target %_Passing TV ≥ 80%
  - **Coefficient of Variation (COV)** calculated based on the $M_r$ values reported at each grid point of the mapping area. The default target COV is ≤ 20%.

An additional feature of VIC dashboard analysis, although not implemented on this project, was intelligent inspection analysis using Ingios i-score “blob” analyzer. An example of the “blob” analysis for the subbase area maps shown in Figure 6 is shown in Figure 7, which identifies regions within the map that are prioritized for additional re-work due to not meeting the target $M_{r,\text{comp}}$ value.
5. SUMMARY AND CONCLUSIONS

In this paper, results of field calibration testing and VIC mapping with stress-dependent $M_r$ values on the on-going I-25 Express Lanes project between Johnstown and Fort Collins, CO, are presented. $M_r$-Comp measured using a 12 in. diameter loading plate at a selected stress level (40 psi), georeferenced spatial maps of VIC-$M_r$-Comp values have been generated on the project for a short period. The VIC calibration relationship from this project yielded an $R^2$ of 0.92 between the measured $M_r$-Comp and VIC-$M_r$-Comp values. Field reference target values for $M_r$-comp have been calculated for this project, based on design layer modulus values.

The real-time on-board display of VIC mapping results showed the percentage passing the reference target values, the compaction quality index, and the coefficient of variation. This enabled the contractor to monitor compaction quality in real-time and make immediate improvements in “weaker” areas, that could potentially go unforeseen with random point testing. By improving compaction and monitoring modulus in real-time, better uniformity in support conditions were achieved.

The VIC compaction reporting system was setup within a secure Microsoft® Azure cloud-based dashboard system, which generated automated email alerts with a link to a secure Ingios web portal when a VIC map is initiated, completed, successfully downloaded, and a report is generated. The dashboard also slowed real-time viewing of the mapping activity. The VIC compaction report was auto-generated within minutes of completing the map in a pdf format include the $M_r$-comp map, material map, pass count map, quality analysis metrics, a data summary table, and compaction statistics.
Results of a next-generation intelligent inspection analysis feature (although not used on this project) using Ingios i-score “blob” analyzer is demonstration in this paper, which can be used to detect and prioritize areas of a map that does not meet the target value and need additional rework. The analysis tool will assist contractor in focusing the rework efforts and the QC/QA inspectors to perform inspection testing to only areas that are needed.

REFERENCES


The Influence of Sloped Shoulder on Pavement Responses Using Full-Scale Experiments

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Raj V. Siddharthan, Elie Y. Hajj, Mohamed Nimeri, Sherif Elfass  
*University of Nevada – Reno, Department of Civil and Environmental Engineering*

**ABSTRACT**

Rural roads, particularly no-shoulder or narrow shoulder roads, are highly prone to slope failure due to the heavy truck loading. Slope failure causes safety problems and traffic delays and it is often difficult to repair quickly. Slope stability is one of the common design problems in geotechnics and several slope stability methods that use classical limit equilibrium analysis are available. In these methods, applied surcharge load and corresponding stresses are distributed in the soil medium often using Boussinesq solution. The calculated horizontal stress distribution by this solution is then doubled to account for the non-existing laterally extended soil condition (i.e., presence of slope). However, this assumption (i.e., stress adjustment factor, SAF, of two) may not be applicable when a layered pavement structure with distinctly different strength and stiffness properties is subjected to a moving load. In this study, in order to investigate the applicability of this assumption (i.e., SAF equal to 2) for a pavement loading condition, two full-scale laboratory experiments were conducted at University of Nevada, Reno. A careful comparison between control experiment (which had no sloped pavement shoulder) and slope experiment (which had sloped shoulder) revealed that the sloped shoulder plays a major role in stress distribution within a typical pavement structure, particularly on the slope side with respect to the surface loading. The influence of side slope on the stress distribution resulted in 40% to 80% increase in the measured stresses. Accordingly, a SAF of 1.6 is recommended when stress distribution near a sloped pavement shoulder is of interest.

**1. INTRODUCTION**

The rural low volume roads (LVRs) are typically narrower than high volume highway roads and vehicles travel close to the pavement edge, and therefore slope failure is one of the major concerns for such roads. Slope failure can become more critical in no-shoulder or narrow shoulder rural roadways with side slopes. Slope failure causes safety problems and traffic delays and it is often difficult to repair quickly.

During the past decades, several factors such as regional change in economic growth patterns, changing manufacturing and farming practices, etc. have raised the need for the movement of unconventional farm machinery and superheavy load (SHL) vehicles in the LVRs (Douglas, 2016; Siddharthan et al., 2005). SHL vehicles are much larger in size and weight compared to the standard trucks and they may involve gross vehicle weights in excess of a few million pounds. These vehicles require specialized trailers and hauling units and they do often travel at much lower speeds (Hajj et al., 2018). Although it is
recommended to keep SHL vehicles as far away as possible from the pavement edge to avoid slope failure, it is not always feasible when a wide heavy vehicle travels along a narrow roadway.

Slope stability analysis is a common design concern in geotechnical practice. Such an analysis encompasses domain, geometry, failure planes, and shear strength parameters, etc. (Das and Sobhan, 2014; Berg et al., 2009; U.S. Army Corps of Engineers, 2003). Classical slope stability methods are based on the limit equilibrium analysis of a mass of soil bounded between assumed possible slip surface(s) and slope surface. Failure is investigated by comparing the driving and resisting forces and moments. In order to estimate the resultant horizontal force due to surcharge load, Boussinesq theory is commonly used in geotechnical practice. This theory assumes elastic, homogenous, isotropic soil medium that extend laterally to infinity. Thus, the calculated horizontal stress using Boussinesq solution is doubled to account for the non-existing laterally extended soil condition (i.e., presence of slope). In other words, the stresses calculated by Boussinesq solution are multiplied by a Stress Adjustment Factor (SAF) of 2. This SAF is routinely used in retaining wall design where the lateral force on the wall need to be estimated (Das and Sobhan, 2014; Berg et al., 2009; U.S. Army Corps of Engineers, 2003). However, the use of Boussinesq theory along with the SAF of two may not be applicable when a layered pavement structure with distinctly different strength and stiffness properties is subjected to a moving load.

2. RESEARCH OBJECTIVE

In this study, the applicability of using SAF of two for a pavement loading condition (i.e., layered medium and impact load) is evaluated. To this end, a full-scale experiment with a sloped edge pavement (i.e., sloped experiment) and a control experiment which had no sloped pavement shoulder were conducted at University of Nevada, Reno. A careful comparison between the two experiments identified the role of a sloped edge in the stress distribution within a typical pavement structure. The current study is part of a Federal Highway Administration (FHWA) project on “Analysis Procedures for Evaluating Superheavy Load Movement on Flexible Pavements” (Hajj et al., 2018).

3. EXPERIMENTAL PROGRAM

As illustrated in Figure 1, Experiment No. 3 that is considered as the control experiment (no side slope) consisted of 5 inch of asphalt concrete (AC), 6 inch of crushed aggregate base (CAB), and 66 inch of subgrade (SG). In Experiment No. 4 (sloped experiment), a similar pavement structure with a side slope of 1:1.5 (33.7 degrees with the horizontal) was constructed (see Figure 2). It should be noted that effort was made to use same materials and apply similar compaction practices in both experiments. A detailed discussion regarding the construction procedure, instrumentation, material properties can be found elsewhere (Nimeri et al., 2018; Nabizadeh Shahri, 2017).
Figure 1. Pavement layer thicknesses and instrumentation plan in Experiment No. 3 (control experiment).
Figure 2. Pavement layer thicknesses and instrumentation plan in Experiment No. 4 (sloped experiment); (a) side view, (b) plan view in a smaller scale.

In both experiments, Falling Weight Deflectometer (FWD) at various load levels (~ 9,000, 12,000, 16,000, 21,000, 25,000 lb) were applied at the pavement surfaces. As illustrated in Figure 2, the FWD loads in Experiment No. 4 were applied at 12, 24 and 36 in. from the edge of the pavement slope (herein referred to as Loc12, Loc24, and Loc36, respectively). As shown in Figure 1 and Figure 2, surface deflections using Linear Variable Differential Transformers (LVDTs) at the center of loading plate and different locations away from the center of the plate were measured. In addition, several Total Earth Pressure Cells (TEPCs) with 4 in. diameter were installed in the base and subgrade layer to capture the load induced vertical stresses during the load application.

4. COMPARISON OF STRESS MEASUREMENTS

First, it deemed necessary to ensure that the pavement layers in both experiments were compacted to a similar and desired level of densities. Figure 3 shows the measured deflection basins from FWD tests in Experiment No. 3 and Experiment No. 4 at Loc36 (i.e., 36 in. from the edge of the pavement slope). This figure implies that when the surface loads were applied far enough from the slope edge in Experiment No. 4, the measured surface deflections were similar to the deflections measured in Experiment No. 3 (control experiment), indicating that the density and stiffness properties of the pavement layers in these two experiments were reasonably similar. Consequently, experiment No. 3 can be treated as a control experiment so that any difference in stress measurements at the same location in both experiments can be attributed to the sloped edge.
To determine the SAF using the conducted experiments, the measured vertical stresses at the location of Total Earth Pressure Cells (TEPCs) in Experiment No. 4 were compared to the corresponding measured stresses in Experiment No. 3. In other words, the stress distributions in these two pavement structures were compared by monitoring the TEPCs measurements that are located at the similar positions relative to the applied surface load.

Figure 4 and Figure 5 show the measured vertical stresses in the subgrade on the nonslope side of the pavement structure with respect to the location of the applied surface load. Compared to the corresponding measured stresses in experiment No. 3, the stress distribution in the nonslope side of the pavement structure was not affected by the sloped edge.
Figure 5. Comparison between measured vertical stresses in Experiment No. 3 and experiment No. 4 (nonslope side, load applied at Loc12 and Loc36, TEPC at 6 inch from subgrade surface, offset from the centerline of the load equal to 24 inch).

Figure 6 and Figure 7 depict the load-induced vertical stresses measured by the TEPCs that were installed directly under the centerline of the load at different depths in the subgrade. These figures reveal the noticeable increase (about 70%) in the measured vertical stresses in Experiment No. 4 compared to the Experiment No. 3 (i.e., control experiment). Figure 8 represents the measured vertical stresses at the middle of the base layer and centerline of the load. It may be noted that, the TEPC installed at this location failed measuring vertical stresses at the load levels of 21,000 and 25,000 lb. This figure also implies that the lack of lateral support due to the presence of slope gave rise to 40% increase in the load-induced vertical stresses in the middle of the base.

Figure 9 to Figure 11 show the load-induced vertical stresses in Experiment No. 4 measured by the TEPCs that were located in the slope side with respect to the location of applied surface load. In other words, the TEPCs were closer to the slope edge relative to the surface loads and therefore, these figures represent the stress distribution adjacent to pavement slope.

As shown in Figure 9, when the surface loads were applied at 12 and 24 in. from the pavement edge, the effect of side slope and the lack of lateral support resulted in 80% increase in the induced vertical stresses. Figure 10 and Figure 11 depict the induced measured stresses as the surface loads were applied at Loc36. These figures represent 40% to 80% raise in the measured vertical stresses.

Based on the above presented observations obtained from these full-scale experiments, it can be concluded that the side slope and the lack of lateral support significantly influence the stress distribution in flexible pavement structures. Furthermore, the impact of slope on the stress distribution becomes more critical as the surface load moves closer to the slope.
Figure 6. Comparison between measured vertical stresses in experiment No. 3 and Experiment No. 4 (load applied at Loc12, TEPC at 20 inch from subgrade surface, centerline of the load).

Figure 7. Comparison between measured vertical stresses in experiment No. 3 and Experiment No. 4 (load applied at Loc12, TEPC at 6 inch from subgrade surface, centerline of the load).
Figure 8. Comparison between measured vertical stresses in experiment No. 3 and Experiment No. 4 (load applied at Loc12, TEPC at middle of the base, centerline of the load).

Figure 9. Comparison between measured vertical stresses in experiment No. 3 and Experiment No. 4 (slope side, load applied at Loc12 and Loc24, TEPC at 6 inch from subgrade surface, offset from the centerline of the load equal to 12 inch).
Figure 10. Comparison between measured vertical stresses in experiment No. 3 and Experiment No. 4 (slope side, load applied at Loc36, TEPC at 6 inch from subgrade surface, offset from the centerline of the load equal to 12 inch).

Figure 11. Comparison between measured vertical stresses in experiment No. 3 and Experiment No. 4 (slope side, load applied at Loc36, TEPC at 6 inch from subgrade surface, offset from the centerline of the load equal to 24 inch).

5. SUMMARY AND CONCLUSION

The low volume roads (LVRs) are extensively used by heavy trucks and unconventional farm machineries and they provide access to the high-volume transportation system. The movements of superheavy load vehicles on these roads have become more common over the years. These vehicles are very large in size
and weight (may involve gross vehicle weights in excess of a few million pounds) and they often travel at much lower speeds. Since the rural LVRs are narrower than high-volume highway roads, the vehicles frequently travel close to the pavement edge. Thus, slope failure has been always considered as one of the major concerns for LVRs.

Slope stability analysis is a common design problem in geotechnics and thus, several slope stability methods using classical limit equilibrium analysis are available. In these methods, applied surcharge load and corresponding stresses are distributed in the soil medium often using Boussinesq theory. This theory assumes elastic, homogenous, isotropic soil medium that extend laterally to infinity. Thus, to account for the non-existing laterally extended soil condition (i.e., presence of slope), the stresses calculated by Boussinesq solution are multiplied by a Stress Adjustment Factor (SAF) of two.

In this study, the applicability of using SAF of two for a pavement loading condition (i.e., layered medium and impact load) was evaluated. A full-scale experiment with a sloped edge pavement (i.e., sloped experiment) and a control experiment which had no sloped pavement shoulder were conducted. Table 1 summarizes the comparison between load-induce vertical stresses measured in Experiment No. 4 (i.e., slope experiment) and Experiment No. 3 (i.e., control experiment). It was found that the side slope can significantly affect the stress distribution within the pavement structure. The influence of side slope on the stress distribution resulted in 40% to 80% increase in the load-induced measured stresses. Accordingly, a SAF of 1.6 is recommended when stress distribution near a sloped pavement shoulder is of interest.

Table 1. Summary of comparison between measured stresses in Experiment No. 4 and Experiment No. 3.

<table>
<thead>
<tr>
<th>Total Earth Pressure Cell</th>
<th>Depth</th>
<th>Surface Load Location in Experiment No. 4</th>
<th>Offset from the Centerline of the Load</th>
<th>% Increase in the Measured Stress Compared to Experiment No. 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>20 in. from subgrade surface</td>
<td>Loc12</td>
<td>0 in</td>
<td>80%</td>
</tr>
<tr>
<td>P10</td>
<td>6 in. from subgrade surface</td>
<td>Loc12</td>
<td>0 in</td>
<td>40%</td>
</tr>
<tr>
<td>P5</td>
<td>Middle of the base</td>
<td>Loc12</td>
<td>0 in</td>
<td>40%</td>
</tr>
<tr>
<td>P9S</td>
<td>6 in. from subgrade surface</td>
<td>Loc12</td>
<td>12 in</td>
<td>80%</td>
</tr>
<tr>
<td>P10</td>
<td>6 in. from subgrade surface</td>
<td>Loc24</td>
<td>12 in</td>
<td>80%</td>
</tr>
<tr>
<td>P9</td>
<td>6 in. from subgrade surface</td>
<td>Loc36</td>
<td>12 in</td>
<td>40%</td>
</tr>
<tr>
<td>P10</td>
<td>6 in. from subgrade surface</td>
<td>Loc36</td>
<td>24 in</td>
<td>80%</td>
</tr>
</tbody>
</table>
REFERENCES


Methodology for the Design of Optimum Asphalt Content of Asphalt-Treated Bases

Danniel D. Rodriguez, Jose Garibay, Soheil Nazarian

Center for Transportation Infrastructure Systems, The University of Texas at El Paso

ABSTRACT

Designing asphalt-treated base (ATB) can follow the principles of a low-quality hot mix asphalt (HMA) design or the approach of a high-quality base. Designing the ATB optimum asphalt content (OAC) per HMA design process requires satisfying volumetric criteria. Designing the OAC for a high-quality base employs the development of an asphalt content-density curve. Following either of these methods typically requires the mix also satisfying minimum strength criteria. Given inherent economic constraints, it is usually not practical to change ATB mix gradations and, thus; difficult to satisfy all the volumetric criteria of a low-quality HMA-driven design. The design process is further varied when considering other procedural factors, such as the type of compactor used (e.g. Texas Gyratory Compactor or Superpave Gyratory Compactor), the applied number of gyrations, and the presence of recycled materials. This study presents a method for designing the optimum asphalt content of ATB utilizing principles of a high-quality base design approach. The impacts from the type of compactor used, the number of applied gyrations, and the presence of recycled asphalt shingles (RAS) are also assessed. A laboratory study was performed with limestone dolomite materials from four different sources. It was found that the ATB design for OAC following a low-quality HMA approach resulted in slighter higher asphalt contents (~OAC 4.5%) when compared to mixes designed through a high-quality base approach (~OAC 3.7%). Within each design approach, the resulting OAC is shown to be independent of the gyratory compactor used and the number of applied gyrations (considering 75 versus 100 gyrations). Results showed the addition of 3% RAS to the mixes resulted in a 10% to 25% increase in the IDT strength.

1. BACKGROUND

Asphalt-stabilized base (ASB) is placed to provide a waterproof, structural underlayment to HMA surface course. ASB is generally composed of lower quality material than that of HMA and also adheres to less stringent quality control standards in the form of wider gradation bands and lower to moderate asphalt contents. The design of ASB is not a well-defined procedure (Nazzal, 2009). The recommendations for ASB design vary. The process can consist of electing a predefined gradation and asphalt content that satisfies performance testing, such as load wheel tracking (LWT) and indirect tensile strength (IDT) (Nazzal, 2009). The design may also be performed in a manner that is similar to that of designing for the optimum moisture content of high-quality granular base, or for the OAC of low-quality hot mix asphalt (HMA). The design of ASB has been evaluated by means of HMA convention through Marshall mix design method (Li, 2010), SUPERPAVE design and Bailey method (Hua, 2008). However, it is often not practical to adjust the gradation in order to satisfy all the volumetric and performance requirements considered in HMA designs. In actuality, the criteria for passing the LWT requirements and design strength are often waived.

Volumetrically, the OAC of an HMA is generally defined at a specified molded density. Figure 1 displays a typical design function for obtaining the OAC. As with typical HMA design, the procedures call for
evaluation of the mix to meet minimum voids in mineral aggregate (VMA) requirements; depending on the type of HMA design mix. The minimum design VMA can be as low as 12% for dense-graded mix and up to 19% for stone matrix asphalt mix (Texas DOT, 2008). Specifying a minimum VMA is also not a common criterion in practice. Employing HMA design for ASB is almost always based on the developed relationship between the asphalt content and total density.

![Figure 1. Design of ASB following low-quality HMA](image)

HMA design is generally performed through use of the Superpave Gyratory Compactor (SGC) or in some highway agencies (such as TxDOT) with the small Texas Gyratory Compactor (TGC). The laboratory study presented in this paper uses both compactors, following Texas specifications.

TGC accommodates HMA design and molds 4 in. x 2 in. specimens. Given the size of the mold, aggregates that are 0.75 in. diameter and larger are scalped out. Compaction occurs at 250°F after 2-hours of curing. The material is pressed with the ram to 50 psi, and then gyrated three times. This process is repeated until the pressure reaches 150 psi after one pump of the ram. The pressure is then pumped to 2,500 psi, and then relaxed to complete the molding process.

Designing with the SGC employs 6 in. x 4.5 in. specimens. Material is compacted at a temperature of 290°F after 2-hours of curing. The compactor tilts and gyrates at a rate of 30.0±0.5 gyrations per minute at an angle of 1.25±0.02°. During compaction, the ram application pressure maintains 87±2 psi perpendicular to the cylindrical axis of the specimen. Specimens are molded up to 75 gyrations or 100 gyrations. However, previous research has shown the locking points of most ASB mixes to be less than 75 gyrations (Hernandez, 2012).

Strength testing for design of ASB has involved unconfined compressive strength (UCS) and indirect tensile strength (IDT). UCS recommended testing requires 6 in. x 8 in. specimens. These size of specimens have been produced through use of the Texas Base Gyratory Compactor (TBGC). However, agency-adopted compaction method has shifted towards primary use of the SGC. It follows that the preference for strength testing consists of IDT on 6 in. x 4.5 in. specimens. The IDT performed in this study applies the compressive load at a controlled deformation rate of 2 in. per minute.
This study will propose a design method that is similar to that of designing high-quality base. The method is evaluated in comparison to HMA design through use of the SGC and TGC. It is also becoming popular for highway agencies to include high percentages of reclaimed asphalt pavement (RAP) and some recycled asphalt shingles (RAS) into the mix designs. Therefore, the materials used for the evaluation all contained at least 20% RAP content. Sample sets consisting of added 3% RAS content were also evaluated. The impact on the OAC by the design convention (high-quality base design vs. HMA design), number of gyrations and RAS content are assessed. Additionally, the impact of the RAS content on strength is also presented.

2. PROPOSED DESIGN PROCESS: HIGH-QUALITY BASE APPROACH

This approach follows a similar process to that of designing for the optimum moisture content of unbound granular bases. As shown in Figure 2, the design OAC is based on relationships between the asphalt content and total density, relative density, and indirect tensile strength (IDT) from a set of at least three laboratory specimens, molded with different asphalt contents. The objective of this design procedure was to obtain a practical OAC between 3% and 6%. This design calls for compacting the specimens with 75 gyrations of SGC.

A sample ASB design is presented in Figure 2 to demonstrate the proposed method. Design criteria of at least an IDT of 85 psi and a relative density of 97% are recommended (Texas DOT, 2013). Depending on the designer preference, the OAC values can be biased toward a “dry mix” and a “rich mix.” The steps associated with this procedure consists of obtaining the following parameters:

1. **AC max density**: is defined as the AC at the maximum total density of the observed data set (Figure 2a).
2. **AC relative density**: the AC values at a relative density of 97% for a “dry” design (left of vertex) and for a “rich” design (right of vertex), as shown in Figure 2b.
3. **AC strength**: the AC values meeting strength of 85 psi for a “dry” design and for a “rich” design. See Figure 2c.
4. **AC critical**: minimum of AC relative and AC strength, respectively for “dry” and “rich” design.
5. **OAC design**: average of AC max density and AC critical, respectively for “dry” and “rich” design (See Figure 2d). In the case where the strength design curve exceeds the design criteria at every point, then the OAC design is defined at 97% relative density for a dry and rich design. This signifies that the strength has no impact on the design since the criteria is met at every asphalt content.
3. EVALUATION OF PROPOSED DESIGN METHOD

3.1 Experimental design

A study was performed to compare the OAC obtained from the proposed high-quality base design method with OAC obtained from low-quality HMA design method. Common base materials from Texas were used in this case study. Table 1 contains the experiment design. The experiment design was executed on virgin material mixed with RAP, and with virgin material mixed with RAP and RAS. Each sample set consisted of one trial specimen per asphalt content. Each sample set was prepared and tested for the high-quality base method and low-quality HMA method. That is, for each set, the volumetric properties were obtained first for OAC analysis with HMA design, and then followed with the IDT for OAC analysis with high-quality base method. In this paper, the OAC values obtained from the two methods are distinguished by: 1) OACbase-design and 2) OACHMA-design. The low-quality HMA method was further evaluated to compare the OAC obtained by compacting with SGC and TGC. The test process...
is depicted in Figure 3. The specimens were mixed, molded, then measured for volumetric properties (bulk specific gravity and theoretical maximum specific gravity) and finally subjected to the IDT strength tests.

Table 1. Experimental design plan and resulting OAC values

<table>
<thead>
<tr>
<th>Mix Design Procedure</th>
<th>No. Gyations</th>
<th>Asphalt Contents</th>
<th>OAC Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>3%</td>
<td>3.5%</td>
</tr>
<tr>
<td>Base design</td>
<td>75</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>HMA Design (TGC)</td>
<td>(psi values)</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>HMA Design (SGC)</td>
<td>100</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

The tested materials represent typical ASB mix designs placed in Texas and were retrieved as raw, unmixed material. To incorporate RAS to the mix designs, 3% of the RAP percentage was replaced with RAS. The following are the material characteristics [material name, aggregate type, RAP mix percentage, RAP asphalt content]: [Mix A, Limestone Dolomite, 30%, 5.2%], [Mix B, Limestone Dolomite, 30%, 5.5%], [Mix C, Limestone Dolomite, 25%, 3.5%], [Mix D, Limestone Dolomite, 20%, 6.0%]. Figure 4 displays the combined gradations for the materials.
3.2. OAC results

As an example, Figure 5a displays the proposed high-quality base design procedure carried out on one of the test materials with 0% RAS content. The specimens were prepared with SGC at nominal AC of 3%, 4.5% and 6%. The resulting dry and rich OAC base design are 3.7% and 6%, respectively. Since the IDT strengths of all specimens exceeded the design criteria, the OAC were controlled solely by density. This pattern was observed for most of the materials. The rich OAC base design values for almost all cases were 6%. Figure 5b displays the results from the same specimens, but based on the low-quality HMA philosophy. The OAC in that case was 4.7% at a design density of 96% (typical design density used). Following the low-quality HMA philosophy, but compacting with the TGC, yielded an OAC of 5.0%.
Shown in Table 2 are the OAC values obtained with 75 and 100 gyrations following the proposed high-quality base design and low-quality HMA design procedures. The results with 0% and 3% RAS are also included. Figure 6 summarizes the average OAC values from the four materials following each design process. The error bars correspond to the respective minimum and maximum OAC values of the four mixes for each design method. Also shown in the figure is the typical OAC range derived in practice (4+0.3% OAC). The proposed high-quality base design method yields an average dry OAC of 3.7%. The addition of 3% RAS does not seem to affect the average OAC. This OAC is slightly lower than that currently derived in practice from following the HMA design methods. HMA design with a) SGC and with b) TGC produce average OACs of approximately 4.8%. This average OAC is slightly greater than that obtained during practice. This greater OAC is due to the wider AC band evaluated for each data set in this experimental design plan in comparison to that performed during practice. This experimental design plan caps each data set to 6% versus 5%-5.5%, as performed in practice. A maximum AC of 5%-5.5% will force a lower OAC value due to the linear fit at 96% target molded density. For common practice, it is recommended for the OAC design sets to be evaluated at a wide AC band (up to 6%) in order to attain truly representative design OAC values. The addition of RAS does not seem to influence the average OAC when following the HMA design method. Likewise, the use of SGC vs. TGC also does not significantly affect the OAC.
Table 2. Obtained OAC values at various test methods, gyrations and compactors

<table>
<thead>
<tr>
<th>Gyrations</th>
<th>Design Process</th>
<th>RAS</th>
<th>OAC Mix A</th>
<th>OAC Mix B</th>
<th>OAC Mix C</th>
<th>OAC Mix D</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>Base design (SGC)</td>
<td>0%</td>
<td>3.7%</td>
<td>3.2%</td>
<td>3.1%</td>
<td>4.6%</td>
</tr>
<tr>
<td></td>
<td>HMA design (SGC)</td>
<td>3%</td>
<td>4.7%</td>
<td>5.0%</td>
<td>4.1%</td>
<td>4.9%</td>
</tr>
<tr>
<td></td>
<td>Base design (SGC)</td>
<td>0%</td>
<td>3.7%</td>
<td>3.0%</td>
<td>3.4%</td>
<td>4.4%</td>
</tr>
<tr>
<td></td>
<td>HMA design (SGC)</td>
<td>3%</td>
<td>5.6%</td>
<td>4.0%</td>
<td>4.9%</td>
<td>5.3%</td>
</tr>
<tr>
<td>100</td>
<td>Base design (SGC)</td>
<td>0%</td>
<td>3.3%</td>
<td>3.0%</td>
<td>4.3%</td>
<td>4.7%</td>
</tr>
<tr>
<td></td>
<td>HMA design (SGC)</td>
<td>3%</td>
<td>4.9%</td>
<td>4.6%</td>
<td>4.4%</td>
<td>4.8%</td>
</tr>
<tr>
<td></td>
<td>Base design (SGC)</td>
<td>0%</td>
<td>3.4%</td>
<td>3.7%</td>
<td>3.8%</td>
<td>4.3%</td>
</tr>
<tr>
<td></td>
<td>HMA design (SGC)</td>
<td>3%</td>
<td>5.4%</td>
<td>4.2%</td>
<td>4.8%</td>
<td>4.9%</td>
</tr>
<tr>
<td>psi values (TGC)</td>
<td>HMA design (TGC)</td>
<td>0%</td>
<td>5.0%</td>
<td>4.8%</td>
<td>5.0%</td>
<td>5.1%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3%</td>
<td>5.0%</td>
<td>4.8%</td>
<td>4.9%</td>
<td>5.6%</td>
</tr>
</tbody>
</table>

Figure 6. Global OAC analysis of tested materials

Figure 7 displays the OAC values for a tested material following each design method to identify any sensitivity to the number of gyrations. For this and the other materials evaluated, the OACs are not too sensitive to the number of applied gyrations, considering that the gyrations are within the general vicinity of the locking points.
Overall, the OAC is more sensitive to the design method followed (high-quality base design vs. low-quality HMA design). The proposed high-quality base design method will yield a slightly dryer OAC than HMA design procedure.

4. IMPACT OF RAS ON ASB STRENGTH

The impact of RAS on IDT strength was assessed by comparing the results from specimens with 0% RAS content to those with 3% RAS. As an example, Figure 8a and Figure 8b display the absolute and relative increase in strength by adding 3% RAS to one of the tested materials. The IDT strength increases with the addition of 3% RAS.

A global analysis of 3% RAS influence on strength at varying asphalt contents is displayed in Figure 9. For each asphalt content, the average, minimum, and maximum strength ratios of 3% RAS to 0% RAS are presented. The strength ratios are almost always equal to or greater than unity; illustrating an increase in IDT strength by adding 3% RAS to the mix designs. The increase in strength is shown to mostly range between 10% and 25%.
5. CONCLUSION

An evaluation of ASB design practice was performed to recommend a procedure that would yield moderate to dry OAC design. A proposed high-quality base design procedure was compared to that of a low-quality HMA to assess the impacts on the obtained OAC. The influence of the type of compactor (SGC vs. TGC), number of gyrations, and the presence of RAS was also assessed. The evaluation was performed on 4 commonly used mixtures in Texas. The results show that high-quality base design process to yield OAC values that are slightly dryer when compared to OACs obtained from HMA design method. In essence, the contractor can choose to follow either design procedure (proposed high-quality base design or HMA design) to obtain an acceptable ASB OAC. For common practice, it is recommended for the OAC design sets to be evaluated at a wide AC band (up to 6%) in order to attain representative design OAC values. The presence of RAS in the mix, the number gyrations (near the locking point), or the type of compactor had no significant influence on the derived design OAC. However, the presence of 3% RAS in the mix designs was shown to increase the IDT strength of the mix by 10% to 25%.

6. ACKNOWLEDGEMENT

The authors would like to thank the Texas Department of Transportation for their guidance and support to perform this study. The authors would like to acknowledge engineering personnel from the TxDOT Construction and Laboratory divisions for their assistance in acquiring raw material for this study. The authors would also like to thank the undergraduate research assistants that performed the laboratory testing.

REFERENCES


Permanent and Resilient Deformation Behavior of Geogrid-Stabilized and Unstabilized Pavement Bases

Prajwol Tamrakar, Mark H. Wayne
Tensar International Corporation

David J. White
Ingios Geotechnics

ABSTRACT

Permanent and resilient deformation responses of unbound aggregate base course and subgrade impact pavement system performance. Several field and laboratory-based testing methodologies are available to measure both permanent and resilient deformations. Laboratory testing typically utilizes a small cylindrical specimen reconstituted from a single material and tested with confining pressures and repeated axial stresses to simulate an idealized field condition. Although convenient, laboratory testing cannot truly represent the field conditions (layers of different materials with variable stress and boundary conditions). In situ, light weight deflectometer (LWD) and falling weight deflectometer (FWD) tests are often used to measure a load-deformation response from a falling weight. Since impact loads are used for these tests, the measured deformations are the peak deformation (not permanent or resilient deformation). When static loading is needed, plate load testing (PLT) is used for measuring permanent deformation of incrementally applied loads. Alternatively, Automated Plate Load Testing (APLT) can be used to apply cyclic (repeated) load-deformation responses using controlled load-time pulses to simulate traffic and measure permanent and resilient deformations. The aim of this paper is to present results of APLT to determine permanent and resilient deformations of geogrid-stabilized and unstabilized pavement bases. Field tests were performed over aggregate base course (ABC) at two sites prior to paving. Both ABCs were prepared using recycled materials. Permanent and resilient deformations were measured at constant and varying cyclic stress conditions using a 300 mm diameter plate. Deformations were found to be sensitive to both the number of loading cycles and the stress level. For the sections presented herein, the unstabilized section experienced higher permanent deformation than the geogrid-stabilized section while the resilient deformation for both sections was similar.

1. INTRODUCTION

The demand for structural design and analysis of rigid and flexible pavements using the Mechanistic-Empirical (ME) approach has significantly increased in the last decade because of its advanced techniques of modeling and predicting performance of pavement system (Darter et al. 2005; Li et al. 2009; Tompkins et al. 2015; Wu et al. 2016). In 2002, the American Association of State Highway Transportation Officials (AASHTO) introduced the Mechanistic-Empirical Design Guide (MEPDG) to replace the 1993 AASHTO Guide for Design of Pavement Structures (Li et al. 2011). Some State DOTs
also followed the path of AASHTO, and developed specifications and design packages based on the ME approach (e.g., Washington State DOT Pavement Guide, MnPAVE by Minnesota DOT, and CalME by California DOT). Pavement performances in the ME approach are predicted by estimating pavement layer deformations due to traffic load intensity, loading rate, mechanical properties of pavement layer, moisture level and foundation conditions.

Due to cyclic traffic loading, unbound pavement layers experience both permanent and resilient deformations (Tamrakar and Nazarian 2018). The rutting performance of the pavement is directly affected by the permanent deformation behavior. The resilient deformation of unbound pavement layers is responsible for maintaining the flexibility and stability of the pavement structure (Lekarp et al. 2000). Several field and laboratory-based testing methodologies are available to measure both permanent and resilient deformations (Tamrakar and Nazarian 2017). The laboratory testing, for example, AASTHO T 307, utilizes a small cylindrical specimen (150 mm in diameter and 300 mm in height) with confining pressures and repeated axial stresses to mimic a range of field conditions. Due to the lack of all pavement layers, such laboratory testing cannot truly represent the field conditions. On the other hand, the light weight deflectometer (LWD) and falling weight deflectometer (FWD) tests are conducted on the unpaved and paved pavement surfaces to measure the load-deformation responses. Since impact loads are used during the LWD and FWD testing, the measured deformations are neither permanent nor resilient. As an alternative, static plate load testing (PLT) is also used in practice. However, such PLT can only measure permanent deformation due to a static load. Dynamic (repeated) load-deformation responses need to be recorded to better characterize traffic load-related behavior of a field constructed aggregate base course. In other words, a PLT with dynamic or repetitive loading conditions over a range of different stress levels is required to mimic field behavior. In response to this need an Automated Plate Load Testing (APLT) system has been developed with the capability to measure in-situ dynamic load-deformation responses through the application of various stress levels and loading cycles (White and Vennapusa 2017).

The aim of this paper is to evaluate permanent and resilient deformations of geogrid-stabilized and unstabilized pavement bases using the APLT system. Field tests were performed over the pavement aggregate base course (ABC) at two sites. Both ABCs were prepared with recycled materials. Permanent and resilient deformations were measured at constant and varying cyclic stress conditions using a 300 mm diameter plate.

2. BASE COURSE STABILIZATION

Base course stabilization is the process of constructing a mechanically stabilized layer (MSL) with the use of geogrid. The term “stabilization” is different from “reinforcement” because the former one is referring to “stiffness enhancement” as well as “stiffness retention for a longer period”. In contrast, the term “reinforcement” implies “adding force” (Giroud and Han 2016). This mechanism is only effective if the forces are large which in turn implies that large vertical permanent deformations exist in the aggregate overlying the geogrid.
When a geogrid is incorporated into a granular material, the aggregate particles interlock with the geogrid and are confined within the geogrid apertures. As a result, the particles are restrained from moving laterally. The lateral restraint provided by the geogrid contributes to reducing induced strain due to traffic loading and thereby increases the stiffness of the granular layer (Wayne et al. 2013; Sun et al. 2018; Wayne et al. 2019). In addition to the lateral restraint, granular particles immediately adjacent to the interlocked particles are themselves restrained by particle to particle interlock. Thus the influence of the geogrid inclusion extends beyond the geogrid particle interface. Cook and Horvat (2014) demonstrated the existence of such variation in particle interlocking using a multi-level shear box. The authors found that the influence of the geogrid on the stiffness of a granular layer will decrease in relation to the distance from the geogrid. This can be represented as zones of confinement from fully confined to unconfined as illustrated in Figure 1. Behaviors of confinement due to different geogrid and material types were also investigated through the use of Discrete or Finite Element Modeling (Konietzky et al. 2004; McDowell et al. 2006; Stahl et al. 2014; Jas et al. 2015; Lees 2017).

3. PROJECT DETAILS

The project sites were located in Los Angeles County and Tulare County, CA. The first project site (Site I) was located at the south-bound lane of Interstate 5. The second site (Site II) was located at Avenue 144. For Site I, the stabilized pavement section consisted of 125-mm-thick ABC with a layer of geogrid at the base/subgrade interface. The properties of multi-axial geogrids are reported in Table 1. The unstabilized pavement section consisted of 275-mm-thick ABC. The ABC was prepared from recycled cement concrete material having maximum particle size of 25 mm. For Site II, the geogrid stabilized pavement section consisted of 225-mm-thick ABC with a layer of geogrid located near base/subgrade interface (see Figure 2). The ABC was prepared from recycled asphalt concrete material having a maximum particle size of 50 mm. Both sites consisted of firm subgrade (subgrade CBR >20%). Other details of ABC, including particle size distribution, are reported in Figure 3 and Table 2.
Table 1. Summary of Geogrid Properties

<table>
<thead>
<tr>
<th>Parameters</th>
<th>TX1</th>
<th>TX2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rib shape</td>
<td>Rectangular</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Aperture shape</td>
<td>Triangular</td>
<td>Triangular</td>
</tr>
<tr>
<td>Rib pitch (mm)</td>
<td>33</td>
<td>40</td>
</tr>
</tbody>
</table>

![Diagram of Pavement Sections (a) Site I and (b) Site II)](image)

Figure 2. Pavement Section of Site I and Site II

![Graph of ABC Gradation](image)

Figure 3. ABC Gradation

4. TESTING PROTOCOL

The APLT system (see Figure 4) consists of an advanced electronic-hydraulic control system for applying cyclic and static load pulses through circular steel plates and high-resolution sensors for measuring
vertical ground displacements (White and Vennapusa 2017). Compared to FWD, the APLT has the advantage of applying a conditioning loading prior to testing and measuring peak, resilient and permanent deformations for each loading cycle. In this study, a 300 mm diameter plate was used to apply cyclic loads over ABC of geogrid-stabilized and unstabilized pavement sections. Each load cycle consisted of 0.2 second and 0.8 second of loading and resting periods. Tests were conducted at constant and varying stress conditions. For a constant stress condition, a total of 10,000 cyclic loads was applied and each load cycle consisted of a peak stress of 103 kPa. For the varying stress condition, 100 cyclic loads were applied at six different stress conditions as shown in Table 3.

Table 2. Pavement Section Details

<table>
<thead>
<tr>
<th>Project Site</th>
<th>Site I</th>
<th>Site II</th>
</tr>
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<tbody>
<tr>
<td>Location</td>
<td>Los Angeles County</td>
<td>Tulare County</td>
</tr>
<tr>
<td>Aggregate base course (ABC) thickness (mm)</td>
<td>Unstabilized 275</td>
<td>250</td>
</tr>
<tr>
<td></td>
<td>Stabilized 125</td>
<td>225</td>
</tr>
<tr>
<td>Location of geogrid</td>
<td>At the interface of ABC and subgrade</td>
<td>25 mm above the interface of ABC and subgrade</td>
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</table>

<table>
<thead>
<tr>
<th>ABC Properties</th>
<th>Material Type</th>
<th>Recycled cement concrete</th>
<th>Recycled asphalt concrete</th>
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<tr>
<td>Particle size distribution (mm)</td>
<td>D10 0.07</td>
<td>0.25</td>
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<td></td>
<td>D30 0.57</td>
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<td></td>
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<tr>
<td></td>
<td>D50 2.20</td>
<td>2.04</td>
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</tr>
<tr>
<td></td>
<td>D60 4.17</td>
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<tr>
<td></td>
<td>D85 11.28</td>
<td>13.21</td>
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</tr>
<tr>
<td>Maximum size (mm)</td>
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<td>50</td>
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<td>Soil classification</td>
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<td>USCS</td>
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<td>SP</td>
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<tr>
<td>Fines content (particle size less 75 µm)</td>
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<tr>
<td>Maximum Dry Density (kg/m³)</td>
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<td>Optimum Moisture Content (%)</td>
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<td></td>
<td>Stabilized 30</td>
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*Data not available

Table 3. APLT Test Details

<table>
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<tr>
<th>Stress Condition</th>
<th>Cyclic Stress, kPa</th>
<th>Numbers of Cycle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant Stress</td>
<td>103</td>
<td>10,000</td>
</tr>
<tr>
<td>Varying Stress*</td>
<td>34</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>69</td>
<td></td>
</tr>
<tr>
<td></td>
<td>103</td>
<td></td>
</tr>
<tr>
<td></td>
<td>138</td>
<td></td>
</tr>
<tr>
<td></td>
<td>207</td>
<td></td>
</tr>
<tr>
<td></td>
<td>276</td>
<td></td>
</tr>
</tbody>
</table>

*500 cyclic loads with a peak stress of 103 kPa were applied prior to the test
Cumulative permanent deformation ($\delta_p$) measured using the constant stress condition can be represented by a power model as shown in equation 1. Monismith et al. (1975) described a similar power model relationship for relating permanent strain to cyclic loading for repeated triaxial laboratory testing.

$$\delta_p = CN^d (1)$$

where, coefficient $C$ is the plastic deformation after the first cycle of repeated loading, and $d$ is the scaling exponent.

This power model has been used to forecast the number of cycles required to achieve a selected permanent deformation. Further, the rate change of the permanent deformation can be used to estimate the post-compaction permanent deformation and the corresponding number of loading cycles. Post-compaction permanent strain is a function of the shear stress magnitude and can reach an equilibrium state following the shakedown concept (Dawson and Wellner 1999). In this study, a change in permanent deformation rate ($\Delta\delta_p$/cycle) of 0.0025 mm/cycle or less was selected to represent the near-linear elastic condition. The number of cycles corresponding to $\Delta\delta_p$ of 0.0025 mm/cycle is referred to as $N^*$, where the application of additional cyclic loadings results in very low accumulation of additional permanent deformation and the composite foundation layers are producing a resilient response.

Figure 4. Automated Plate Load Testing (APLT) System

a) APLT Unit

b) APLT Loading Plate
5. PRESENTATION OF RESULTS

Figure 5 presents the results of APLT conducted at the constant stress condition for the stabilized and unstabilized pavement sections of Site I. The solid lines in the figure represent permanent deformation (PD) experienced by the pavement sections at different loading cycles whereas the dotted lines represent the resilient deformation (RD). For both sections, the permanent deformation increased with increasing load cycles. The unstabilized section experienced approximately 4 times higher deformation than that for the stabilized section. A summary of permanent deformation test results is presented in Table 4. The table shows that N* values for the stabilized and unstabilized section are about 1.6k and 12.5k cycles, respectively. This fact indicates that the geogrid-stabilized pavement section exhibits a full resilient response at the early stages of loading. Ideally this early stage loading is completed during the construction process and prior to placement of the overlying concrete or asphalt.

![Figure 5. Permanent Deformation (PD) and Resilient Deformation (RD) Measured at Constant Stress Condition for Site I](image)

Although the results showed a difference in the permanent deformation for the unstabilized and stabilized sections, the resilient deformations were similar except for the initial loading cycles (<2000 loading cycles). Overall, the resilient deformation for both sections was similar despite of thicker unstabilized ABC and thinner stabilized ABC.

Table 4. Summary of permanent deformation test results

<table>
<thead>
<tr>
<th>Site</th>
<th>Pavement</th>
<th>C</th>
<th>d</th>
<th>R²</th>
<th>N*</th>
<th>δ_p (mm) at N*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site I</td>
<td>Stabilized</td>
<td>0.521</td>
<td>0.052</td>
<td>0.92</td>
<td>1,569</td>
<td>0.765</td>
</tr>
<tr>
<td>Site I</td>
<td>Unstabilized</td>
<td>1.529</td>
<td>0.089</td>
<td>0.89</td>
<td>12,520</td>
<td>3.556</td>
</tr>
<tr>
<td>Site II</td>
<td>Stabilized</td>
<td>0.167</td>
<td>0.071</td>
<td>0.96</td>
<td>760</td>
<td>0.269</td>
</tr>
<tr>
<td>Site II</td>
<td>Unstabilized</td>
<td>0.668</td>
<td>0.055</td>
<td>0.91</td>
<td>2,228</td>
<td>1.023</td>
</tr>
</tbody>
</table>

Figure 6 shows permanent and resilient deformations experienced by the pavement sections during the test conducted at the constant stress condition for Site II. Like Site I, the stabilized section also showed resistance against permanent deformation. However, the magnitude of deformations was less than those for the Site I. For Site II, the unstabilized section experienced approximately 4.4 times more
deformation than the stabilized section. Overall resilient deformation for both sections was similar. Table 4 shows that $N^*$ values for the stabilized and unstabilized section are about 760 and 2228 cycles, respectively.

![Figure 6. PD and RD Measured at Constant Stress Condition for Site II](image)

Figure 6. PD and RD Measured at Constant Stress Condition for Site II

Figure 7 presents the results of APLT conducted at the varying stress condition for Site I. Permanent deformation increased with both loading cycles and the stress level. Figure 8 shows the difference in permanent deformation between the stabilized and unstabilized sections at different stress levels. Up to a stress level of 138 kPa, the average difference in permanent deformation was 1.48 mm. Differences in permanent deformation at a stress level of 207 kPa and 276 kPa were 1.67 mm and 1.90 mm, respectively. Compared to the average deformation of 1.48 mm (i.e., at the stress level ≤ 138 kPa), the deformation at 207 kPa and 276 kPa increased by 13% and 28%. This fact indicates that the geogrids contribute in reducing permanent deformation at the higher stress levels.

![Figure 7. PD and RD Measured at Varying Stress Condition for Site I](image)

Figure 7. PD and RD Measured at Varying Stress Condition for Site I

Figure 7 shows the resilient deformation for the stabilized section is slightly lower than that for the unstabilized section of Site I. The difference in resilient deformation between the stabilized and unstabilized section is also reported in Figure 8. The pattern of resilient deformation difference was also
like the pattern of permanent deformation difference. However, the magnitude of deformation difference was considerably low, indicating that the thinner stabilized ABC and thinner unstabilized ABC possess similar resilient characteristics at different stress levels.

![Figure 8. Difference in PD and RD at Different Stress Levels for Site I](image)

For Site II, the APLT testing at varying stress states was conducted only for the stabilized section. Therefore, it was not possible to compare deformation differences at different stress levels for the stabilized and unstabilized sections. Instead, the permanent and resilient deformations measured at varying stress states for Site I and Site II are compared in Figure 9. Both permanent and resilient deformations for Site I are slightly higher than that for Site II, and both sites show a similar pattern of increase in deformation with loading cycles and stress levels.

The average difference in permanent deformation up to the stress level of 138 kPa was 0.49 mm. The deformation difference above the stress level 138 kPa was 0.63 mm. Similarly, the average resilient deformation difference at all stress levels was 0.13 mm. Although the stiffness of subgrade for Site I is greater than Site II, Site II exhibited better performance over Site I.

Several factors such as aggregate material type, aggregate gradation, ABC layer thickness, and geogrid type might have contributed to such difference in performance.
6. SUMMARY AND CONCLUSION

To evaluate permanent and resilient deformation behavior of geogrid-stabilized and unstabilized pavement sections, a series of field tests was conducted using the Automated Plate Load Testing (APLT) system. The pavement aggregate bases were prepared from either recycled cement concrete or recycled asphalt concrete. Two types of multi-axial geogrids were considered for the study. Permanent and resilient deformations were measured at constant and varying cyclic stress conditions using a 300 mm diameter plate. Based on this research, the following conclusions can be drawn:

- Permanent and resilient deformations are directly affected by the number of loading cycles and the stress levels.
- On performing APLT at the constant stress condition, the unstabilized section experiences approximately 4 times greater deformation than the geogrid-stabilized section.
- When the aggregate base course is prepared from the recycled cement concrete, the thicker unstabilized section and thinner stabilized section exhibit similar resilient deformations.
- The difference in permanent deformation between the stabilized and unstabilized sections increases with the increase in the stress level, indicating that the geogrids contribute in reducing permanent deformation at the higher stress levels.
- Geogrids improve performance of aggregate base course prepared with recycled materials.
- The number of cycles corresponding to $\Delta \delta_p$ of 0.0025 mm/cycle is referred to as $N^*$, where the application of addition cyclic loading results in very low accumulation of additional permanent deformation and the composite pavement foundation produces a resilient response. The $N^*$ cycles were 87.5% and 66% less for the stabilized versus unstabilized sections for Site I and Site II respectively. This fact indicates that the geogrid-stabilized pavement sections exhibit a full resilient response at the early stages of loading. Future research should examine whether a correlation exists between pavement performance and low $N^*$ values.
REFERENCES


Design and Construction of Bases and Subbases for Concrete Pavement Performance

Shreenath Rao, Hesham Abdualla

Thomas Yu, P.E.
*Federal Highway Administration*

**ABSTRACT**

Effective base and subbase (referred to in this document as foundation) design and construction is necessary to ensure good, long-term performance of pavements. However, structural models used in mechanistic-empirical (ME) pavement design do not show significant impact of foundation layers on pavement performance, especially for unbound layers beneath concrete pavements. This is in contrast with experience of pavement engineers, who generally agree that the foundation layers perform important functions, and thus contribute significantly to performance. The principal weakness of ME pavement design models is that they are based only on pavement response, assuming the support conditions are even and remain consistent throughout the life of the pavement. Seasonal changes in the foundation layers are modeled, but only as uniform changes in layer moduli values. Such changes have negligible effect on pavement response, especially for concrete pavements. Pavement foundations can and do degrade unevenly over time, especially if the design is inadequate to resist deformation and deterioration over time. The benefits provided by quality foundation layers are through the prevention of adverse incidents that can lead to localized or progressive failures. The relatively simple, idealized structural models used in ME designs are not meant to consider complex mechanisms involved in failures resulting from foundation problems. Thus, qualitative understanding of the failure mechanism is important to devise effective pavement layer designs that avoid foundation problems. Ideally, an effective foundation should not degrade over time and should be designed against deformations, decompaction, and infiltration of fines into the pavement layers, such that the engineering properties of the foundation assumed in design are spatially consistent and retained throughout the life of the pavement. This paper will demonstrate the effective uses of foundations for concrete pavements based on the evaluation of a series of case studies encompassing a range of conditions wherein the foundation layers contributed significantly (either positively or negatively) to overall pavement performance that is not fully accounted for in ME design. Four of the ten case studies (Ontario, Michigan, Missouri, and Virginia) developed under this study and focusing on drainage aspects of pavements are included in this paper. The case studies are used in this study to develop guidelines for proper design and construction of quality bases and subbases for concrete pavements.

1. **BACKGROUND**

Base course is a layer of the pavement structure immediately beneath the surface course. It typically consists of high quality aggregate such as crushed gravel, crushed stone, or sand that provides a uniform
foundation support and an adequate working platform for construction equipment. Base may consist of unbound materials, such as gravel or crushed stone, or stabilized materials, such as asphalt-, cement- or lime-treated materials. The subbase course is typically a granular borrow that is placed between the base and subgrade. It can be constructed as either a treated or untreated layer. Untreated or unbound aggregate subbase layers are characterized in a manner similar to the subgrade in pavement design. The material quality requirements of strength, plasticity, and gradation for subbase are not as strict as for a base. The subbase course must be better quality than the soil subgrade, the subbase is often omitted if soil subgrades are of high quality. Depending on site conditions, subgrade improvements may also be performed. However, the role of different base and subbase layers and rationale for using different base types and layering are not well documented. Many agencies specify typical base and subbase layers based on historical performance and their own experiences. For example, it is unclear where and why a treated base should be used, or why one type of treated base is preferred over another.

Pavement engineers generally agree that foundation layers perform important functions, including providing uniform support, controlling pumping and erosion, protecting against frost heaving, and reducing moisture related damage to paving materials. Based solely on structural analysis, the benefits of unbound aggregate base cannot be demonstrated as the structural models used in mechanistic-empirical (ME) pavement design do not show significant impact of foundation layers on pavement performance. From structural perspective, the most efficient means of providing adequate structure is by providing sufficient surface thickness, concrete or asphalt. However, experience shows that concrete pavements placed directly on subgrade do not perform well in most areas, because of pumping and migration of fines resulting in variability in foundation support. So, it should be clear that the foundation layers perform a different function than the surface layer, which is the main structural component to resist the applied loads. However, when evaluating the benefits of foundation layers, attempts are often made to quantify benefits only in terms of structural effect.

Ultimately, the benefits provided by the foundation layers can be related to the structural and functional performance; however, the benefits are more in the way of preventing bad things from happening that can lead to localized or progressive failures and increased roughness. The relatively simple, idealized structural models used in ME designs are not meant to consider complex mechanisms involved in failures resulting from subgrade and foundation problems. This is not to say that the current ME design models are deficient; it would not be practical nor necessary for design procedures to model complex failure mechanisms associated with foundation problems. For the purposes of pavement design, qualitative understanding of the failure mechanism and protecting against those failures is sufficient to devise effective pavement layer designs that avoid foundation problems. A pavement foundation that does not degrade over time does not need to be replaced. A permanent foundation has obvious advantages in environmental impact, and it could also have significant economic advantages. In congested areas, not having to replace the foundation could be highly advantageous in expediting pavement rehabilitations and reconstructions.
2. FUNCTION OF PAVEMENT FOUNDATION IN RIGID PAVEMENTS

Pavement foundation in rigid pavements has less appreciable impact on the structural capacity and the primary function of the foundation layers is providing a uniform support for the concrete slabs. A uniform and sound quality support layer enhances the rigid pavement performance more than a stronger and non-uniform support (ACPA 2007, Hein et al. 2017). The functions of the rigid pavement foundation are as follows:

- Provide a uniform support to the PCC layer with adequate stiffness.
- Offer a stable construction platform.
- Prevent loss of slab support due to erosion and pumping.
- Provide resistance against frost-heave and soil expansion.
- Separate the subgrade from the main structural component.
- Improve drainage and prevent moisture-related damage.
- Provide a gradual vertical transition in layer moduli (stiffness) from the slab to the subgrade.

If the primary functions of the rigid pavement foundation are not adequately considered during the design process or it is not properly constructed, the pavement system may not achieve the desired performance. Moreover, misuse of a foundation layer beneath a rigid pavement may lead to premature deficiencies. For instance, the base and subbase type and thickness should be selected based on specific site conditions. More often than not, the base and subbase type is selected based on a variety of factors such as the agency policy, cost and availability of materials, and past experience. Given these conditions, the base type and thickness should still be selected to meet the needs (e.g., drainage, protect against frost heaving, protect against swelling and unstable soils) of the project site.

3. QUALITATIVE DESCRIPTION OF PAVEMENT FOUNDATION

3.1 How Rigid Pavements Work

PCC slabs have elastic modulus that is an order of magnitude higher than asphalt concrete. The typical flexural strength is about 700 psi and the modulus of elasticity is about 5 million psi. Therefore, unlike flexible pavement structures, which transfers the wheel load gradually to the layers underneath (see Figure 1), the traffic load applied to rigid pavement structures is primarily distributed by the concrete slabs over a wider area before it is transmitted to the layers beneath the slabs (Hein et al. 2017). As such, the pavement responses induced in the layer below the concrete slabs, including the stresses (i.e., pressure) and strains as well as the deflections, are relatively smaller.

Previous studies showed that the load-induced compressive stress on top of subgrade in rigid pavements is substantially lower than its bearing strength. As an example, 12,000 lb tire load with 100 psi contact stress applied on a typical rigid pavement structure induces compressive stress of about 7 psi for the corner loading. In this case, the induced stress on top of subgrade drops to as low as 3 psi for interior loading. Such observations confirm that concrete pavements obtain the desired structural capacity from concrete slabs and therefore, the uniformity and stability of support layers in rigid pavements is a more important than their stiffness and strength (ACPA 2007).
3.2 Failure Mechanisms

The purpose of a uniform support for concrete pavement is to ensure that pavement will attain its service life and uniformly distribute loads over the foundation throughout the entire service life. A uniform support can be achieved by reducing the effect of three key factors: frost heave, pumping of fine-grained soils, and volume changes of the soil. Other factors responsible for non-uniform support include variability in compaction, in cut/fill and transitions, and ineffective drainage system. Table 1 summarizes the causes and effects of non-uniform support on the performance of concrete pavement and the recommended practicable solutions to eliminate such problem (Hein et al. 2017, ACPA 2007, ACPA 1995, Christopher et al. 2006, Snethen et al. 1977).

4. DESIGN OF BASES AND SUBBASES FOR CONCRETE PAVEMENT

The selection of base and subbase type for a given a project should be based on (1) the function of base/subbase layer with the pavement structure, (2) improve the short and long-term performance, (3) cost-effective approach, and (4) local experience (Hall et al. 2005). Modulus of subgrade reaction (known as k-value) is typically used to quantify the stiffness (strength) of rigid pavement support. Composite k-value is a representative of pavement foundation stiffness consisting base and subbase. The k-value is determined by plate load test in accordance with AASHTO T122 and ASTM D1196. The stiffness of pavement support may increase by placing subbase and base layer on top of subgrade. However, increasing the support strength (or stiffness) to reduce the PCC thickness, to expedite the construction process, or as a surrogate for improving the durability of the base is not recommended. Increasing k-value within the typical range does not substantially affect the required thickness of concrete slab (ACPA 2007).

Aggregate base and subbase with 15% or more fines (i.e., passing the sieve No. 200) are highly prone to pumping. The use of non-erodible or treated base and subbase materials can control and prevent pumping. The requirements in AASHTO M155 entitled “Standard Specification for Granular Material to Control Pumping under Concrete Pavement” should be followed when the unbound granular materials is to be used (AASHTO 2004). In general, the higher the application of heavy truck traffic, materials with lower fine content and lower plasticity should be selected.
### Table 1. Failure Mechanisms and Recommended Solutions for Obtaining a Uniform Support

| ITEMS                  | CAUSE                               | EFFECT                              | SOLUTION                                                      |
|------------------------|-------------------------------------|-------------------------------------|                                                               |
| Pumping                | • Base erodibility                  | • Faulting                         | • Use less erodible base materials                           |
|                        | • Fine grained subgrade              | • Fatigue                           | • Limit the amount of fines to 10% or less for aggregate base course |
|                        | • Present of free water              | • Corner cracking                   | • Use a well-design drainable system                        |
|                        | • Water table                        | • Lane shoulder drop off            | • Use a separation layer when using unstabilized granular layer |
|                        | • Heavy truck                        | • Loss of support                   | • Use dowel bars                                             |
| Frost heave            | • Frost susceptible soil             | • Random cracking                   | • Use non-frost-susceptible materials                       |
|                        | • Source of water                    | • Blocked drainage                  | • Cover frost-susceptible soil with sufficient thickness of non-frost-susceptible material |
|                        | • Freezing temperature penetrating the soil | • Reduction in bearing capacity |                                                               |
|                        | • Random cracking                    | • Loss of support                   |                                                               |
| Expansive soil         | • Degree of moisture changes within the soil | • Random cracking                  | • Consider use of stabilization and membranes               |
|                        |                                     | • Longitudinal cracks near the pavement’s edge | • Remove and replace small areas of swelling soils |
|                        |                                     | • Significant surface roughness     | • Covering the soils with a sufficient depth               |
|                        |                                     | • Loss of support                   | • Identify optimum moisture content                         |
| Longitudinal or transverse Cut/fill | • Variability of soil along the project | • Increase long-term roughness | • Use quality materials for fill sections to ensure long-term performance |
|                        |                                     | • Settlements, which causes random cracking | • In cut-sections, drainage system may be needed if ground water table is high |
|                        |                                     | • Loss of support                   |                                                               |

Stiffer bases are not necessarily better support under rigid pavements as they fail to conform to the shape of the curled PCC slabs and may lead to loss of support, higher curling stresses, and subsequent cracking. It should be noted that providing thicker concrete slab, higher concrete strength, the use of dowel bars and widened slabs are more economical to substantially reduce the cracking potential in concrete slabs and pumping of materials. A stiff support has potential to cause cracking because of the higher environmentally-induced stresses in the slabs. This can be detrimental for relatively young
concrete slabs leading to development of random cracks. It is recommended that the compressive strength of cement treated bases and lean concrete bases should range from 300 to 800 psi and 750 to 1,200 psi, respectively (Hein et al. 2017).

Stabilized bases including cement treated bases and lean concrete bases have potential to expand and contract due to moisture and temperature variations. These movements can sometimes induce stresses greater than the strength of freshly placed surface PCC (when the strength in the freshly placed PCC is low as it is hydrating and gaining strength), thus increasing the potential for early-age cracking in the PCC layer. In addition, rough slab-base interfaces increases frictional forces at the interface due to the excessive axial restraint to volumetric shrinkage and to thermal expansion and contraction (Hall et al. 2005). To mitigate this potential risk, it is common practice to have a debonding separator layer (like a plastic sheet) between the cementitious stabilized base and the PCC layer. However, an unbonded base contributes less to the long-term fatigue performance of concrete pavement as compared to a fully bonded base, and this may need to be considered in the pavement design process, for example, by increasing the thickness of the PCC layer. This is less of an issue with dense asphalt-treated bases which are sufficiently flexible and do not expand and contract due to thermal effects to the same extent as cementitious stabilized bases.

To provide drainable base layers, permeable granular or stabilized bases with drainage system or free draining delighted bases can be used. Permeable granular layers should only be used where there is potential for moisture damage to pavement on roadways with medium to heavy truck traffic, and should be properly designed and constructed. However, the owner agency should have a commitment to regular inspection and routine maintenance of the edge-drains or the exposed (daylighted) area of the aggregate drainage layer. An open-graded base needs a suitable separator layer beneath it to prevent subgrade fines from migrating up into and clogging the base. This may be an appropriately graded untreated aggregate subbase, an appropriate geotextile fabric, or a layer of subgrade soil treated with sufficient lime or cement to achieve good long-term stability and resist erosion. Stabilized open-graded drainage layers have very little aggregate passing the No. 200 sieve. Asphalt cement contents typically range between 1.6 and 1.8 percent by mass of aggregates. Cement treated open graded drainage layers are typically produced with a water to cement ratio of 0.37 and a cement content of 185 to 220 lbs/yd$^3$ (Hein et al. 2017). Permeable bases must be constructed strong enough to resist construction traffic and paving machine without deformation (Hall et al. 2005). The recommend permeability values are ranging from 500 to 800 ft/day with taking in consideration the stability of the bases (Hein et al. 2017).

The use of “daylighted” base course that is exposed to the open along the edge of the pavement is recommended to drain water infiltrating from the surface into the base layers, particularly in situations where moisture conditions are not extremely severe. Daylighting allows water to slowly drain out of the pavement structure without the use of edge drains. Daylighted bases are well suited for roadways with flat grades (1% or less) and shallow ditches, where it is difficult to outlet drainage pipes at an adequate height above the ditch. However, it requires careful construction and periodic maintenance to keep the exposed edge clear of soil, vegetation, and debris, and prevent clogging. Typical maintenance activities include weeding and manual removal of debris. The bottom of the exposed edge of the daylighted base
should be at least 6 in. above the 10-year storm flow line of the ditch to prevent water from backing up into the daylighted base during or after a heavy rainfall. Daylighting the base layers is more “forgiving” than using edge drains. With edge drains, there is the potential for trapping water within the pavement layers causing a “bathtub” effect and resulting in significantly greater damage, if they get clogged from not being regularly maintained or from improper installation. However, when properly maintained, edge drains are effective and drain water efficiently out of the pavement system, particularly in areas with high water tables and cut sections.

5. CASE STUDIES FOR EFFECTS OF BASES AND SUBBASES ON PAVEMENT PERFORMANCE

5.1 U.S. 460 Bypass, Appomattox County, VA

The project is located on the U.S. 460 bypass in the northern part of Appomattox County, Virginia. The dowelled jointed plain concrete pavement (JPCP) section, an approximate 2.8-mile long section, of the U.S 460 bypass exhibited premature failure at several location approximately 5 years (i.e., 1998) after paving. The Virginia DOT engineers and researchers conducted field and laboratory investigations to identify the causes of premature failures and evaluate the condition of the pavement section (Hossain and Elfino 2005, Elfino and Hossain 2007). The project is located in a wet-freeze climate and the average daily traffic (ADT) in 2003 was 13,000 with 10% truck traffic.

5.1.1 Design and Construction

The U.S. 460 bypass was designed to carry an equivalent of 8 million single axle loads (ESAL) with an expected design life of 30 years. The following design was used for this section:

- 9.0 in. dowelled JPCP slab with 15 foot spacing.
- 4.0 in. cement-stabilized open-graded drainage layer (OGDL).
- 6.0 in. cement-treated soil, using 10% hydraulic cement by volume.
- 9.0 to 6.0 in. variable depth jointed concrete undowelled tied shoulder.
- 4.0 in. aggregate base materials for shoulder (VDOT Type 1, Size 21A).
- Pavement edge drain UD-4 in accordance with the standard pavement edge drainage and outlet pipes.

The subgrade soil was classified as A-7-5 red clay and silt with a CBR of 9.

5.1.2 Performance

A visual survey was conducted to evaluate the cause of premature pavement failure. The survey results showed that about 24% of the eastbound slabs were distressed, compared with 12% of the westbound slabs. The pavement exhibited mid-slab cracks, broken joint seals, lane-shoulder drop-off and pumping, and joint faulting. Field and laboratory investigations were performed to assess the causes of pavement distresses. The overall observation of the laboratory and field investigation can be summarized as follows:

- A majority of the drainage layer was clogged and filled with red soil (see Figure 3a).
- Cracks propagate through the drainage layer in the mid-slab crack core sample.
- Water trapped underneath the slab was observed under damaged slabs during coring.
- The OGDL was not extended over the edge drainage in some areas (see Figure 3b).
5.1.3 Lesson Learned/Summary

- A poor drainage system and increased truck traffic can significantly affect pavement performance.
- If the OGDL is not continued to the edge drain, trapped water in the drainage layer will seep vertically and cause an increase in base/subbase and subgrade moisture.
- Water abrades the soil cement base/subbase under repeated heavy loads leading to localized loss of support, disintegration, resulting in pavement distresses, including structural and durability Figure 4.

5.2 U.S. 63, Callaway County, MO

The Missouri River flooding significantly damaged pavements, culverts, bridges, etc. in Jefferson City, Missouri, resulting in the closure of roadways, delay in traffic, and economic losses to the city. The roadway was completely washed out due to the flooding in 1993. The project is located on southbound US-63 in Callaway County, MO, just across the Missouri River from Jefferson City, MO. The original pavement design of southbound US 63 sections consisted of 9 inches of joint reinforced concrete pavement (JRCP) with 61 foot joint spacing on 4 inches of dense graded crushed rock base. The Missouri Department of Transportation (MoDOT) researcher and engineers conducted a comprehensive study to further enhance the pavement design of US 63 sections to resist such environment. The major finding of the study led to developing a new standard specification provision of a thick daylighted rock base, which
has capabilities to drain water from the pavement structure and to improve the load-bearing capacity of pavement structure.

5.2.1 Design and Construction
The new design of US 63 section consisted of 12 inches of doweled joint plain concrete pavement (JPCP) with 15 foot joint spacing over 24 inches daylighted rock base and was constructed in 1994. This was the first implementation of daylighted rock base in Missouri. The 24 inches base was selected to increase the structural capacity as well as improve drainage during heavy rain or flood periods. The grade was raised about 6 feet because of the flooding damage. The daylighted rock base was placed on the top of the subgrade. A cross slope gradient from the median to the outside fill slope was provided on the top surface of the subgrade to remove water effectively from the pavement structure, prior to the placement of the 24 inch rock fill base. The subgrade soils in this area consisted of A-6 and A-7-6 soils.

5.2.2 Performance
A visual survey of US-63 sections conducted in 2016 and revealed that all sections performed in excellent condition and there were no signs of cracking or faulting (see Figure 5a). In early 2018, a second survey was conducted indicated excellent conditions, which could be attributed to the effectiveness of the daylighted rock base (see Figure 5b). The pavement section was constructed in October of 1994 and has performed extremely well with minimal cracking, faulting, and roughness. Minimal maintenance has been done on the section since construction and all joints look exceptional. The section experienced another flood in 1995 and maintained in good condition. The success of this pavement was attributed to the daylighted 2-foot rock base and its superior drainage capabilities. After 24 years of relatively heavy traffic, US 63 section is still in perfect structural condition and no repairs were performed. IRI measurements were taken from 2007 to 2017 for the project and the data shows the consistency of the roughness of the pavement over 10 years of initial life.

![Figure 5. Pavement Performance of US-63, Callaway County, MO: (a) performance in 2016 and (b) performance in 2018](image)

5.2.3 Lesson learns
- The stability and drainability of base material is essential to enhance the performance of pavement during heavy rain or flooding incidents.
The original base, 4 inches dense graded crushed rock, was filled with sand and relatively undrainable. Concrete pavement constructed over a dense-graded rock base has a high risk of being damaged during flooding incidents due to ineffective base drainability.

The use of a thick daylighted rock base significantly improved the long term pavement performance. The 24 inch daylighted rock base was effective for removing water from the pavement structure, which enhanced the JPCP performance and eliminated the moisture-related damages.

5.3 U.S. 23, Monroe County, MI

In 1992, Michigan DOT constructed an Aggregate Test Road on southbound US-23 with the main purpose of studying the effect of frost susceptible coarse aggregate on concrete durability. The project starts just north of the US-23 and US-223 interchange and ends at the border line between Michigan and Ohio. The test road was constructed with concrete mixtures including five different coarse aggregates (Groups A through E) of varying degrees of freeze-thaw properties. The coarse aggregate type for group A was 6AA crushed limestone, group B was 6AA blast furnace slag, group C was 6A natural gravel, group D was crushed limestone from different quarry, and group E was natural gravel. All other factors of the concrete mix design were kept the same. The average annual daily traffic (AADT) was about 20,000 with 18% of commercial (Hansen et al. 2007, Quiroga 1992).

5.3.1 Design and Construction

The original pavement was removed to the existing sand subbase. The new pavement structure consisted of a 10.5 inches jointed reinforced concrete pavement (JRCP) with 27 ft. joint spacing, on a 4 inches asphalt treated permeable base (ATPB) on a 3 inches gravel separator layer. Half of each of the five test sections was built on the original poorly-draining subbase while the other half constructed on a well-draining permeable sand subbase to evaluate the effect of subbase layer on concrete performance. The existing subbase material exhibited a much finer mix compared to the new subbase which greatly affects the drainage properties of the materials. The existing subbase was considered impermeable, and the new subbase is very drainable. The other half was constructed on a well-draining, special select subbase which showed extremely high drainability values ranging from 198 ft/day to 288 ft/day, well exceeding the specification requirement of 7.7 ft/day. The subgrade soil below the pavement structure consist of a wet clay. During reconstruction, subgrade undercuts were performed at locations with unstable grade, followed by the installation of a 4.0 inch underdrain and backfill.

5.3.2 Performance

The main purpose of the test road was to study the effect of freeze-thaw on pavement performance. All JRCP sections did not exhibit any distresses related to freeze-thaw problems such as joint deterioration or D-cracking. The ATPB was a major factor in preventing D-cracking along with a good air-void system of the concrete. Mid-panel deflections were measured and there were smaller deflections underneath the well-draining subbase compared to the existing poor subbase. Dowel bar looseness was also observed which contributed to higher deflections and poor load transfer. After 23 years, all section performed well with the exception of section B. Section B (i.e., aggregate type was blast furnace slag) exhibited significant full lane width mid-panel cracks in about 75 percent of the truck-lane panels which was followed by crack spalling. Full depth repairs were made after 19 years of service. It was observed
from coring that some minor deterioration of the ATPB at the crack edge occurred in isolated cases which caused some erosion issues in section B. All joints did not exhibit pumping and joint faulting was less than 0.04 inches. Overall, the project performed very well regarding freeze-thaw resistance, durability, drainage, and distresses. The freeze-thaw performance was attributed to the well-draining ATPB layer which prevents water from accumulating at the bottom of the PCC layer.

5.3.3 Lessons learned
- A well-draining base/subbase structure improved pavement performance as well as freeze-thaw resistance.
- Higher mid-panel deflections were observed for the “existing” poor-draining subbase compared to the well-draining subbases.
- The ATPB has the potential to drain water from pavement, prevent pavement becomes saturated, which eliminating the effect of freeze-thaw damage and moisture related distresses.

5.4 Ontario, Canada
The Ontario Ministry of Transportation (MTO) is responsible for the management of 10,300 miles of paved roads and rigid pavements are about 6% of the total. The Ontario MTO conducted several forensic investigations and gathered the information from other highway agencies in North American to develop specification for three types of open graded drainage layers (OGDL). They categorized the OGDL into three types; (1) untreated, (2) asphalt cement treated, and (3) Portland cement treated. Beginning in the early 1980s, the MTO constructed a series of test sections to monitor the performance of the drainage system and pavement. The key design considerations for the OGDL layers include:

- Permeability of the OGDL (ensure water movement away from the travel lanes).
- Stability/strength (to allow proper placement and compaction as well as support for the pavement surface).
- Collector system (ensure water entering the pavement will be moved away from the travel lanes and ensure long-term performance of the system – not clog).
- OGDL protection (ensure the OGDL and drainage system are not clogged by fine aggregate and soil particles reducing the system permeability).

Based on the key findings of the OGDL research investigation, the MTO developed new specifications requiring that a 4 inch layer of OGDL be placed beneath the concrete slab in all new rigid pavement designs (Marks et al. 1992, Hajek et al. 1992, Bradbury and Kazmierowski 1993, and Kazmierowski et al. 1999). The gradation of the OGDL consists of coarse aggregates retained on the No. 4 sieve size. As untreated aggregates were considered not to be stable enough to support construction traffic without distortion, the OGDL is treated with 1.8 percent asphalt cement. In addition, the longitudinal drainage system was modified to be integral with the OGDL to ensure that water entering the system exits the pavement as soon as possible. The OGDL should be extended 3 feet past the edge of the concrete pavement or paved shoulder, if present.

5.4.1 Design and Construction
This section demonstrates the design and construction of highway using three types of OGDL bases. The highway 115 is located near the city of Pererborough. A section of Highway 115 Pererborough with a total length of 10.20 mile was built in 1991 with three different OGDL to evaluate the performance of
each type. Section one (0.6 mile long) consisted of 8 inches JPCP, on 4 inches of untreated OGDL with increase in percent of passing No. 4 to increase layer stability, on 4 inches of aggregate base, over 12 inches of aggregate subbase. Section two was similar to section one, but with 4 inches of cement treated base (200 lb/yd$^3$) instead of untreated OGDL. Section three was similar to section one, but with asphalt cement treated base (1.8 percent) instead of untreated OGDL. The purpose of 4.0 aggregate base was used to act as the filter layer between OGDL and the subgrade. The longitudinal subdrain was placed under the shoulder, 2 feet away from the lane edge. A 4 inches diameter outlets was placed at 330 feet intervals to roadway ditches. The cement treated OGDL was placed with the concrete slipform that was used to place the concrete and there was no issues during the placement reported. Minor damage of the surface of the cement treated OGDL was observed during the placement of concrete pavement. The cement treated OGDL was cured by water “sprinkling” every 2 hours for 8 hours. The asphalt treated OGDL was placed using hot mix asphalt paver with no placement issues. The untreated OGDL was placed using trucks and a grader to achieve the design profile.

5.4.2 Performance
Laboratory testing was conducted to evaluate the permeability of the three OGDL. The results indicated that all three types of OGDL met the initial permeability and stability requirements. The untreated OGDL was able to carry construction traffic without any significant damage. The FWD testing was conducted in 1992-1993 indicted that the deflection of cement treated OGDL was 17 percent less than asphalt treated OGDL and about 28 percent less than untreated OGDL. In general, the performance of the Highway 115 pavement has been excellent. There were several issues related to late season construction that were documented on this contract including; cold weather concrete delivery, subgrade instability in cut to fill transition, and premature cracking due to late saw cutting of transverse joints.

In 2005, a pavement evaluation was undertaking to identify and prioritize concrete pavement restoration requirements for the pavements. The evaluation included a detailed pavement surface condition survey, Falling Weight Deflectometer (FWD) testing, subgrade and pavement layer materials testing, MIT Scan to check dowel bar alignment, and Ground Penetrating Radar (GPR) testing and test pits at the side of the roadway to verify the operation of the drainage system. The results of the pavement investigation completed in 2005 revealed 0.5 percent cracked slabs (2 slabs) in the eastbound direction of the highway and 2.4 percent (50 slabs) in the westbound direction at an age of 13 years. By this time, the pavement had carried approximately 4.67 million equivalent single axle loads (ESALs). These slabs were replaced in a 2006 construction contact. The majority of the slab replacements were at cut to fill transition areas. In addition, the concrete pavement was diamond ground for smoothness/friction in 2011 and then grooved in 2014. As of 2017, the pavement had carried approximately 13.3 million ESALs.

5.4.3 Lessons learned
- Open graded drainage layers and their drainage system should be protected from the intrusion of fines. The movement of fine soil particles such as silty clay by pumping action of repetitive axle loading can lead to early pavement failures.
- OGDL layers should be separated from the subgrade through the use of a granular layer. The use of granular layers was found to be more effective than using a geotextile.
The continuity of the OGDL and subdrain system to remove water from the pavement is critical to prevent water from being trapped within the pavement structure. The OGDL should not be left uncovered for long periods or over the winter. Asphalt treated OGDL can more easily be completed when the layer has been allowed to cool below a temperature of 150°F.

6. SUMMARY AND CONCLUSION

The goal of this paper was to identify and document the useful information related to the effect of the pavement foundation on the performance of concrete pavements. The function of pavement foundation include prevention of pumping, protection against frost action, drainage, prevention of volume change of the subgrade, increased structural capacity, and a stable construction platform. The primary function of base is to prevent pumping, and therefore it must be free draining or highly resistant to erosion.

Various case studies of the effect of pavement foundation on concrete performance were summarized based on data/field investigations. These case studies show the effect of base/subbase in terms of increasing or decreasing the overall pavement performance. The case studies included in this paper documents the substantial effect of drainage on the structural and functional performance of concrete pavements. Poorly designed or constructed drainage systems have a detrimental effect of pavement performance, while removal of water through well-designed and well-constructed drainage system is crucial for long term pavement performance in areas where the potential for moisture damage is high.

7. REFERENCES


Development of a 3/4-Inch Minus Crushed Base Course Specification

Eli Cuelho, TRI Environmental, Inc., Austin, TX, USA

ABSTRACT

Highway base courses are typically constructed using crushed and processed aggregate. Crushed base aggregates are typically a cost effective means of carrying loads and reducing the thickness of the asphalt layer in paved roads. For some projects in Montana, however, obtaining materials that meets the current specifications for crushed base course is becoming uneconomical due to declining resources. Montana specifications currently exist for a 2-inch minus (Grade 5A) and 1½-inch minus (Grade 6A) crushed base course; however, gravel sources in parts of Montana are becoming limited, making the option to use a ¾-inch gravel base desirable. The objective of this project was to develop a standard specification for a new gravel base course with nominal maximum aggregate size of ¾ in. This was accomplished by conducting a review of current ¾-inch minus specifications from around the U.S., using that information to generate a preliminary specification to create ¾-inch minus mixes, testing the material properties of these mixes, and modifying these mixes to determine the effect changes in the gradation primarily had on its strength, stiffness and permeability. Based on the results of multiple statistical evaluations as well as qualitative comparisons, it was concluded that a ¾-inch minus gradation specification will perform at least as well as Montana’s existing CBC-6A materials and better than CBC-5A materials. Gradation limits for a new ¾-inch minus, Montana Grade 7A, crushed base course were developed.

1. INTRODUCTION

This project was initiated to determine the viability of a ¾-inch minus gradation specification for crushed base course materials for the state of Montana to allow gradations with smaller nominal aggregate sizes to be produced for road construction purposes, the results of which are fully documented in a final report to the Montana Department of Transportation by Cuelho (2016). The first step in this investigation was to review U.S. state and federal standard specifications to document existing ¾-inch minus base course specifications. Standard specifications from all 50 states were reviewed to extract gradation specifications for ¾-inch minus base course aggregates used as the compacted structural layer in highway construction. Information from that review helped identify a starting point from which to develop a standard specification for Montana crushed aggregate courses.

Samples of aggregate were collected from eight different gravel pits geographically located throughout Montana. When necessary, portions of these gravel samples were crushed to create ¾-inch minus mixes, and then these gradations were further modified to evaluate the effect that gradation had on their engineering properties.
The primary objective of this project was met by analyzing two specific aspects of the ¾-inch minus gradations: 1) whether aggregates whose maximum particle size was ¾ inch would perform at least as good as Montana’s current 5A (2-inch minus) and/or 6A (1½-inch minus) crushed aggregate base course materials, and 2) what the effect changes in the gradation had on the material properties within the specified limits. The first goal was accomplished by comparing data from the ¾-inch minus materials tested during this project to data from an earlier study conducted on Montana CBC-6A and CBC-5A materials by Mokwa et al. (2007). The second goal was accomplished by qualitatively analyzing the performance data from finer and coarser gradations for each of the eight Montana sources. The results from this analysis were used to suggest a viable ¾-inch minus gradation specification for crushed base course materials for the state of Montana. Other states experiencing shortages in aggregate base course materials having larger sized particles may wish to adopt the results of this study or implement similar studies to evaluate the feasibility of modifying their crushed base course gradation specifications.

2. REVIEW AND COMPARISON OF STATE AND FEDERAL ¾-INCH MINUS BASE COURSE SPECIFICATIONS

Current Montana specifications exist for a 2-inch minus (Grade 5A) and 1½-inch minus (Grade 6A) crushed aggregate course (MDT, 2014 – §701.02.4); however, because gravel sources in Montana are becoming limited, investigating the use of a gravel specification for maximum particle sizes less than ¾ inch is desirable. The first step in this investigation was to review other state’s standard specifications to document whether or not a standard ¾-inch minus base course specification is currently being utilized. Information from these specifications helped identify a starting point from which to develop a standard specification for Montana crushed aggregate courses having smaller maximum aggregate size.

Standard specifications from all 50 states were reviewed to extract gradation specifications for ¾-inch minus base course aggregates used as the compacted structural layer in highway construction. Only a few states had standard specifications for ¾-inch minus base course materials. Based on a qualitative analysis of the specifications and further discussions with personnel from departments of transportation having the most promising specifications, a ¾-inch minus specification from Colorado DOT was selected as the interim ¾-inch minus specification to produce material mixes for this project. The Colorado specification was used extensively for base course aggregate, and was originally thought to be developed using specifications from the Federal Highway Administration (FHWA), which were also reviewed as part of this effort.

3. CHARACTERIZATION OF ¾-INCH MINUS BASE COURSE MATERIALS FROM MONTANA

Samples of aggregate were collected from eight different gravel pits geographically located throughout Montana (Figure 1). Samples were obtained from all five transportation districts in Montana, and are generally representative of aggregates used for road construction purposes within those districts. Five of the eight sources were Montana Type 6A CBC 1½-inch minus aggregates (Sources B, C, F, G and H); and three sources were ¾-inch minus materials (Sources A, D and E). For the 1½-inch minus materials, stones larger than ¾ inch were crushed using a miniature jaw crusher until all of the material passed the ¾-inch sieve. The crushed material was simply added back to the entire mix creating a simulated ¾-inch minus mix, hereafter referred to as the “Prepared” mix. Using the Prepared gradation as a base, a
second mix was created by making it coarser or finer to determine the effect that changes in the gradation had on its engineering properties. This mixture is hereafter referred to as the “Modified” mix. Particle size distributions were compared to the upper and lower gradation limits associated with Colorado’s ¾-inch minus specification, which was identified by the Technical Panel as the target gradation for this effort. Gradation results from the Prepared and Modified mixes are plotted with respect to the Colorado ¾-inch minus specification upper and lower limits, as shown in Figure 2.

Figure 1: Map of material sources in Montana.

Figure 2: Sieve analysis results of a) Prepared and b) Modified gradations with respect to Colorado specification limits.

General properties of the aggregate mixes were determined using several laboratory tests, including: particle size distribution (ASTM D6913, ASTM D1140, and MT-202), fractured face count (ASTM D5821 and MT-217), Modified Proctor density (ASTM D1557 and MT-230), relative density (ASTM D4253 and ASTM D4254), and specific gravity (ASTM D854, ASTM C127, MT-205, and MT-220). Performance properties were evaluated using R-value (ASTM D2844), direct shear (AASHTO T236), and permeability (ASTM D2434 and AASHTO T215) tests. Friction angle, initial stiffness, and ultimate secant stiffness
were determined from direct shear tests. A summary of the properties for all of the gravel mixes is presented in Table 1.

Table 1: Summary of relevant properties for all gravel mixes

<table>
<thead>
<tr>
<th>General Attributes</th>
<th>Source A</th>
<th>Source B</th>
<th>Source C</th>
<th>Source D</th>
<th>Source E</th>
<th>Source F</th>
<th>Source G</th>
<th>Source H</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel (%)</td>
<td>55</td>
<td>64</td>
<td>75</td>
<td>78</td>
<td>88</td>
<td>65</td>
<td>79</td>
<td>61</td>
</tr>
<tr>
<td>Sand (%)</td>
<td>40</td>
<td>31</td>
<td>18</td>
<td>15</td>
<td>11</td>
<td>33</td>
<td>18</td>
<td>34</td>
</tr>
<tr>
<td>Fines (%)</td>
<td>5</td>
<td>5</td>
<td>7</td>
<td>7</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>Fractured Faces (%)</td>
<td>29</td>
<td>31</td>
<td>44</td>
<td>47</td>
<td>35</td>
<td>40</td>
<td>33</td>
<td>35</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.67</td>
<td>2.71</td>
<td>2.57</td>
<td>2.67</td>
<td>2.55</td>
<td>2.66</td>
<td>2.71</td>
<td>2.70</td>
</tr>
<tr>
<td>Initial Stiffness, k @ 10 psi (kip/in)</td>
<td>12.5</td>
<td>14.2</td>
<td>14.4</td>
<td>21.7</td>
<td>16.7</td>
<td>20.1</td>
<td>23.5</td>
<td>20.9</td>
</tr>
<tr>
<td>Secant Stiffness, @ 10 psi (kip/in)</td>
<td>0.95</td>
<td>2.84</td>
<td>3.82</td>
<td>2.22</td>
<td>2.40</td>
<td>1.84</td>
<td>1.88</td>
<td>3.51</td>
</tr>
<tr>
<td>Friction Angle (deg)</td>
<td>35</td>
<td>59</td>
<td>71</td>
<td>58</td>
<td>59</td>
<td>56</td>
<td>60</td>
<td>67</td>
</tr>
<tr>
<td>Permeability (ft/hr)</td>
<td>0.55</td>
<td>0.63</td>
<td>47.78</td>
<td>3.19</td>
<td>52.72</td>
<td>17.15</td>
<td>0.28</td>
<td>47.12</td>
</tr>
</tbody>
</table>

* Percent retained above the #10 sieve
* Percent retained between the #10 and #200 sieves
* Percent passing the #200 sieve
* For particles retained above the #4 sieve
* Based on dry relative density tests
* Based on wet relative density tests
* Average of all permeability tests performed

4. DATA ANALYSIS AND RESULTS

The primary objective of this project was accomplished by analyzing the results of the characterization and performance test results for all the mixes. The first part of the analysis was to determine whether aggregates whose maximum particle size was ¾ inch would perform at least as good as Montana’s current 5A and/or 6A crushed aggregate base course materials. This was accomplished through multiple laboratory tests to characterize the material properties of the ¾-inch minus mixes from around Montana, and comparing that data to the results from laboratory tests conducted on CBC-6A and CBC-5A materials (Mokwa et al., 2007). The second part of the analysis was to determine the effect changes in the gradation had within the specified limits. This was accomplished by qualitatively analyzing the performance data from finer and coarser gradations for each of the eight Montana sources. The results from this analysis were used to suggest a viable ¾-inch minus gradation specification for crushed base course materials for the state of Montana.

4.1 Statistical Analysis

The results of strength, stiffness and permeability tests conducted on the eight ¾-inch minus mixes from Montana were compared to the results from laboratory tests previously conducted on Montana CBC-6A and CBC-5A materials by Mokwa et al. (2007). Statistical analyses of average values based were conducted using a two-sided t-test (for samples having unequal variance) to determine if apparent trends in measured laboratory test results represent true differences between aggregate types. The two-sample t-test is a statistical test used to determine if the averages of the two data sets are
statistically different from one another based on a mathematical evaluation of the data scatter. In cases where the averages are statistically different, a direct comparison of the mean values indicates which value is greater. Otherwise, the means are considered statistically equal.

The output from this analysis is a parameter called a p-value. In this study, the p-value ranges from 0.500 to 1.000 (based on the one-tailed distribution). Although not typically shown this way, the p-values can be used to determine how two averages compare to one another. P-values closer to 0.500 indicate that the means are statistically more similar to one another and p-values closer to 1.000 indicate the means are statistically more different from one another. For the purposes of comparison, and taking into account the relatively variability typical observed in geotechnical test data, a p-value greater than 0.850 was selected to indicate that the two means were statistically different from one another, while p-values between 0.500 and 0.850 indicated that the means were statistically the same.

There are great number of comparisons that are possible to compare the performance of the ¾-inch minus materials to the Montana 6A and 5A materials characterized in Mokwa et al. (2007); therefore, the number of these comparisons was limited to those most important to base course applications – strength, stiffness, and drainage. Material properties determined during this study related to these characteristics are R-value, friction angle, secant stiffness, initial stiffness and permeability. Comparisons were accomplished by grouping the data into meaningful data sets. The first comparisons were centered on data sets grouped by material type. Four different groupings of materials were used to make these comparisons: 6A materials from Mokwa et al. (2007), 5A materials from Mokwa et al. (2007), a combined group of ¾-inch minus Prepared and Modified mixes, and a combined group of 6A and 5A mixes. The combined mean values associated with each of these data sets are listed in Table 2.

<table>
<thead>
<tr>
<th>Performance Parameter</th>
<th>6A</th>
<th>5A</th>
<th>6A&amp;5A</th>
<th>¾-Inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-value</td>
<td>74.5</td>
<td>72.0</td>
<td>73.7</td>
<td>74.1</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>58.9</td>
<td>56.3</td>
<td>58.0</td>
<td>56.1</td>
</tr>
<tr>
<td>$k_v @ 10$ psi</td>
<td>2.71</td>
<td>2.08</td>
<td>2.50</td>
<td>2.41</td>
</tr>
<tr>
<td>$k_i @ 10$ psi</td>
<td>21.23</td>
<td>14.59</td>
<td>19.02</td>
<td>17.39</td>
</tr>
<tr>
<td>Permeability (ft/hr)</td>
<td>11.2</td>
<td>1.5</td>
<td>7.9</td>
<td>11.5</td>
</tr>
</tbody>
</table>

The results of the various comparisons between these data sets are listed in Table 3. In general, it was observed that the ¾-inch minus materials were relatively similar in performance to the 6A materials, with the exception that the initial stiffness was slightly greater in the 6A materials. There were no statistically significant differences in the R-value or friction angle between any of the materials. The ¾-inch minus materials performed better than the 5A materials (similar to the 6A materials). Bold numbers signify values greater than 0.850 indicating a statistically relevant difference between the two means.
Table 3: T-statistic results for general comparisons

<table>
<thead>
<tr>
<th>Performance Parameter</th>
<th>6A to 5A</th>
<th>¾-Inch to 6A</th>
<th>¾-Inch to 5A</th>
<th>¾-Inch to 6A &amp; 5A</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-value</td>
<td>0.742</td>
<td>0.557</td>
<td>0.748</td>
<td>0.577</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>0.792</td>
<td>0.791</td>
<td>0.531</td>
<td>0.738</td>
</tr>
<tr>
<td>$k_u$ @ 10 psi</td>
<td>0.976</td>
<td>0.849</td>
<td>0.876</td>
<td>0.632</td>
</tr>
<tr>
<td>$k_i$ @ 10 psi</td>
<td>0.990</td>
<td>0.986</td>
<td>0.903</td>
<td>0.815</td>
</tr>
<tr>
<td>Permeability (ft/hr)</td>
<td>0.986</td>
<td>0.527</td>
<td>1.000</td>
<td>0.821</td>
</tr>
</tbody>
</table>

An attempt was also made to determine the effect that modifying the gradation within the acceptable limits of the ¾-inch minus specified range had on its material properties. In this case, materials that were finer were compared to materials that were coarser. The percent of gravel was used as a means of separating the data sets into groups. A threshold of 70 percent gravel (defined as materials retained above the #10 sieve) was used to delineate between coarser and finer materials, with materials having greater than 70 percent gravel being assigned to the coarser group and materials having less than 70 percent gravel being assigned to the finer group. Each of the broader groups of data (Montana 6A and 5A materials from Mokwa et al. (2007) and the ¾-inch minus materials) were split into these two broad groupings based on this criteria. The few number of data points prevented further parsing of the data beyond the following four categories: 6A & 5A – coarser (6/5-C), 6A & 5A – finer (6/5-F), ¾-in minus – coarser (3/4-C), and ¾-inch minus – finer (3/4-F). For the purposes of this comparison, 6A and 5A were also combined into a single data set, mainly because there was not enough data to facilitate meaningful statistical comparisons if parsed too small. Two additional data sets were created by combining all of the coarser materials and finer materials, All-C and All-F, respectively. Combined mean values for each category are listed in Table 4. The two-sample t-test described above was used to compare the means from these data sets. The results of the statistical comparisons are summarized in Table 5. Again, bold numbers signify values greater than 0.850 indicating a statistically relevant difference between the two means.

Table 4: Combined mean values for comparisons based on percent of gravel

<table>
<thead>
<tr>
<th>Performance Parameter</th>
<th>6/5-C</th>
<th>6/5-F</th>
<th>3/4-C</th>
<th>3/4-F</th>
<th>All-C</th>
<th>All-F</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-value</td>
<td>74.6</td>
<td>72.5</td>
<td>76.9</td>
<td>71.9</td>
<td>76.0</td>
<td>72.1</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>60.0</td>
<td>55.5</td>
<td>58.0</td>
<td>54.6</td>
<td>58.8</td>
<td>54.9</td>
</tr>
<tr>
<td>$k_u$ @ 10 psi</td>
<td>2.79</td>
<td>2.14</td>
<td>2.43</td>
<td>2.40</td>
<td>2.58</td>
<td>2.32</td>
</tr>
<tr>
<td>$k_i$ @ 10 psi</td>
<td>20.79</td>
<td>16.79</td>
<td>18.54</td>
<td>16.50</td>
<td>19.48</td>
<td>17.73</td>
</tr>
<tr>
<td>Permeability (ft/hr)</td>
<td>13.5</td>
<td>1.0</td>
<td>15.8</td>
<td>8.8</td>
<td>14.9</td>
<td>6.8</td>
</tr>
</tbody>
</table>
Table 5: T-statistic results for comparisons based on percent of gravel

<table>
<thead>
<tr>
<th>Performance Parameter</th>
<th>6/5-C to 3/4-C</th>
<th>6/5-F to 3/4-C</th>
<th>6/5-C to 3/4-F</th>
<th>3/4-C to 3/4-F</th>
<th>All-C to All-F</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-value</td>
<td>0.706</td>
<td>0.769</td>
<td>0.955</td>
<td>0.604</td>
<td>0.997</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>0.879</td>
<td>0.684</td>
<td>0.585</td>
<td>0.739</td>
<td>0.871</td>
</tr>
<tr>
<td>$k_u$ @ 10 psi</td>
<td>0.970</td>
<td>0.820</td>
<td>0.721</td>
<td>0.544</td>
<td>0.812</td>
</tr>
<tr>
<td>$k_i$ @ 10 psi</td>
<td>0.992</td>
<td>0.620</td>
<td>0.995</td>
<td>0.978</td>
<td>0.972</td>
</tr>
<tr>
<td>Permeability (ft/hr)</td>
<td>0.992</td>
<td>0.620</td>
<td>0.995</td>
<td>0.978</td>
<td>0.972</td>
</tr>
</tbody>
</table>

Several statistically relevant differences are apparent. Perhaps most notably, the comparison between the coarser and finer 6A & 5A materials showed that mean friction angle, secant stiffness and initial stiffness, and permeability were all statistically greater in the coarser 6A & 5A materials (referring to far left column in Table 5). Likewise, the R-value, friction angle, and permeability were greater in the coarser materials in general (referring to far right column in Table 5). Other relevant comparisons showed that R-value and permeability were greater in the coarse ¾-inch minus material when compared to the finer 6A & 5A material. Also, the permeability in the finer ¾-inch minus material is greater than the finer 6A & 5A material. Lastly, the performance of the coarser ¾-inch minus material is generally similar to the coarser 6A & 5A material (refer to column second from the left in Table 5).

4.2 Qualitative Analysis

As described earlier above, two gravel mixes were prepared for each of the eight Montana gravel sources by crushing materials greater than ¾-in. and adding them back into the mixture (Prepared mix), then modifying the gradation by removing and/or adding certain sized particles (Modified mix). These manipulations either made the gradations finer or coarser depending on the quantity and size of materials added or removed. While the degree of coarseness or fineness is somewhat arbitrary, for the purposes of this analysis, the area under the gradation curve was used as the means to quantitatively determine how much finer or coarser the Modified mixes were in comparison to the Prepared mix. Gradations with greater area were finer and those with lower areas were coarser. The amount of change in either direction was the most important outcome, and was expressed as a percent change. Differences in the material properties of finer or coarser mixes from each source were compared using the laboratory test data summarized in Table 1. Changes in the individual properties between the Prepared mixes to the Modified mixes were also expressed in terms of percent change. The results of this analysis are summarized in Table 6.
Referring to Table 6, each source was qualitatively evaluated to determine overall the effect was from adjusting the gradation finer or coarser for each source. Overall, making the mixes finer generally caused a decrease in the friction angle, secant strength and permeability. Materials that had a significant increase in fines showed the greatest decrease in permeability. These results indicate that the upper bound of the gradation is most critical. It is therefore recommended that the upper bound of finer materials be decreased to ensure that the permeability of these materials is not negatively affected. The effective diameter (i.e., the diameter of the particle size associated with 10 percent passing, or D_{10}) has been shown to influence the permeability of sands and gravels (Chapuis, 2004); however, the D_{10} of the Source D and Source E materials showed only a modest change between the Prepared and Modified mixes. It was also noticed that the amount passing the #40 sieve in these two mixes greatly decreased, which may have also contributed to the improvement in permeability. Specifically, gradations with greater amounts passing the #40 sieve had significantly lower permeability (i.e., Source A-Prep, Source A-Mod, Source D-Prep, Source E-Prep, and Source H-Mod).

5. MONTANA ¾-INCH MINUS SPECIFICATION

The purpose of this project was to determine whether crushed base course materials that had a maximum particle size of ¾-in. would perform at least as well as current Montana CBC-6A and CBC-5A base course materials and, if so, establish the boundaries of a gradation specification for these materials. The analysis conducted during this project showed that, overall, ¾-inch minus base course materials work at least as well as Montana CBC-6A and better than Montana CBC-5A materials. It also showed that ¾-inch minus materials within the preliminary specified range performed better near the bottom of the range (i.e., coarser materials) than finer materials. For the most part, the performance characteristics were acceptable for the ¾-inch minus materials tested; however, Sources A and H had lower friction angles, stiffnesses, and poor permeability. Sources D-Prep and E-Prep had poor drainage, and Source F-Mod had lower strengths and relatively low permeability. The final gradation bounds of this suggested gradation are shown in Figure 3, as compared to the Montana 6A and 5A materials. The practicality of producing these mixes and their constructability is under qualitative review by the state.
6. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Gravel bases are a critical component of roads, providing drainage, structural support, and load distribution. Montana specifications currently allow 2-inch minus (Grade 5A) and 1½-inch minus (Grade 6A) crushed base course materials (Section 701.02.4); however, gravel sources in Montana are becoming limited, particularly in the eastern regions of the state, making the option to use a ¾-inch gravel base desirable. The objective of the proposed project was to develop a standard specification for a new gravel base course with nominal maximum aggregate size of ¾ in. The first step in this investigation was to review other state and federal standard specifications to document existing ¾-inch minus base course specifications. Information from that review helped identify a starting point from which to develop a standard specification for Montana crushed aggregate courses. A ¾-inch minus specification from the state of Colorado was used as a preliminary specification in order to produce ¾-inch minus mixes with Montana aggregates for testing purposes.

The most important engineering characteristics of any base course aggregate are strength, stiffness, and drainage capacity. Therefore, the second step in this investigation was to test a variety of crushed aggregates from several ¾-inch minus gradations from various sources throughout Montana to determine their general properties and performance characteristics. Material properties were quantified by synthesizing and analyzing results from the following laboratory tests: geotechnical index tests (particle size distribution, fractured face count, modified Proctor density, relative density, and specific gravity), direct shear, R-value, and permeability.

Data from these tests were compared to existing performance data from crushed base course mixes CBC-6A and CBC-5A previously documented by Mokwa et al. (2007). A two-sided t-test was used to determine whether the averages of the two data sets were statistically different from one another based on a mathematical evaluation of the data scatter. When the performance characteristics of the ¾-inch minus materials were compared to those of the 6A and 5A materials, the following conclusions were made.
The ¾-inch minus materials perform similarly to the CBC-6A materials, with the exception that the initial stiffness is slightly greater in the 6A materials.

Similar to the 6A materials, the ¾-inch minus materials performed better than the 5A materials.

There were no statistically significant differences in the R-value or friction angle between any of the materials.

A second analysis was conducted to evaluate the effect that modifying the gradation had on the performance of the ¾-inch minus gravel. A threshold of 70 percent gravel was used to delineate between coarser and finer materials, with materials having greater than 70 percent gravel being coarser and with materials having less than 70 percent gravel being finer. The following general conclusions were made based on statistical comparisons of these data sets.

- The mean friction angle, secant stiffness and initial stiffness, and permeability were all statistically greater in the coarser 6A & 5A materials when compared to the finer 6A & 5A materials.
- The R-value, friction angle, and permeability were greater in the coarser materials in general when compared to the finer materials.
- The R-value and permeability were greater in the coarser ¾-inch minus materials when compared to the finer 6A & 5A materials.
- The permeability in the finer ¾-inch minus materials are greater than the finer 6A & 5A materials.
- The performance of the coarser ¾-inch minus materials are generally similar to the coarser 6A & 5A materials.

A qualitative analysis was also performed to determine the effect that modifying the mixes by making them either finer or coarser had on their material properties. The following conclusions were drawn from this analysis.

- Overall, making the mixes finer caused a decrease in the friction angle, secant strength and permeability.
- Materials that had a significant increase in fines showed the greatest decrease in permeability.
- Specifically, gradations with greater amounts passing the #40 sieve had significantly lower permeability (i.e., Source A-Prep, Source A-Mod, Source D-Prep, Source E-Prep, and Source H-Mod).

Based on the results of this project, a new specification for a ¾-inch base course specification within the state of Montana (Montana CBC-7A) was suggested. The practicality of producing these mixes and their constructability is under qualitative review by the state.

REFERENCES


Determination of Resilient Modulus for Mechanistic-Empirical New Flexible Pavement Design

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University of Nevada - Reno, Pavement Engineering and Science Program, Dept. of Civil and Environmental Engineering

ABSTRACT

Unbound and subgrade materials were sampled from District 1 and various testing were conducted to determine numerous properties and characteristics including; materials classification (AASHTO and USCS), R-value, moisture-density relations, unconfined compressive strength, and resilient modulus. The resilient modulus test was conducted according to AASHTO T 307 procedure. The stress dependent resilient modulus models were developed for the unbound and subgrade materials. In summary, the stress dependent behavior of the resilient modulus for base and borrow materials in NDOT District I was found to fit very well the theta model. Meanwhile, the stress dependent behavior of the resilient modulus for the subgrade materials fitted very well both the universal model and Uzan model. The MEPDG procedure was used to determine the design resilient modulus for the new design projects. It was observed that the design resilient modulus of the subgrade layer is independent of the pavement structure while the design resilient modulus of the borrow and base layers are dependent on the pavement structure. The statistical analyses of the generated data indicated that the design resilient modulus of the subgrade layer for new projects can be estimated based on the R-value or the unconfined compressive strength properties. However, the design resilient modulus of the borrow and base layers for new projects can only be estimated based on the R-value.

1. INTRODUCTION

The American Association of State Highway and Transportation Officials (AASHTO) adopted the Mechanistic-Empirical Pavement Design Guide (MEPDG) as an interim pavement design standard in 2008 (AASHTO 2008). The MEPDG is currently being implemented in the AASHTOWare®Pavement ME design software. The MEPDG conducts advanced mechanistic analysis of the pavement structure while taking into consideration the combined contributions of; traffic, climate, and materials properties. It follows a hierarchical approach in defining the required engineering properties of the pavement structure. In the case of unbound materials in base, subbase, and subgrade layers, the required engineering properties include the resilient modulus (Mr) and Poisson’s ratio (μ).

The impact of Mr on the response of the pavement structure to the combined actions of climate and traffic loads is highly significant, therefore, the Mr value of each pavement layer must be accurately specified. While the RLT provides a fundamental approach to characterize the nonlinear stress-dependent behavior of unbound materials, the test itself is time-consuming and costly. In light of these issues, most state highway agencies have elected to estimate the Mr from empirical properties such the California bearing ratio (CBR), R-value, or the unconfined compressive strength (UCS). The objective of this research study was to develop a prediction model for the resilient modulus of the unbound materials to be used for new design projects in Nevada DOT District 1.

2. LITERATURE REVIEW

Several studies have correlated the Mr to the unconfined compressive (UC) strength test for fine-grained soils as summarized in Table 1. The review of the data shows that the prediction of Mr for fine-
grained soils is significantly improved (i.e. higher $R^2$) when additional information from the UC test and on the properties of the soil are incorporated in the model in addition to the UC strength.

Table 1. Summary of Mr prediction models from unconfined compressive strength test.

<table>
<thead>
<tr>
<th>Study</th>
<th>Model</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drum et al. (1990)</td>
<td>$Mr = 45.8 +0.00052(1/a)+0.188(UCS)+0.45(PI)+0.216(y_d)-0.25(S)-0.15(P_{200})$</td>
<td>0.83</td>
</tr>
<tr>
<td>Lee et al. (1997)</td>
<td>$Mr = 695.4(S_{u1%}) − 5.93(S_{u1%})^2$</td>
<td>0.97</td>
</tr>
<tr>
<td>Hossain and Kim (2014)</td>
<td>Static Compaction $Mr = 6082+142(UCS)$</td>
<td>0.64</td>
</tr>
<tr>
<td></td>
<td>Impact Compaction $Mr = 4283+143(UCS)$</td>
<td>0.73</td>
</tr>
<tr>
<td></td>
<td>Static Compaction $Mr = 7884.2+99.7(UCS)+193.1(PI)-47.9(P_{200})$</td>
<td>0.86</td>
</tr>
<tr>
<td></td>
<td>Impact Compaction $Mr = 6113.0+95.1(UCS)+173.7(PI)-27.8(P_{200})$</td>
<td>0.91</td>
</tr>
<tr>
<td></td>
<td>$Mr = 657(S_{u1%}) − 6.75(S_{u1%})^2$</td>
<td>0.97</td>
</tr>
</tbody>
</table>

Table 2 summarizes the approaches used by selected agencies to determine the design resilient modulus of unbound layers (Von Quintus et al. 2015). The observation from Table 2 is that almost no agency performs repeated load triaxial resilient modulus tests for measuring Mr.

3. LABORATORY TESTING

The sampled materials were subjected to five groups of laboratory testing: Soil Classification, Moisture-density Relationship, Repeated Load Triaxial Resilient Modulus, Unconfined Compressive Strength, and Resistance Value “R-value”.

Table 2. Methods used to estimate design resilient modulus for selected agencies.

<table>
<thead>
<tr>
<th>State DOT</th>
<th>Test Procedure</th>
<th>Mr Correlated with and/or Determined</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arizona</td>
<td>NCHRP 1-28A</td>
<td>R-value and a library of Mr values</td>
</tr>
<tr>
<td>Colorado</td>
<td>AASHTO T 307-99</td>
<td>R-value and a library of Mr values</td>
</tr>
<tr>
<td>Florida</td>
<td>AASHTO T 307-99</td>
<td>LBR-value, backcalculation, and a library of Mr Values</td>
</tr>
<tr>
<td>Georgia</td>
<td>AASHTO T 307-99</td>
<td>Soil Support, Physical properties, library of Mr values</td>
</tr>
<tr>
<td>Idaho</td>
<td>AASHTO T 307-99</td>
<td>R-value and a library of Mr values</td>
</tr>
<tr>
<td>Michigan</td>
<td>AASHTO T 307-99</td>
<td>Library of Mr values and backcalculated from deflection basins</td>
</tr>
<tr>
<td>Missouri</td>
<td>AASHTO T 307-99</td>
<td>Regression equations from soil physical properties</td>
</tr>
<tr>
<td>Mississippi</td>
<td>AASHTO T 307-99</td>
<td>CBR and a library of Mr values</td>
</tr>
<tr>
<td>Montana</td>
<td>AASHTO T 307-99</td>
<td>Library of Mr values and backcalculation</td>
</tr>
<tr>
<td>Pennsylvania</td>
<td>AASHTO T 307-99</td>
<td>Unconfined compressive strength, library of Mr values</td>
</tr>
<tr>
<td>Tennessee</td>
<td>AASHTO T 307-99</td>
<td>Index of soil properties</td>
</tr>
<tr>
<td>Texas</td>
<td>AASHTO T 307-99</td>
<td>Texas Triaxial Classification Value</td>
</tr>
<tr>
<td>Virginia</td>
<td>AASHTO T 307-99</td>
<td>Unconfined compressive strength</td>
</tr>
<tr>
<td>Wisconsin</td>
<td>AASHTO T 307-99</td>
<td>Regression equations from soil physical properties</td>
</tr>
<tr>
<td>Wyoming</td>
<td>AASHTO T 307-99</td>
<td>R-value and a library of Mr values</td>
</tr>
</tbody>
</table>
The results of the triaxial testing of the base, borrow, and subgrade materials were used to develop the AASHTO T 307 standard procedure was followed for determining the Mr of the sampled materials. The loading pattern for the Mr test consists of a repeated axial cyclic stress of fixed amplitude with a loading duration of 0.1 second followed by a rest period of 0.9 second. The AASHTO standard stipulates detailed testing procedures for unbound materials, which include loading sequences, confining pressures, maximum axial stresses, cyclic stresses, constant stresses, and the number of loading applications. Overall, base materials are subjected to higher stresses during the testing than the subgrade soils despite the similarities in the testing sequences. non-linear models that relate the Mr to the stress conditions. For the base and borrow materials, the

Theta model was used to represent the stress-hardening behavior. For the subgrade material the Uzan and the Universal models were used (Puppala 2008). The constitutive model equations are given below.

\[
\text{Theta Model: } \quad M_R = K \theta^n \\
\text{Uzan Model: } \quad M_R = K \theta^n \sigma_d^n \\
\text{Universal Model: } \quad M_R = k_1 p_a \left( \frac{\theta}{p_a} \right)^{k_2} \left( \frac{\tau_{oct}}{p_a} + 1 \right)^{k_3}
\]

Where:
- \( K, n, \text{ and } m \): regression coefficients
- \( \theta \): bulk stress, psi
- \( \sigma_d \): deviator stress, psi
- \( k_1, k_2, k_3 \): regression coefficients
- \( p_a \): atmospheric pressure, psi
- \( \tau_{oct} \): octahedral shear stress, psi

The resilient modulus value was obtained from the average value of the last five cycles for each sequence. The method of least squares in Microsoft Excel was used to develop the regression coefficients in the constitutive models. The Theta model showed good correlation for the base and borrow materials. Both the Universal and Uzan models showed good correlations for the subgrade materials. One of the borrow material’s (Contract 3583) constitutive model was similar to subgrade material. The variation of resilient modulus with different state of stress for the base, borrow, and subgrade materials are presented in Figures 1–3, respectively.
Figure 2. Variation of resilient modulus of borrow materials with bulk stress.

Figure 3. Variation of resilient modulus of subgrade materials with bulk stress.

Unconfined compressive strength (UCS) tests were conducted in accordance with AASHTO T 208. The continuous stress-strain responses were recorded to produce a complete stress-strain diagram. As was discovered in the literature review, the inclusion of the stress-strain parameters may significantly improve the correlation.

Samples were prepared at the optimum moisture content and maximum dry density with the vibratory compactor. A 6.0 inch diameter by 12.0 inch height mold was used to meet the requirement of; maximum particles size has to be smaller than one-sixth of the specimen diameter. Tests were conducted at a strain rate between 0.2 and 2 percent per minute. Two replicates were tested for each source of material. Table 3 summarizes the unconfined compressive strength properties for the base, borrow, and subgrade materials.
The R-value testing is an empirical measure of unbound materials strength and expansion potential, which has been used in designing flexible pavements in several states including Nevada. The R-value of the collected base, borrow, and subgrade materials were determined in accordance with the NDOT test method Nev. T115D. Steel mold with the diameter of 4 inch and height of 5 inch was used to prepare the sample. The mechanical kneading compactor was used to compact with 100 tamps were applied at 200 psi foot pressure. Table 4 summarizes the R-values of the evaluated materials.

### Table 3. Summary of unconfined compressive strength test results.

<table>
<thead>
<tr>
<th>Material</th>
<th>UC (psi)</th>
<th>Strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Base</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3546</td>
<td>2.8</td>
<td>0.65</td>
</tr>
<tr>
<td>3583</td>
<td>7.3</td>
<td>0.51</td>
</tr>
<tr>
<td>3597</td>
<td>3.4</td>
<td>0.60</td>
</tr>
<tr>
<td>3605</td>
<td>6.6</td>
<td>0.64</td>
</tr>
<tr>
<td>3607</td>
<td>9.7</td>
<td>0.72</td>
</tr>
<tr>
<td>3613</td>
<td>3.7</td>
<td>0.51</td>
</tr>
<tr>
<td><strong>Borrow</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3546</td>
<td>1.3</td>
<td>0.60</td>
</tr>
<tr>
<td>3583</td>
<td>5.6</td>
<td>0.80</td>
</tr>
<tr>
<td>3597</td>
<td>6.6</td>
<td>0.78</td>
</tr>
<tr>
<td>3613</td>
<td>4.1</td>
<td>0.54</td>
</tr>
<tr>
<td><strong>Subgrade</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>I-15/Goodsprings</td>
<td>5.1</td>
<td>0.50</td>
</tr>
<tr>
<td>US-95/Searchlight</td>
<td>8.5</td>
<td>0.57</td>
</tr>
<tr>
<td>NV-375/Rachel</td>
<td>2.7</td>
<td>0.76</td>
</tr>
<tr>
<td>US-95/Bonnie Claire</td>
<td>7.6</td>
<td>0.78</td>
</tr>
<tr>
<td>US-93/Crystal Spring MP62</td>
<td>8.8</td>
<td>0.70</td>
</tr>
<tr>
<td>US-93/Crystal Spring MP67</td>
<td>8.9</td>
<td>0.72</td>
</tr>
</tbody>
</table>

### Table 4. Summary of R-value test results.

<table>
<thead>
<tr>
<th>Base Materials</th>
<th>3546</th>
<th>3583</th>
<th>3597</th>
<th>3605</th>
<th>3607</th>
<th>3613</th>
</tr>
</thead>
<tbody>
<tr>
<td>83</td>
<td>80</td>
<td>71</td>
<td>78</td>
<td>85</td>
<td>83</td>
<td></td>
</tr>
<tr>
<td><strong>Borrow Materials</strong></td>
<td>3546</td>
<td>3583</td>
<td>3597</td>
<td>3607</td>
<td>3613</td>
<td></td>
</tr>
<tr>
<td>78</td>
<td>44</td>
<td>78</td>
<td>78</td>
<td>84</td>
<td></td>
<td></td>
</tr>
<tr>
<td>84</td>
<td>75</td>
<td>80</td>
<td>74</td>
<td>74</td>
<td>71</td>
<td></td>
</tr>
</tbody>
</table>

4. DESIGN MR VALUE

The steps to determine the design value for the unbound layers (aggregate base, borrow materials, and subgrade soil) using the repeated load triaxial resilient modulus tests are listed below. These steps are in accordance with the MEPDG Manual of Practice as well as in the final report for NCHRP project 1-37A (ARA 2004).
1. Assume a trial flexible pavement structure.
2. Calculate the at-rest stress state for the base and subgrade layers.
3. Start with the subgrade and move upward in the pavement structure to establish the design resilient modulus.
4. For the design truck axle load and season, calculate the load-related stresses.
5. Superimpose the at-rest and load-related stresses.
6. The stress state at which the elastic modulus and laboratory resilient modulus are equal is the design value.
7. Check the design resilient modulus determined for the lower unbound layers to be sure it is the same, as previously determined. This step is an iterative process to determine a stable design resilient modulus.

The above procedure was used to identify the design Mr values for new flexible pavement designs conducted following the NDOT pavement design method. Table 5 summarizes the established design Mr values for new pavement designs at the various locations of materials sampling.

Table 5. Summary of Design Resilient Moduli values for Flexible Pavement Structures.

<table>
<thead>
<tr>
<th>Material</th>
<th>5.5 inch AC, 16 inch CAB and 10 inch Borrow</th>
<th>8.5 inch AC, 18 inch CAB and 10 inch Borrow</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Resilient Modulus (psi)</td>
<td>Resilient Modulus (psi)</td>
</tr>
<tr>
<td>Base</td>
<td>Borrow</td>
<td>Subgrade</td>
</tr>
<tr>
<td>---------------</td>
<td>--------</td>
<td>---------------------------</td>
</tr>
<tr>
<td>3583</td>
<td>3546</td>
<td>US-93/Crystal Spring MP62</td>
</tr>
<tr>
<td>3583</td>
<td>3546</td>
<td>Borrow 3583</td>
</tr>
<tr>
<td>3583</td>
<td>3597</td>
<td>US-93/Crystal Spring MP62</td>
</tr>
<tr>
<td>3583</td>
<td>3597</td>
<td>Borrow 3583</td>
</tr>
<tr>
<td>3583</td>
<td>3613</td>
<td>US-93/Crystal Spring MP62</td>
</tr>
<tr>
<td>3583</td>
<td>3613</td>
<td>Borrow 3583</td>
</tr>
</tbody>
</table>

5. RESILIENT MODULUS PREDICTION MODEL DEVELOPMENT

The goal of this analysis is to develop a prediction model for the design Mr to be used in the structural design of new flexible pavements as function of empirical and physical properties for the unbound materials. The properties considered in the development of the prediction model, included; the design Mr determined in section 4 as the dependent variable and R-value, unconfined compressive strength, materials passing sieves #200, #40, 3/8”, maximum dry density, optimum moisture content, and plasticity index as the independent variables. In addition, the pavement equivalent thickness was identified as a critical parameter in the determination of the design Mr for the base and borrow layers.
Based on the results of the statistical analysis, it was observed that the design Mr of the subgrade does not change with the pavement structure. However, the design resilient modulus of base and borrow layers change significantly with the pavement structure. Accordingly, the data for the base and borrow layers were combined to develop a single prediction model while the prediction model for the subgrade layer was established separately.

Two sets of models were examined; a) a prediction model based on the unconfined compressive strength and b) a prediction model based on the R-value. Based on the analysis of the multiple models, the resilient modulus of the subgrade can be estimated from the R-value or the UCS. However, for the base and borrow materials, the R-value only should be used to predict the resilient modulus as presented in Equations 4 – 7.

\[
\text{Ln}(\text{Mr}_{SG}) = 7.4514+0.0036\times\text{P}#200-0.0129\times\text{P}#3/8+0.0158\times\gamma_d+0.0973\times\text{UCS}+0.0311\times\text{PI}
\]

\[
\text{Ln}(\text{Mr}_{SG}) = 3.1784+0.018\times\text{R-value}+0.0136\times\text{P}#40+0.0315\times\gamma_d+0.0433\times\text{PI}
\]

\[
\text{Ln}(\text{Mr}_{CAB}) = 7.3224+0.0366\times\text{R-value}-0.0656\times\text{P}#40+0.0256\times\text{P}#3/8-0.0893\times\text{OMC}-0.0270\times\text{Heq}
\]

\[
\text{Ln}(\text{Mr}_{BOR}) = 8.9671+0.0102\times\text{R-value}+0.0123\times\text{P}#3/8-0.0743\times\text{OMC}-0.0189\times\text{Heq}
\]

Where;

\text{P}#200: passing sieve #200, %
\text{P}#3/8: passing sieve #3/8, %
\text{P}#40: passing sieve #40, %
\gamma_d: dry unit weight (pcf)
\text{UCS: unconfined compressive strength, psi}
\text{PI: plasticity index}
\text{OMC: optimum moisture content, %}
\text{Heq: pavement equivalent thickness, inch}

Based on the analysis of the data generated from this experiment, a correlation was found possible between the equivalent thickness and depth from pavement surface to the location where the state of stress was calculated (D). The depth of location for state of the stress calculation for the aggregate base and borrow layers are determined at their quarter depth, while for the subgrade, stresses are determined at 18 inches into the subgrade. According to the MEPDG procedure, a trial pavement structure must be assumed in the design process. Therefore, using the assumed pavement structure, the depth to the state of stress calculation can be determined for each layer and used to calculate the equivalent thickness in terms of the layer being analyzed using Equations 8 and 9. Once the equivalent thickness is computed, the resilient modulus of the layer being analyzed can be estimated from the models presented in Equations 6 and 7 and can be used as a Level 2 input for the AASHTOWare® Pavement ME Design software (AASHTO 2008).

\[
\text{Heq}_{New-CAB} = 2.2432 \times D - 1.9263 
\]

\[
\text{Heq}_{New-Bor} = 1.3211 \times D + 9.6409
\]

Where;

\text{Heq}_{New-CAB}: equivalent thickness of the base layer for new design, inch
\text{Heq}_{New-Bor}: equivalent thickness of the borrow layer for new design, inch
\text{D}: depth of location for state of stress calculation (base and borrow), inch
6. CONCLUSIONS AND RECOMMENDATIONS

The major objective of this study is to develop a resilient modulus prediction model for unbound materials to be used for new flexible pavement design projects in NDOT District 1. Based on the generated data from the experiment and the statistical analyses, the following observations and conclusions can be made:

- The stress dependent behavior of the resilient modulus for the base and borrow material fits very well the Theta model.
- The stress dependent behavior of resilient modulus for the subgrade materials fits very well both the universal model and Uzan model.
- The resilient modulus of base and borrow materials is significantly influenced by the pavement structure.
- The design resilient modulus of the subgrade layer for new pavement designs can be predicted based on UCS or R-value.
- The design resilient modulus of the base and borrow layers for new pavement designs can be predicted based on R-value from.

7. REFERENCES


Evaluation of Geocell Reinforced Backfill for Airfield Pavement Repair

Lyan Garcia, Jeb Tingle
U.S. Army Engineer Research and Development Center

ABSTRACT

After natural disasters, such as earthquakes or floods, or when geological features produce sinkholes, temporary airfield pavement repairs should be accomplished quickly and with limited materials and manpower to restore flight operations as quickly as possible. Typically, airfield pavement repairs have used traditional backfill material options such as crushed stone and flowable fill. In the event of a natural disaster or emergency, traditional materials may be temporarily unavailable due to disruptions in normal supply chains. For temporary or emergency repairs, an airfield mat surfacing or rapid-setting concrete cap could be used to provide a quick repair to reopen the airport for emergency flight operations. Under emergency conditions, it is desirable to reduce the logistical burden while providing a suitable repair, especially in areas with weak subgrades. Geocells are cellular confinement systems of interconnected cells that can be used to reinforce geotechnical materials. The primary benefit of geocells is that lower quality backfill materials can be used instead of crushed stone to provide a temporary repair. This paper will summarize a series of field experiments performed to evaluate different geocell-reinforced thicknesses to verify their effectiveness at supporting heavy aircraft loads. Results provide specific recommendations for using geocell technology for backfill reinforcement for emergency airfield repairs.

1. INTRODUCTION

The concept of using sand-confinement systems, or geocells, as an expedient construction method for reinforcing pavement base courses above soft ground was first investigated by Webster and Alford (1978). The study focused on military truck traffic, and findings supported the use of geocells as a suitable construction technique where sand was readily available. The investigations were followed by Webster (1979, 1980) to determine optimum geocell grid size (cell diameter and thickness/height), cell shape, geocell material, and minimum surfacing requirements for handling over-the-shore military 5-ton truck operations. A geocell with a height of 8 in. was recommended, although a geocell with a 6-in.-height was recommended if the sand had a good gradation. As cell area increased, the performance tended to decrease; the authors recommended cell areas of 36 to 44 square in. Among grid materials recommended for further evaluation were aluminum and plastic. Webster (1984, 1986) further refined the geocell technology into a high-density polyethylene (HDPE) grid that could be easily collapsed and then expanded on site to an area of approximately 8 ft. by 20 ft. of total panel size. HDPE strips were ultrasonically welded at intervals of 13 in. In a program sponsored by the Waterways Experiment Station in Vicksburg, MS, geocells were demonstrated for military road construction and were ultimately proven as a practical solution for increasing the offload efficiency during over-the-shore operations. Webster (1986) published specifications for plastic grid sections as a result of these demonstrations, and the
The geocell concept was commercialized with the product called “Geoweb”. At this point, however, geocells were still being evaluated for application to construction or repair of airfield pavements.

Read and Dukes (1988) investigated the use of geocells to reinforce the base course in bomb damage crater repairs to alleviate the requirement for high-quality, well-graded crushed stone. The repairs consisted of two layers of geocell-reinforced sand (16 in. total reinforced thickness) placed on a clay subgrade with a California bearing ratio (CBR) of 5. Each repair was surfaced with fiberglass matting and trafficked with a load cart simulating the F-15 aircraft (355 psi tire, 30,600-lb wheel load). Nearly all deformation occurred in the sand grid layers due to compression of the sand fill, local shear within the cells, and lateral spreading of the grids. The authors concluded that compaction was a major factor affecting performance and also recommended a coarser sand.

Since then, expedient airfield pavement repair with geocells has been established as a standard procedure with detailed methods included in the Unified Facilities Criteria (UFC): Airfield Damage Repair (UFC 3-270-07, 2002). When combined with the recommended fiberglass foreign object damage (FOD) cover, the repair method is approved for C-130 operations on runways and taxiways. Geocell reinforcement is a useful alternative to traditional backfill methods such as crushed stone or rapid setting flowable fill because lower quality backfill materials (i.e. indigenous materials) can be used to provide a temporary repair. The procedure requires two 8-in.-thick layers of geocell (Figure 1) filled with an indigenous material (ideally sand) placed perpendicular to each other over a minimum subgrade CBR of 4. Geocell material and geometry requirements align with the specifications of the Geoweb product.

![Figure 1. Airfield pavement repair method using geocell reinforcement.](image)

New geometries and geocell materials have been introduced commercially for several years, but research that specifically addressed the use of new products for airfield pavement repair was limited. To address this capability gap, the U.S. Army Engineer Research and Development Center in Vicksburg, MS initiated a research program that aimed at evaluating new geocell products/materials and geometries, and their combination with good-quality and poor-quality fill materials. Information from the program would help develop new product specifications and requirements for airfield pavement repair, and ideally validate other geometries and materials that are commercially available for this application. This
paper summarizes the results of one of the objectives of the program, which focused on determining optimal geocell height(s) that could support modern heavy aircraft loads, such as the C-17 globemaster. Simulated repairs were built at the Waterways Experiment Station, trafficked with simulated aircraft loading, and deformation was monitored to measure the performance of the repairs.

2. MATERIALS

2.1 Sand and Clay

To evaluate the influence of geocells, the field experiments were carried out using a concrete sand as filling material. The material was classified as a poorly graded sand (SP) according to the Unified Soil Classification System (ASTM D2487). The geocell-reinforced SP was placed over a low plasticity clay (CL) subgrade to provide a uniform foundation throughout the tests. Specific gravity (ASTM D854) was determined for each soil, and modified proctor testing (ASTM D1557 – Method C) and laboratory California bearing ratio (CBR) testing (ASTM D1883) were performed on each soil. Physical properties of the soils are shown in Table 1.

Table 1. Soil characteristics and classification data.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Poorly Graded Sand (SP)</th>
<th>Low Plasticity Clay (CL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>2.65</td>
<td>2.75</td>
</tr>
<tr>
<td>Optimum water content (%)</td>
<td>1.9</td>
<td>14.5</td>
</tr>
<tr>
<td>Maximum dry density (pcf)</td>
<td>111.5</td>
<td>115.0</td>
</tr>
<tr>
<td>CBR at optimum water content (%)</td>
<td>20.5 (est.)</td>
<td>57 (est.)</td>
</tr>
<tr>
<td>Cc</td>
<td>0.85</td>
<td>--</td>
</tr>
<tr>
<td>Cu</td>
<td>2.27</td>
<td>--</td>
</tr>
<tr>
<td>LL</td>
<td>--</td>
<td>37</td>
</tr>
<tr>
<td>PI</td>
<td>--</td>
<td>14</td>
</tr>
<tr>
<td>Finer than #200</td>
<td>1.7</td>
<td>98.8</td>
</tr>
</tbody>
</table>

2.2 Geocells and geotextile

The geocells used in this study were made of high-density polyethylene (HDPE) strips that were textured, perforated, and ultrasonically-welded at intervals of 14 in. along their length. The system was expandable on-site to form a 3D grid structure. The individual cells were approximately 8.8 in. by 10.2 in. when the section was expanded. Three different geocell heights were evaluated: 4 in., 6 in. and 8 in. A photograph of a typical geocell section used in this study is shown in Figure 2.

Measured mechanical properties for each geocell thickness, including the ultrasonic weld properties, are reported in Table 2. Peel, shear and splitting strength tests were conducted on the seams and failure occurred on the parent HDPE sheets at the perforations. A 6-oz, non-woven, needle punched, polypropylene geotextile fabric was used at the interface of geocell layers and at the surface of the CL subgrade layer.
2.3 Fiberglass-Reinforced Polymer (FRP) FOD cover

The Fiberglass-Reinforced Polymer (FRP) FOD cover panels are made of layers of glass fiber and aramid fabric impregnated with polyester resin (Rushing et al. 2016). A full-panel measured 18 ft. by 6.6 ft., and a half-panel measured 9.3 ft. by 6.6 ft. Each panel was connected using metal bushings and bolts. One repair required one full-panel and two half-panels. The leading and trailing edges of the repair were anchored to the pavement using a heavy-duty sleeve concrete expansion anchor.

3. EXPERIMENTAL PROGRAM

3.1 Experiment Layout

The field experiment was performed at the Outdoor Pavement Testing Facility, East Campus of the Waterways Experiment Station, Vicksburg, MS. Four (4) simulated craters were cut to 8.5 ft. square and excavated to a depth of 34 in. from concrete with a thickness of 15 in. A layer of CL was placed and compacted at the bottom of the repairs to provide a consistent subgrade throughout the tests. Two independent series of tests were conducted. Series 1 evaluated the 4-in., 6-in., and 8-in. geocell heights (i.e. different backfill reinforcement thicknesses), as shown in Figure 3. Series 2 aimed at validating the
results of the first series of tests by reinforcing the same thickness of backfill (24 in.): 4 layers of the 6-in.-thick geocell and 3 layers of the 8-in.-thick geocell (Figure 4). The backfill in each series was SP. Each repair was surfaced with the FRP FOD cover. Simulated single-wheel C-17 aircraft traffic was applied to the finished repairs and deformation was monitored at different pass levels.

Figure 3. Series 1 repairs.

Figure 4. Series 2 repairs.

4. CONSTRUCTION

Each simulated crater was initially prepared by cutting the PCC surface with the 60-in. wheel saw attachment (SW60) on the compact track loader (CTL), which was representative of cutting around upheaval in a realistic scenario. Once the repair was cut to size, the PCC was broken into smaller pieces with the moil-tipped hammer attachment on the excavator. Large debris and material underneath the PCC surface were then removed with the bucket attachment on the excavator. Each crater was excavated to a minimum depth of 34 in. to ensure that a uniform natural foundation was reached. The CL layer was placed in approximately 3- to 4-in.-thick compacted lifts (5- to 6-in.-thick loose). Compaction of each lift was carried out by applying two coverages with a rammer. A coverage consisted of starting compaction in one corner of the repair and working towards the center of the repair in a circular motion, and then returning in a circular motion towards the corner where compaction began.

Once the surface of the CL layer was finished, the horizontal dimensions of the crater were measured so that the geocell sections and geotextile could be cut to size. Once geocell sections were cut, the installation of the geocell-reinforced SP for each repair was as follows (as recommended in UFC 3-270-
(1) the surface of the CL layer was lined with the geotextile; (2) the first layer of geocells was expanded parallel to the centerline. Rebar was placed at all corners of the repair to hold the expanded geocell section in place, and SP was used to fill some of the geocell pockets along the edges to help hold the section in place (Figure 5a); (3) the geocell section was backfilled by dropping the SP vertically into the repair to avoid displacing the section (Figure 5b). The section was overfilled by approximately 2 in. and the SP was spread evenly; (4) the SP was compacted using a plate compactor (Figure 5c) until at least 2 coverages were completed. Any excess material was struck off level with the top of the geocells; (5) the surface of the first layer of geocells was lined with the geotextile; (6) the second layer of geocells was expanded perpendicular to the centerline and held in place, as described above; (7) the second layer of geocells was backfilled and compacted; (8) for Series 2 repairs, the next layer(s) were built using the same process. For Series 1 and 2 repairs, the top reinforced layer was compacted with the vibratory roller attachment on the CTL (Figure 5d). A 1- to 2-in. buffer of SP was placed above the top reinforced layer prior to installing the FRP FOD cover. The SP backfill in the control repair that did not have geocell reinforcement was built in 6-in.-thick lifts. Each repair was then surfaced with the FRP FOD cover.

![First geocell layer secured at edges.](image1)

![Backfilling with SP.](image2)

![Compacting with plate compactor.](image3)

![Compacting with CTL roller attachment.](image4)

Figure 5. Geocell-reinforced backfill construction process.

Data collection performed during construction included oven moisture (ASTM D2216), nuclear gage (ASTM D6938) and dynamic cone penetrometer (ASTM D6951) tests. Each lift was also surveyed to verify the thickness. The in-situ soil properties for both series’ of repairs prior to trafficking are reported in Table 3 and Table 4. Note that the CBR determined from the DCP at the top layer was not reported due to lack of confinement of the SP at the top 6 in. to 8 in.
Table 3. In-situ soil properties for Series 1 repairs.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Property</th>
<th>No Geocell</th>
<th>Thickness of Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Control</td>
<td>2 layers of 4-in. geocell (8 in.)</td>
</tr>
<tr>
<td>Geocell-reinforced SP (Layer 2)</td>
<td>Nuclear Wet Density (pcf)</td>
<td>113.7</td>
<td>114.6</td>
</tr>
<tr>
<td></td>
<td>Nuclear Dry Density (pcf)</td>
<td>110.7</td>
<td>109.7</td>
</tr>
<tr>
<td></td>
<td>Nuclear Moisture (%)</td>
<td>2.7</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>Oven Moisture (%)</td>
<td>3.1</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td>Thickness (in.)</td>
<td>7.4</td>
<td>4.5</td>
</tr>
<tr>
<td>Geocell-reinforced SP (Layer 1)</td>
<td>Nuclear Wet Density (pcf)</td>
<td>112.7</td>
<td>110.8</td>
</tr>
<tr>
<td></td>
<td>Nuclear Dry Density (pcf)</td>
<td>109.6</td>
<td>106.4</td>
</tr>
<tr>
<td></td>
<td>Nuclear Moisture (%)</td>
<td>2.8</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>Oven Moisture (%)</td>
<td>3.0</td>
<td>3.2</td>
</tr>
<tr>
<td></td>
<td>CBR (from DCP)</td>
<td>7.0</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>Thickness (in.)</td>
<td>9.4</td>
<td>4.0</td>
</tr>
<tr>
<td>CL</td>
<td>Nuclear Wet Density (pcf)</td>
<td>112.0</td>
<td>102.9</td>
</tr>
<tr>
<td></td>
<td>Nuclear Dry Density (pcf)</td>
<td>99.9</td>
<td>91.8</td>
</tr>
<tr>
<td></td>
<td>Nuclear Moisture (%)</td>
<td>12.3</td>
<td>12.1</td>
</tr>
<tr>
<td></td>
<td>Oven Moisture (%)</td>
<td>16.1</td>
<td>15.6</td>
</tr>
<tr>
<td></td>
<td>CBR</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td></td>
<td>Thickness (in.)</td>
<td>16.8</td>
<td>25.5</td>
</tr>
</tbody>
</table>
Table 4. In-situ soil properties for Series 2 repairs.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Property</th>
<th>Thickness of Reinforcement = 24 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Three 8-in. geocell layers</td>
</tr>
<tr>
<td>Geocell-reinforced SP (Layer 4)</td>
<td>Nuclear wet density (pcf)</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Nuclear dry density (pcf)</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Nuclear moisture (%)</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Oven moisture (%)</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Thickness (in.)</td>
<td>---</td>
</tr>
<tr>
<td>Geocell-reinforced SP (Layer 3)</td>
<td>Nuclear wet density (pcf)</td>
<td>109.1</td>
</tr>
<tr>
<td></td>
<td>Nuclear dry density (pcf)</td>
<td>103.8</td>
</tr>
<tr>
<td></td>
<td>Nuclear moisture (%)</td>
<td>5.1</td>
</tr>
<tr>
<td></td>
<td>Oven moisture (%)</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td>CBR (from DCP)</td>
<td>---</td>
</tr>
<tr>
<td></td>
<td>Thickness (in.)</td>
<td>9.86</td>
</tr>
<tr>
<td>Geocell-reinforced SP (Layer 2)</td>
<td>Nuclear wet density (pcf)</td>
<td>105.3</td>
</tr>
<tr>
<td></td>
<td>Nuclear dry density (pcf)</td>
<td>99.9</td>
</tr>
<tr>
<td></td>
<td>Nuclear moisture (%)</td>
<td>5.5</td>
</tr>
<tr>
<td></td>
<td>Oven moisture (%)</td>
<td>4.9</td>
</tr>
<tr>
<td></td>
<td>CBR (from DCP)</td>
<td>6.7</td>
</tr>
<tr>
<td></td>
<td>Thickness (in.)</td>
<td>8.0</td>
</tr>
<tr>
<td>Geocell-reinforced SP (Layer 1)</td>
<td>Nuclear wet density (pcf)</td>
<td>105.0</td>
</tr>
<tr>
<td></td>
<td>Nuclear dry density (pcf)</td>
<td>100.4</td>
</tr>
<tr>
<td></td>
<td>Nuclear moisture (%)</td>
<td>4.6</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td></td>
</tr>
<tr>
<td>------------------------------</td>
<td>-------------</td>
<td>------------------------------</td>
</tr>
<tr>
<td>Oven moisture (%)</td>
<td>4.5</td>
<td>6.6</td>
</tr>
<tr>
<td>CBR (from DCP)</td>
<td>8.3</td>
<td>9.6</td>
</tr>
<tr>
<td>Thickness (in.)</td>
<td>7.9</td>
<td>5.8</td>
</tr>
<tr>
<td>Nuclear wet density (pcf)</td>
<td>117.2</td>
<td>118.6</td>
</tr>
<tr>
<td>Nuclear dry density (pcf)</td>
<td>103.6</td>
<td>104.2</td>
</tr>
<tr>
<td>Nuclear moisture (%)</td>
<td>13.1</td>
<td>13.8</td>
</tr>
<tr>
<td>Oven moisture (%)</td>
<td>17.0</td>
<td>15.7</td>
</tr>
<tr>
<td>CBR (from DCP)</td>
<td>14.0</td>
<td>17.0</td>
</tr>
<tr>
<td>Thickness (in.)</td>
<td>8.8</td>
<td>8.8</td>
</tr>
</tbody>
</table>

5. TRAFFICKING AND DATA COLLECTION

Simulated traffic was applied to the repairs with a single-wheel C-17 load cart equipped with a 50-in. by 21-in., 30-ply tire inflated to 142 psi and loaded so that the test gear was supporting 38,500 lb (Figure 6). This weight represents the maximum load on one tire of a C-17 aircraft allowed in contingency operations. A normally distributed pattern of simulated traffic was applied in a 4.5-ft.-wide traffic lane. Traffic was applied by driving the load cart forward and then backward in the same wheel path, then moving laterally approximately one tire width on each forward pass.

![Figure 6. Tracking with the single-wheel C-17 load cart on the finished repairs.](image)

Data collection at each repair included rod and level measurements at the centerline and along two cross sections that were offset 3 ft. from the edges of the repair. Rod and level measurements along the cross section were taken on the loaded FRP FOD cover surface, where two (2) 2-kip blocks (much less
weight than the load cart) were placed next the cross section. This was done in attempt to measure the deformation of the backfill throughout trafficking without having to disassemble the panels, since the FRP FOD cover tended to bridge over any deformation on the reinforced SP backfill underneath. All measurements were taken prior to starting trafficking, at different pass levels throughout trafficking, and after trafficking was completed. Measurements were also taken during construction and after trafficking was completed at each construction layer to measure the deformation at different depths. Failure of the repairs was defined as 3 in. of rutting on the FRP FOD cover surface.

6. RESULTS AND DISCUSSION

A summary of the deformation measured throughout trafficking of the Series 1 repairs is shown in Figure 7. Note that trafficking on all repairs had to be stopped after 3 in. of deformation were exceeded on the repairs with 8 in. and 16 in. of reinforcement. Instability of the load cart at these repairs made it difficult to access the other two repairs. However, sufficient data had been collected on all repairs for analysis. The rate of deformation at the repair with 8 in. of reinforcement was the highest, followed by the control (no geocell) repair. The repairs with 16 in. and 12 in. of reinforcement performed well and had similar rates of deformation. In general, 12 in. of reinforcement showed the best performance. Post traffic inspection of the repairs revealed cell wall buckling at the top and bottom layers of the 8-in.-thick geocells (Figure 8), where sand was displaced from adjacent cells and the wall bent. In all geocell-reinforced backfill repairs, cell penetration was observed, where the geocells pushed downward relative to the sand fill into the fabric membrane layer underneath; however, the geotextile did not tear as a result of this. Post-test deformation measurements at each layer showed that the deformation at the surface of the CL subgrade reduced as the thickness of reinforcement was increased, showing the effectiveness and impact of the geocell at distributing the applied load so as not to overstress the subgrade.

![Figure 7. Deformation of Series 1 repairs at different pass levels.](image-url)
Since the 8-in.- and 6-in.-thick geocells both performed well and had similar rates of deformation, additional testing was performed to eliminate the variable of the thickness of reinforcement and the height of the CL layer relative to the surface of the repair (i.e. the effect of the subgrade on overall rutting performance). Series 2 aimed at specifically addressing the effect of geocell height on performance by making a relative comparison. The thickness of reinforcement was the same, but a different combination of layers was used (Figure 4). A summary of the deformation measured throughout trafficking of the Series 2 repairs is shown in Figure 9. The rate of deformation for the repair with three layers of 8-in. geocells was higher than that of the repair with four layers of 6-in. geocells, where the latter sustained nearly twice the number of passes before failure by deformation. Similar rates of deformation were noted when compared to the Series 1 repairs, which may be attributed to having moisture and densities closer to optimum in the Series 1 repairs. Cell penetration was observed at all of the layers, including the subgrade layer; however, the geotextile did not tear as a result of this. Post-test deformation measurements at each layer showed that the deformation at the surface of the subgrade was similar for both repairs and remained under 1 in.

Figure 8. Buckling of the cell wall of the 8-in.-thick geocell after Series 1 trafficking was completed.

Figure 9. Deformation of Series 2 repairs at different pass levels.
Results from the Series 1 and Series 2 trafficking showed that an increase in the thickness of reinforcement provides increased protection to the subgrade, but there exists an optimum geocell thickness (6 in.) for protecting the subgrade. Better compaction can be achieved with thinner geocell layers, and the possibility of cell wall buckling is reduced with shorter geocell height. Based on these results, it appears that the 6-in. geocells are also applicable for airfield pavement repairs and can be used as an alternative to the traditional 8-in.-thick geocells used in this application.

7. CONCLUSIONS AND RECOMMENDATIONS

This paper summarizes the results from full-scale testing of airfield pavement repairs using an SP backfill reinforced with varying thicknesses of geocells. The objective was to verify if a different geocell thickness could be used for this application instead of the 8-in.-thick geocells that are traditionally required for airfield pavement repairs. Geocell thicknesses of 4 in., 6 in. and 8 in. were evaluated in two series of full-scale tests.

With the exception of the repair with two layers of 4-in.-thick geocell (8 in. of reinforcement), the geocells improved the load-deformation behavior of the repair through lateral confinement of the SP. With an increase in the thickness of reinforcement, the geocell mattress was able to distribute the surface loading over a wider area to better protect the subgrade. However, surface deformation was higher for thicker layers of geocell. Compaction appeared to be a major factor in the performance of the geocells, where the 6-in.-thick geocells achieved better compaction. Although none of the reinforced SP layers reached 100% of the maximum dry density, Series 1 repairs with geocell heights of 6 in. and 8 in. had similar rates of deformation to Series 2 repairs, despite the fact that Series 2 repairs had 24 in. of reinforcement. Series 1 repairs reached at least 95% of the maximum dry density of the SP, while Series 2 repairs reached up to 93% (moisture contents were much higher than optimum). Based on the results of this testing program, a minimum of 12 in. of geocell reinforcement (two layers with 6 in. of reinforcement) is recommended for airfield pavement repairs. This is favorable for procurement because the 6-in.-thick geocells tend to be more readily available and are cheaper since they require less material. Note that the conclusions presented in this paper are based on the materials tested in this study under specific loading conditions.

REFERENCES


Webster, S. L. (1986). *Sand-grid Demonstration Roads Constructed for JLOTS II Tests at Fort Story, Virginia* (No. GL-86-19). Waterways Experiment Station, Vicksburg, MS.


Use of Geosynthetics in Diverse Railroad Applications

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ABSTRACT

Railroads often face a wide range of track substructure issues that require different types of solutions. Multiple geosynthetic designs available today allow railroads to determine appropriate geosynthetics to address various substructure issues that railroads encounter. In this paper, case studies representing two commonly encountered railroad substructure issues are introduced: (1) ballast pockets on a soft railroad embankment, and (2) subgrade mud pumping in an often-flooded railway cut. Both situations required frequent maintenance, and subgrade remediation was desired to reduce maintenance activities in both areas. Ballast pockets are a common soft subgrade/embankment issue for railroads because the ballast pocket can trap water and keep the subgrade material in a softened and weakened state. A continual high-moisture state results in soil that is vulnerable to deformation, which then requires frequent surfacing and ballast dumping to maintain track surface. A combination of geogrid, ballast renewal, and ballast drains was installed at a ballast pocket location to stabilize the ballast layer and drain excess water at the bottom of the ballast pocket. The results from this study showed a significant reduction in maintenance activity and a payback period of less than a year.

Cut regions are often problematic areas because excess water from the surrounding region will runoff into the lower-elevation track, often resulting in flooded track. In addition, this can cause subgrade mud pumping as the saturated subgrade (shale in this case) breaks apart and fine particles pump upwards into the ballast. Ideally, drainage solutions would be used but this is not possible due to space and elevation restrictions. Therefore, multiple geosynthetics that act as a barrier between this site’s shale subgrade and ballast were installed. The results showed a significant reduction of fines in the ballast layer.

These two case studies show that geosynthetics can be a useful railroad remediation solution as long as the root cause of the issue is appropriately identified and addressed.

1. INTRODUCTION

Maintaining a properly functioning track substructure is important for railroad operations and asset management because poor substructures can often result in increased track geometry degradation and decreased rail, tie, and fastener lifespans. This can decrease operational capacity due to slow orders and increases track maintenance costs.
The functional railroad track substructure consists of four major components: ballast, subballast, subgrade, and drainage (Li et al., 2016; AREMA, 2018). Ballast is typically made up of one to two-inch angular crushed rocks that provides a strong trackbed, distributes loads to the subballast and subgrade, and allows the drainage of water. The subballast is an intermediate layer that acts as a barrier between the large ballast particles and the subgrade; however, it can serve as a second foundation layer to help distribute loads if a weak subgrade is present. The subgrade is usually the natural soil but can also be fill in the case of embankments. External drainage typically consists of ditches that allow for the runoff water to be directed away from the track.

Over time, the track substructure can experience a wide range of issues from the breakdown of ballast, inadequate drainage, and subgrade deformation, each of which requires different maintenance or remedial solutions to address the root cause of the issue. One long-term strategic research goal of Transportation Technology Center, Inc. (TTCI)’s track substructure program is to better organize and understand which solutions have shown to be successful for each substructure issue and help determine variables that may help contribute or impede the success of the that solution. This paper presents two recent case studies performed by TTCI that show how geosynthetics helped mitigate two common substructure issues.

2. BALLAST POCKET IN RAILROAD EMBANKMENT

2.1 Background

Ballast pockets are a common subgrade issue that often results in higher track geometry degradation rates and therefore requires more track maintenance, leading to lower operational capacity. Remediation techniques that can reduce track geometry degradation at ballast pockets are available if the root cause is identified and addressed (Basye and Wilk, 2019). The underlying mechanism producing ballast pockets is typically a soft-subgrade embankment that progressively deforms vertically and laterally over time (Li et al., 2016; Basye and Wilk, 2019). New ballast is added during track maintenance to restore track geometry elevations. This cycle can develop into a ballast pocket over time. An illustration of a ballast pocket is shown in Figure 1. Ballast pockets can range from a local 2-ft section to over 15-ft deep sections that span hundreds of feet along the track. Ballast pocket deformation also is frequently non-symmetrical under the track, resulting in lateral or vertical deformation that manifests as cross-level and alignment issues.
Soft-subgrade embankments typically result from the initial construction, often over 100 years ago, using poor local subgrade materials because people then did not have the geotechnical knowledge of today and did not anticipate the large increase in loading and traffic volumes. The poor subgrade materials include (but are not limited to) low or high-plasticity clays, organic soils, cinders, and peat, in addition to degradable and uncompactible materials such as tree stumps and large boulders.

Ballast pockets often trap water within the pocket (see Figure 1), commonly referred to as the “bathtub effect”, which keeps the subgrade material at the ballast-subgrade interface at a high-moisture level, which typically has a significantly lower strength than the same material in a dry condition (Li, 2018).

An additional mechanism that may lead to rapid track geometry degradation is degraded ballast and fine material contaminating the ballast in the ballast pocket. Previous studies and historical knowledge show that fine-contaminated ballast can reduce track strength, stiffness, and drainage, especially when wet (Wilk et al., 2019), often leading to higher rates of the settlement in the ballast layer.

Therefore, it is believed that successful remediation of ballast pockets requires reducing or eliminating the soft subgrade embankment deformation. Addressing fine-contaminated ballast near the surface may be an additional need depending on the situation. Remediation strategies to reduce the track geometry degradation at ballast pockets include:

- Installing ballast drains at the low ballast pocket region to remove excess water and increase embankment strength,
- Stabilizing the subgrade embankment material (grout columns for example) to directly increase embankment strength, or
- Stabilizing the upper ballast (geosynthetics or hot-mixed asphalt (HMA)) to improve stress distribution and reduce the stress in the lower embankment. A second benefit of stabilizing the ballast layer is the removal of fouled material which may be detrimental to track geometry, especially when wet (Wilk et al., 2019).
2.2 Case Study #1

The first case study involves the stabilization of a ballast pocket along a 500-ft clay fill embankment on a Norfolk Southern track with geogrid and ballast drains. The embankment is approximately 100-years old and the track carries about 17 MGT of railway traffic per year.

Prior to remediation, maintenance was required about once a week to maintain proper track geometry. Ground Penetrating Radar (GPR) and Cone Penetration Tests (CPT) were used to characterize the site and determine the locations of the deepest ballast pockets. The results from the site characterization showed ballast pockets extending 7-ft to 8-ft deep and soil subgrade resilient modulus values from cone resistance testing between 2.5 to 4.0 ksi, which relates to a moderately soft subgrade (Read et al., 2011).

In 2012, triaxial geogrid and ballast drains were installed along the embankment. The triaxial geogrid was installed 12-inches below the bottom of the tie to ensure the geogrid would not be damaged during tamping and clean ballast was installed to ensure the rock/geogrid interaction was optimized and remove any fine-contaminated ballast near the surface. A photograph of the geogrid installation is shown in Figure 2. The benefits of the geogrid include stabilizing and strengthening the upper ballast region, prevention of downward ballast migration, and improving load distribution to the lower embankment. Two ballast drains were installed at the lowest ballast regions found from the GPR investigation with the purpose of removing water trapped within the ballast pockets, resulting in improvement of the shear strength of the clay embankment.

Figure 2. Photograph of Geogrid Installation
2.3 Results

Immediately after remediation, required surfacing dropped from weekly to yearly. In recent years, it has increased to about five times a year (shown in Figure 3), likely due to asymmetrical lower embankment movement which is causing the geogrid to deform and migrate, reducing its effectiveness. More information about the lower embankment movement can be found in another publication (Li, 2018) and that aspect of the study emphasizes the need to investigate the lower embankment and be aware of transition areas when the geogrid ends. Due to the significant reduction in maintenance requirement, a cost-benefit analysis showed the payback period of the installation was less than a year.

![Figure 3 Surfacing Cycle Rate at Captina](image)

3. SUBGRADE PUMPING IN RAILROAD CUT

3.1 Background

Railroad track located in cuts or below-grade regions often experience increased track geometry degradation resulting from drainage and mud pumping issues. Since the track in these situations are located at a low-point, surrounding water tends to drain into the track and leaves a water table presence only a few inches below the track or even above the track. Draining the excess water near the track is difficult without significant remediation efforts. Figure 4 shows an illustration of a flooded cut.
One problem of a flooded track on a shale subgrade is that the water can weaken the shale subgrade and increase the risk of subgrade particles pumping up into the ballast, contaminating the ballast with fines. This can result in increased settlement, especially when wet (Wilk et al., 2019), so reducing fines in ballast is beneficial to track performance.

Fine-contaminated ballast from subgrade infiltration is overall rare in railroad tracks as past investigations showed that 5% of fines comes from subgrade infiltration, as opposed to 73% from ballast breakdown, 7% from surface infiltration, and 15% from other sources (Selig et al., 1992). However, the study also showed that fines from subgrade infiltration can be prevalent if the subgrade consists of materials that can easily be broken down when saturated (shales or mudstones for example). This suggests that remediation for fine-contaminated ballast will differ depending on how the fines contaminate the ballast. For example, an appropriate solution for subgrade pumping will not be appropriate if the fines are from ballast breakdown or infiltrate from the surface.

3.2 Case Study #2

The second case study involves Norfolk Southern track located within a cut region that is approximately 20 feet lower than the surrounding topography, as shown in Figure 4. The underlying subgrade consists of relatively impermeable silts and shales with very little capacity for drainage. The track is located between two other tracks, further inhibiting the ability to drain the water in the location. The track has an annual tonnage of approximately 20 MGT.

Due to the depressed track orientation, standing water is often observed in ditches along the track and often corresponds with mud pumping, which can increase track degradation rates. Figure 5 shows the cut region, the excess water effects, and mud pumping in the track.
The fines were determined to be from the shale subgrade degrading into fines, then pumping into the surface based on a visual analysis of the fines and trench excavations. In these situations, draining excess water or installing a barrier layer would address the underlying mechanisms producing the fine-contamination because the saturated conditions allows the shale to break up and pump upwards. Due to the difficulty of draining the excess water in this location, barrier methods were explored. Three different geosynthetic test sections and a control section were installed to compare the effectiveness of the geosynthetic systems. The three tested geosynthetics were: Biaxial Geogrid bonded to a geofabric, Tracktex, and 6-inch high Geoweb underlain by geofabric. Each of these geosynthetic systems were tested in an 80-ft section along with a fourth 80-ft control section that had no remedial geosynthetic. Each of the remedial systems is a separation layer consisting of a geofabric or cloth, which provided essential fines migration capabilities. In addition, the biaxial geogrid should provide some resistance to lateral deformation, and the geocell should afford even higher resistance to lateral deformation of the overlying subballast.

The installation occurred in April 2016 and the design details and the test layout are shown in Figure 6.
3.3 Results

To quantify the ability of the geosynthetics to reduce subgrade pumping and fine particles within the ballast, physical samples of shoulder locations were taken in each of the test and control sections. A site visit in July 2018 occurred to assess the ballast and geosynthetic condition after approximately 40 to 50 MGT. The ballast was exposed by backhoe and used to gather samples from different locations and elevations for a gradational assessment. Shoulder cleaning had been done in the control ballast zone in early 2018, so samples were collected immediately adjacent to the control zone in (representative) undisturbed ballast.
The percentage of fine material passing the #4 sieve and the associated ballast fouling index (FI) for each of the geosynthetics showed significant improvement compared to the control samples. The FI provides a practical recognition of the deleterious effects of fines in the ballast, and thus gives the fines percentage extra statistical weight. It is defined as the summation of percentage by weight of ballast material passing the 4.75 mm (No. 4) and the 0.075 mm (No. 200) sieves (Li et al., 2016). For both metrics, the three test locations showed about a 70% reduction in fines within the ballast than the control. This is shown in Figure 7.

![Figure 7: Sampling Results](image)

In addition to the beneficial fine filtering effects afforded by the geofabrics, the presence of reinforcement grids or cells that some of the systems have is designed to provide interlocking support of the ballast, even when fines begin to accumulate near the base. The geogrids and cells also provide tensional resistance to dynamic deformation as they engage, thus developing a stiff matt-like foundation and reducing ballast wear.

4 CONCLUSION

Ballast pockets and flooded cuts represent two subgrade issues that result in increased levels of track maintenance and reduced operational capacity. Ballast pockets develop from the progressive deformation of the subgrade and continual addition of ballast to maintain track geometry. Ballast drains are the common fix, which can have mixed results when used by themselves, but the combination of ballast drains to remove excess water, renewal of the top-12 inches of ballast to remove fine particles, and the additional installation of geogrid 12-inches below the surface to further strength the ballast layer and improve load distribution reduced required maintenance from weekly to yearly for this
particular case study. To optimize the process, investigation methods such as GPR or Cone Penetration Tests can be used to identify the deepest ballast pocket location.

Flooded cuts present an issue because excess water from the surrounding region tends to accumulate in the tracks, which is the low region in the area. The problem can be accentuated if the subgrade is easily broken (shale, for example) and fine particles can upwardly pump into the ballast, contaminating the ballast section. Typical drainage techniques are often not feasible due to limitations on where to redirect the water and surrounding tracks. In the case study, geosynthetics were used as a barrier layer between the ballast and shale subgrade to prevent upward fine pumping. The geosynthetics successfully reduced the fine levels in the ballast, which can reduce the needs for ballast maintenance.

These two case studies show that common subgrade issues can be remediated with geosynthetics if the root cause or causes are properly addressed. TTCI plans to continue investigating these and other situations to better determine root causes and how geosynthetics and other remedial solutions can address these issues.

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REFERENCES

Estimation of Soil-Water Characteristic Curves in Fouled Ballast

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ABSTRACT

Ballast fouling compromises track structure integrity by impeding drainage and altering the ballast strength. These losses can lead to ballast failure and, potentially, derailment. There is a need for improving the fundamental understanding of ballast degradation characteristics so fouled ballast can be identified in the field more rapidly. The objective of this paper is to discuss the unsaturated characteristics of fouled ballast. Unsaturated porous media are materials with both air and water in the pore space between particles. As such, clean and fouled ballast in the field are unsaturated materials. Strength, deformation, and fluid flow behavior in unsaturated media are complex functions of particle properties, degree of saturation, suction, and liquid-air interface properties. Suction of unsaturated geo-materials, such as fouled ballast, directly relates the materials engineering behavior to the moisture content. Soil water characteristic curves (SWCCs) relate geo-material volumetric moisture content to suction potential. In this study, SWCCs were measured from samples of processed fouled ballast collected from two sites using the transient water release and imbibition method. These processed fouled ballast materials contained particles finer than the 3/8 sieve (9.53 mm), thereby representing the fouling material directly and the worst case scenario. One of the processed fouled ballast samples was then mixed with clean ballast to create SWCCs at three fouling percentages. While increased fouling is expected to result in greater water holding ability, the results presented here represent the first time this capacity has been quantified using SWCCs. The long-term goal of this research is to establish and link the suction and strength characteristics of fouled ballast, including the ballast aggregates larger than 3/8 sieve, at different percentages of fouling. A fundamental understanding of the role of fouling material, fouling index, and moisture content will improve the reliability of non-surface geophysical methods, such as ground penetrating radar, for identifying fouled ballast in situ.

1. INTRODUCTION

In the field, ballast is an unsaturated porous media, with both air and water in the particle pore space. Strength, deformation, and fluid flow behavior in unsaturated media are complex functions of material properties (particle and pore size), particle angularity, degree of saturation, pore air and pore water pressure (suction), and liquid-air interface properties (Fredlund et al. 1978; Lu and Likos 2006). Ballast fouling, i.e., the increase in small particles from degradation of the ballast, infiltration of external fine particles such as coal dust, and pumping of the subgrade material over time (Selig and Waters 1994), alters ballast physical properties. Depending on the pore size distribution, or more specifically the degree of fouling in the pore space, ballast water holding capacity changes from its designed free-draining condition and its strength is reduced (e.g., Selig and Waters, 1994; Han and Selig, 1997).
Many researchers have studied the impact of different particle size distribution and fouling material on ballast strength (e.g., Indrarantra et al., 1998; Qian et al., 2016; Ishikowa et al., 2016; Kashani et al., 2018). Because this reduced strength impacts track performance and safety, track inspections are routinely conducted with various visual and nondestructive methods. One promising nondestructive method is ground penetrating radar (GPR) (e.g., Anbazhagan et al., 2016; Leng and Al-Quadi, 2010; Sussman et al., 2003). GPR measurements are dependent on the dielectric constant which is a function of many material properties; however, it is primarily a function of the material volumetric moisture content (Daniels, 2004). Roberts et al. (2009) identified a nearly linear relationship between moisture content and fouling index (i.e., higher moisture content meaning higher fouling); however, they noted that both fouling and moisture increased dielectric properties, at different degrees, and the relative influence due to moisture alone versus fouling could not be determined. In most of the documented shear strength studies and GPR studies, the ballast moisture contents have either been at the extremes (i.e., dry or fully saturated) or evaluated at arbitrary points between these extremes. There is a need to fully understand the role of moisture content versus the degree of fouling on ballast to characterize the strength and deformation as well as to improve GPR reliability in identifying fouled ballast.

Soil Water Characteristic Curves (SWCCs) are well established principles, originally developed in soil physics for soil-water-plant systems (Buckingham, 1907). Soil suction relates the moisture condition of soil to its changing engineering behavior in unsaturated conditions. SWCCs measure the change in suction over a range of water contents between the minimum and maximum water holding capacity of a porous media. Previous researchers have not measured how the water holding capacity (and related suction) changes with increased fouling. The objectives of this study are to measure the SWCC of fouled ballast and identify how the percentage of fouling changes ballast SWCCs. The long-term goal of this research is to understand fouled ballast characteristics (i.e., strength, water holding capacity, and dielectric constant) as a function of the percentage of fouling. Previous researchers have established a relationship between dielectric constant and SWCCs (Sahin et al. 2016). Therefore, understanding the SWCC of fouled ballast is critical to determine the relevant contribution of fouling versus increased moisture on the change in dielectric constant measured with GPR. The inherent uncertainty of current GPR methods to identify fouled ballast will be reduced with a greater understanding of material characteristics as a function of moisture.

2. MATERIALS AND METHODS

2.1 Ballast Samples

The clean ballast used in this study was granitic (igneous intrusive) ballast from a quarry in Oklahoma mixed to meet American Railway Engineering and Maintenance-of-Way Association (AREMA) #4 gradation (AREMA, 2019). The clean ballast is noted as AREMA #4. Two processed fouled ballast samples were collected from mainline track in the Midwest, noted as Samples A and B. These samples were washed and then sieved following ASTM C136 (ASTM 2014) and ASTM D422 (ASTM 2007) and only the material that passed the 3/8 sieve was kept. Note that washing the samples removed the fines (i.e., material that passed the No 200 sieve); however these materials were donated after washing and
sieving the ballast during inspections. Additional fouled ballast samples were made by mixing the AREMA #4 ballast and Sample A to different percentages of fouled material by mass.

All laboratory prepared samples classified as fouled based on the Selig and Waters (1994) fouling index,

\[ F_i = P_{4} + P_{200} \]  

where \( P_{4} \) is the percentage of material by mass that passes the No 4 sieve (4.75 mm) and \( P_{200} \) is the percentage of the material that passed the No 200 sieve (0.075 mm). The AREMA #4 ballast classified as clean and Samples A and B were highly fouled based on the fouling index. The AREMA #4 ballast classified as poorly-graded gravel (GP) according to the Unified Soil Classification System (ASTM D2487). Samples A and B classified as poorly-graded sand with gravel (SP). The 60% and 50% fouled samples classified as well-graded gravel with sand (GW). The 36% fouled sample classified as GP with sand. The sand fractions of the prepared samples and Sample A were primarily coarse and medium sand. The sand fraction of Sample B was primarily medium and fine sand. The particle size distribution curves of all materials are shown in Figure 1. All samples had negligible fines, likely due to the washing of the fouled samples. Material properties, including particle classification based on ASTM D2487 and grain size distribution characteristics, are shown in Table 1.

![Figure 1. Particle size distribution curve for clean ballast, fouled samples (Samples A and B), and laboratory prepared fouled ballast samples. Particle size classification based on ASTM D2487 (2017).](image-url)
Table 1. Material Characteristics

<table>
<thead>
<tr>
<th>Sample</th>
<th>Gravel (%)</th>
<th>Sand (%)</th>
<th>Fines (%)</th>
<th>D_{50} (mm)</th>
<th>d_{10} (mm)</th>
<th>C_u</th>
<th>C_c</th>
<th>FI</th>
</tr>
</thead>
<tbody>
<tr>
<td>AREMA #4</td>
<td>100</td>
<td>0</td>
<td>0</td>
<td>29</td>
<td>17</td>
<td>1.88</td>
<td>0.97</td>
<td>Clean</td>
</tr>
<tr>
<td>Sample A</td>
<td>41</td>
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<td>2</td>
<td>2.8</td>
<td>0.17</td>
<td>28.2</td>
<td>0.69</td>
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</tr>
<tr>
<td>60% Fouled</td>
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<td>1</td>
<td>7.6</td>
<td>0.31</td>
<td>29.7</td>
<td>2.56</td>
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<tr>
<td>50% Fouled</td>
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<td>29</td>
<td>1</td>
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<td>0.39</td>
<td>58.9</td>
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<tr>
<td>Sample B</td>
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<td>70%</td>
<td>10%</td>
<td>0.66</td>
<td>0.075</td>
<td>1.57</td>
<td>1.05</td>
<td>Highly Fouled</td>
</tr>
</tbody>
</table>

2.2 Soil Water Characteristic Curves

SWCC were measured for the processed fouled materials (i.e., Samples A and B) using the transient release and imbibitions method developed by Wayllace and Lu (2012). The K-State device uses a 62 mm diameter cell with a 300 kPa high air entry pressure ceramic disk. No particles were larger than 10.3 cm, or one-sixth the cell diameter, following accepted practice leading to negligible scaling effects (Marachi et al., 1972; ASTM 2016). Samples were oven dried and 170 g were placed in the cell. Light tamping was used in layers to ensure uniform density and consistent sample height. Samples were saturated through the bottom of the cell under a vacuum at the top of the cell. Two pressure increments are applied using axis translation in the transient water release and imbibitions method. The water transient outflow or imbibition from the sample in response to the applied pressure is measured. The outflow is then used as an objective function for an inverse model to determine the hydrologic parameters needed to create a SWCC. The van Genuchten (1980) model was used to create the SWCCs herein as it is widely used in practice and appropriately fits most SWCC data. Equation 2 shows the van Genuchten model, which relates volumetric water content and matric suction in soil, based on Mualem (1976),

\[
\frac{\theta_s - \theta_r}{\theta_s - \theta_f} = \left( \frac{1}{1 + (\alpha \psi)^n} \right)^m
\]  

where \(\theta_s\) is the saturated volumetric water content, \(\theta_r\) is the residual water content, \(\psi\) is matric suction, and \(\alpha\), \(n\), and \(m\) are empirical fitting parameters. Specifically, \(\alpha\) is the inverse of the air entry value in the sample, \(n\) is the pore size distribution parameter, and \(m\) is \((1-1/n)\). The saturated and residual water contents were calculated based on weight-volume relationships and measured be the beginning and end of each TRIM test.

The SWCC from Sample A was then used to calculate the SWCC of laboratory prepared ballast with target percentages of fouling using the Bouwer and Rice (1984) correction. These laboratory prepared ballast samples were created by mixing the AREMA #4 ballast with a target amount of Sample A. Special care was taken to ensure the density of the fouling material in the prepared samples matched the density of Sample A in the transient release and imbibitions method cell. Bareither and Benson (2013) evaluated the Bouwer and Rice (1984) correction for samples with larger (i.e., gravel sized) particles and noted that the dry density of the finer fraction (Sample A) must match the dry density of the finer fraction in the bulk samples (laboratory prepared samples) to use the correction. Although the goal was to create samples in three fouling index levels, it was not possible to create a moderately
fouled or moderately clean sample while maintaining the dry density of the fouling material at the same level as Sample A. The lowest achievable fouling index was 22 (fouled), in the 36% fouled sample. The Bouwer and Rice (1984) correction was required because the current cell diameter cannot accommodate the larger ballast aggregates and still meet the one-sixth particle to cell diameter ratio. A larger cell is currently being to validate the results presented herein. Bower and Rice (1984) established that bulk soil volumetric water content, $\theta_b$, can be computed from the volumetric water content of a finer fraction (i.e., with larger particles excluded) by

$$\theta_b = (1 - \frac{V_p}{V})\theta_f \quad (3)$$

where $V_p$ is the volume fraction of larger particles in the total soil sample and $\theta_f$ is the volumetric water content of the finer soil fraction. In this study, $\theta_f$ is the fouled ballast measured in the transient release and imbibitions method cell and $\theta_b$ is the laboratory prepared sample with AREMA #4 ballast and Sample A. Therefore, theoretical SWCC of the three laboratory prepared samples were created by applying Equation 3 to the measured SWCC of the processed fouled ballast.

3. RESULTS AND DISCUSSION

Figure 2 shows the results from Samples A and B. Figure 2a is the measured and modeled water outflow. The Sample A model very nearly fit the measured data with an R-squared for the regression of predicted versus observed of 0.997 and Sample B had an R-squared of 0.996 indicating good fit in the inversions for the SWCCs shown in Figure 2b. The observed data in Figure 2b were the saturated volumetric water content before applying suction, the measured volumetric water content at the low suction increment, and the measured residual volumetric water content at the high suction increment. These SWCCs represent the worst case scenario, in which only the processed fouled materials (i.e., no aggregates larger than the 3/8 sieve) were measured. For example, as the Sample A matric suction in Figure 2b passed the air entry value, there is a relatively large range of volumetric water contents where the suction minimally increases (from approximately 0.4 to 4 kPa) until the residual water content is reached. This indicated that there was a large range of water holding capacity in the fouled material. Note that the low suction increment (1 kPa) was selected as suction near typical air entry value for poorly graded sand, but because there was an 18% difference in volumetric water content between fully saturated and the volumetric water content at 1 kPa, this initial suction was likely too high. Despite the higher first increment in Sample A, there was still high fit of the measured-modeled outflow and fit between the observed and modeled SWCC. Sample B was run at a low suction increment of 0.5 kPa, however no outflow was observed so the suction was increased to 0.6 kPa. This lead to outflow and was slightly higher than the air entry value as desired. Note that the SWCC shapes are similar, however the water holding capacity of sample B was lower than A. The shape and range of matric suction and volumetric water content are similar to SWCCs for poorly graded sand with gravel in both samples (Yang et al. 2004; Fredlund et al. 2012).
To simulate a fouled SWCC with ballast aggregate, the Sample A SWCC in Figure 2b was corrected following the Bouwer and Rice (1984) and with the fouled ballast samples at the gradations shown in Figure 1 and Table 1. These samples were made with a clean/fouled ballast mixture such that the sample contained 60%, 50%, and 36% fouling material by mass. Note that the inclusion of the clean ballast changed the USCS classification from sand with gravel to gravel with sand. The range of suction and volumetric water content of the SWCCs in Figure 3 are similar to previous studies on gravel (Li and Zhang 2007; Ba et al. 2013). In Figure 3, all corrected SWCCs compared to the Sample A SWCC shifted to the left, as expected from the correction. The 60% fouled sample had a theoretical saturated volumetric water content of 29.6% from the correction and a calculated saturated volumetric water of 30%, based on soil properties, indicating close agreement for the correction. The 50% fouled and 36% fouled samples were similarly within 2% comparing the calculated volumetric water content and the saturated volumetric water content from the correction. As the percentage of fouling material relative to the ballast decreased, so did the saturated volumetric water content and residual water content, though the decrease in residual water content was relatively lower. In the 35% fouled sample, the volumetric water content at 1 kPa suction was 12% whereas the measured volumetric water content at 1 kPa in Sample A was 22%. Also, as the percent fouling increased, the range of volumetric water content increased. For example, in the 36% fouled sample, the volumetric water content at 0.4 kPa was 20% and 3.6% at 4 kPa. In the 60% fouled sample, the volumetric water content was 29% at 0.4 kPa and 5% at 4 kPa. Thus, the 67% increase in fouled materials increased the water holding capacity of the ballast by 46% over the same range of matric suction. The relationship between the increased percentage of fouling and increased saturated volumetric water content supports the findings of Roberts et al. (2009). The results in Figure 3 also highlight that not only does the saturated volumetric water content increase with fouling but the range of water holding capacity (i.e., between residual and saturated) increases with fouling.
The SWCCs in Figure 2 highlight how ballast samples that classify the same according to the USCS system and have the same fouling index classification may still have different water holding capacity. Figure 3 shows how the percentage of ballast fouling impacts ballast water holding capacity (and suction). The samples with only fouled ballast materials had a higher residual water content, meaning more water is trapped in the pore space at the end of drying compared to samples with ballast aggregate. Note that there is extremely limited water holding capacity in clean ballast as it is intended to be free draining. These results highlight that as the percentage of fouling increases, so does the water holding capacity of the ballast and over a wide range of volumetric water contents. Because GPR is primarily controlled by the dielectric constant, this wide band of volumetric water content over a relatively low range of suctions may explain why GPR results can be highly variable with in situ moisture conditions. In other words, there may be a wide range of potential moisture contents within very similar in situ ballast conditions that are impacting the GPR interpretations.

Furthermore, it is well established that fouled ballast and the associated reduced drainage hinders the transfer of the load from the wheels to the supporting substructure (Armstrong 2008). This rapidly deteriorates the rail support structure and increases the likelihood of failure. In extreme cases, when not remediated, fouled ballast can lead to train derailment. However, there has been much anecdotal evidence gathered in talking with field engineers that there are areas that continually become fouled, but show no deterioration of the track substructure. In order for a method to nondestructively identify all fouled ballast successfully, the characteristics of the fouled material must be understood. Because suction directly relates the water content of a material to its engineering behaviors, the next steps of this research will be to measure the fouled ballast strength, at target volumetric water contents identified from the measured SWCCs between residual and saturated volumetric water contents. The
strength testing will contribute to the fundamental understanding of the characteristics of fouled ballast as a function of water content and suction to fully understand the material properties and how this relates to in situ behavior.

The theoretical SWCC of clean ballast (i.e., with no fouling) would be a nearly flat line with very small difference in suction at the saturated and residual volumetric water contents. This study was limited because it was not possible to make samples with lower degrees of fouling at the same fouling material density with the current experimental set up, therefore the next step of this research is to increase the size of the cell to measure the SWCC of fouled ballast. This will validate the corrected curves presented herein and allow for direct measurements at lower degrees of fouling. The findings herein are also limited by the number of samples and grain size distributions. The SWCCs of additional fouled materials with more fines and with different parent fouling materials, such as coal dust, rather than the breakdown of ballast, are also currently being measured.

4. CONCLUSIONS

The findings presented in this paper show that increasing the percent fouling increases the water holding capacity of ballast with relatively small changes in ballast suction. The SWCCs of two fouled ballast samples were measured and theoretical SWCC of ballast at different percentages of fouling were created. There is very limited information regarding the SWCC of ballast, specifically related to the degree of fouling. Understanding the SWCC of fouled ballast is critical to determine the relevant contribution of the fouling material versus the contribution of moisture content on the dielectric constant. By understanding these variables, the uncertainty of nondestructive methods, such as GPR, to identify fouled ballast may be reduced, ultimately improving the effectiveness of track performance and safety inspections.

5. ACKNOWLEDGEMENTS

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REFERENCES


Unsaturated Analysis of Water Flow in Granular Layers of Inundated Pavements

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**ABSTRACT**

The occurrence of extreme rainfall events and heavy floods has increased in recent years due to the global warming and climate changes. Transportation infrastructure systems, such as highways, bridges, embankments, etc. are affected by these extreme environmental events. These effects, however, may not be assessed using regular analysis/design tools. In the particular case of pavement structures, the ingress of water decreases the resilient modulus of aggregate layers, and hence, results in accelerated rate of structural deterioration. This research aims at evaluating the hydraulic behavior of inundated pavement structures, which is the first step towards better understanding of the impact of flooding on the pavement’s remaining life. A finite difference scheme was utilized to model the variations of the degree of saturation within the granular layers of the pavement with time. To achieve more realistic results, the principles of unsaturated flow in porous media were adopted. The basic characteristics of granular material, e.g. gradations, were considered using well-developed correlations extracted from the literature so that the parameters required for a more-sophisticated unsaturated analysis could be estimated based on basic material properties. A sensitivity analysis was carried out to assess the influence of these material properties on the hydraulic response of pavements to inundation. The results of the proposed numerical models can be translated into resilient modulus profiles which, in turn, may be used for a subsequent structural analysis. Such structural analysis is favorable for highway agencies and state DOTs by enabling them to update their pavement management systems for rehabilitation or repair of flooded sections.

1. **INTRODUCTION**

As shown in Fig. 1, the number of severe flooding events and heavy rainfalls has increased in the recent past across the U.S. and around the world due to the climate change and global warming (EPA, 2016). The rehabilitation of the transportation infrastructure that have been damaged due to such extreme events requires billions of dollars. Pavement structures, in particular, are vulnerable to the two types of flooding, namely river and flash flooding. Sustained water within the pavement structure can adversely influence the bearing capacity of the pavement and can result in its premature failure. In the case of river flooding, which is accompanied by prolonged inundation, significant ingress of water into the pavement is almost inevitable. The detrimental effects due to the presence of water in pavement structures include the reduction of shear strength and resilient modulus of the soil layers, asphalt stripping in flexible pavements, pumping effect in rigid pavements, migration of fines into drainable...
layers, frost/heave and thawing in cold regions, and swelling in expansive soils (Huang, 2004). Premature pavement failures can impose unforeseen maintenance/rehabilitation costs to highway agencies. They also may result in serious safety hazards to the users.

Several researchers have reported on the structural damage and premature failures of inundated pavements in the literature. For instance, Gaspard et al. (2007) investigated the influences of hurricanes Katrina and Rita on the transportation infrastructure of Louisiana. They tested several submerged pavement sections in on-going construction projects. They found pavements were damaged because the subgrade became very weak. They also reported that the damages were more severe in asphalt pavements compared to the concrete ones. Sulhana et al. (2016) conducted a similar study in Queensland, Australia to assess the post-flood behavior of inundated flexible pavements. They performed Falling Weight Deflectometer (FWD) tests on flooded as well non-flooded sections to demonstrate the relative damage of pavements due to the submergence. They suggested to assess the structural damage of a flooded section in a form of performance model (i.e. structural number versus time). They proposed an empirical equation to estimate the reduction of the structural number of the flooded sections as suggested by Austroads (2010) to assess the immediate impacts of flooding on the pavements and the associated costs of future rehabilitation.

Unsaturated flow within a conventional layered pavement in the aftermath of flood-related inundation is numerically modeled in the current study. A brief overview of the theory of unsaturated flow is presented in section 2. Section 3 introduces the software and procedure utilized for modeling the post-flood internal flow. A parametric study on the impact of key parameters on the post-flood response of pavement is described in section 4. In addition, some recommendations are made based on the results of this section. The conclusions of the paper are included in section 5.

2. THEORETICAL BACKGROUND

The aggregate layers of pavements undergo continuous changes of degree of saturation as a result of the hydrological cycles, such precipitation, infiltration and evaporation, results in of granular layers above the free ground water table level. Although the granular base layer and its underlying natural
Subgrade soil experience some saturation periods, they should be generally regarded as partially saturated for more realistic analysis. The key characteristic of the material required for unsaturated flow analysis is the Soil Water Characteristic Curve (SWCC) which essentially expresses a measure of water content against soil’s negative pressure, or suction (Fredlund et al., 2012).

Empirical and semi-empirical equations have been proposed for SWCC (e.g. Brooks and Corey, 1964; and Van Genuchten, 1980; Fredlund and Xing, 1994). Leong and Rahardjo (1997) compared the most popular forms of SWCC and recommended the equation proposed by Fredlund and Xing (1994), which is:

\[
\theta_w = \left[1 - \frac{\ln \left(1 + \frac{h}{h_r} \right)}{\ln \left(1 + \frac{h}{h_r} \right) \left[ \ln \left( \exp (1 + \frac{h}{a}) \right) \right]^n} \right] \theta_{sat}
\]

(1)

where, \( \theta_w \) is volumetric moisture content; \( \theta_{sat} \) is saturated volumetric moisture content; \( a, b, c, \) and \( h_r \) are fitting parameters.

The most robust theoretical framework for characterization of water movement in unsaturated zone is the partial differential equation of Richards. Owing to its sound physical basis, the Richards’ equation (RE) has been widely used in fundamental research and scenario analyses of water flow in variably saturated soil. Several numerical solutions have been proposed for the classical form of RE. However, the RE is difficult to solve because of its parabolic form and the nonlinearity of soil hydraulic functions. In addition, abrupt changes in moisture conditions near the soil surface cause steep wetting/drying fronts, posing convergence issues (van Dam and Feddes, 2000).

3. DEVELOPMENT OF MODEL

VS2DI software, developed by the US Geological Survey (USGS), can be used for modeling one- and two-dimensional flow in variably saturated media (Hsieh et al., 1999). VS2DI is a finite difference solution to the RE for water movement under various initial and boundary conditions. Characteristics relations between the moisture content, hydraulic heads and relative hydraulic conductivity may be represented by a number of predefined models or by tabular data points. VS2DI graphical user interface (GUI) consists of a preprocessor as well as a postprocessor. The preprocessor is used to define the model’s domain, hydraulic and transport properties, initial and boundary conditions, grid spacing, and other model specifications. Once all required data are given to the preprocessor, the simulation results are displayed for each time step through a simple animation representation by the postprocessor. Simulation results can be displayed as contours of pressure head, moisture content, saturation, velocity or flux.

Several researchers have successfully modeled the unsaturated flow in pavement layers using VS2DI. For instance, Stormont and Zhou (2005) used VS2DI to study the performance of a pavement system with edge drain subject to surface infiltration. They acknowledged the uncertainties associated with such a numerical analysis due to a lack of confidence on unsaturated hydraulic properties of the material. Yet, they observed that the numerical results were generally consistent with the field observations.
A three-layer pavement section consisted of 3-in. of HMA, 12-in. of unbound aggregate base and natural subgrade was modeled using VS2DI. The initial and boundary conditions should be adequately considered in order to realistically simulate the post-flood water regimen within the pavement layers. As the initial condition, the pavement layers were assumed to be fully saturated prior to the analysis. In addition, an initial elevation of water on top of the pavement, which was equivalent to initial heads, was considered as shown in Figure 2.

![Figure 2. The geometry of the modeled pavement structure.](image)

A number of material properties, such as porosity, SWCC, and saturated permeability ($k_{\text{sat}}$), were required for that unsaturated analysis. Since direct measurement of some of these properties (e.g., SWCC) is difficult and expensive, available correlations can be utilized to estimate indirectly such properties based on more basic ones, e.g., gradation and plasticity. Zapata (1999) proposed correlations for the estimation of SWCC of coarse- and fine-grained geomaterials with regard to $D_{60}$ and weighted plasticity (wPI), respectively. On the other hand, several equations have been proposed for estimation $k_{\text{sat}}$. The equations adopted by AASHTO’s mechanistic empirical design guide, MEPDG, (ARA, 2004) for aggregate base materials is a function of $D_{60}$ only. However, as Table 1 suggests, this equation may result in significant error of prediction when compared to measured values. The equation of Moulton (1980) expresses $k_{\text{sat}}$ as a function of effective diameter $D_{10}$, fine percentage of the material (bP200), and porosity. This equation which makes relatively more reliable predictions was utilized for the parametric study explained in the following section.

4. SENSITIVITY ANALYSIS

This section presents the results of a parametric study on the post-flood hydraulic behavior of a pavement section (Fig. 2) analyzed using unsaturated flow principles. For each time step, the hydraulic response was represented by saturation profile, due to variations of one parameter when the other parameters are set to nominal (constant) values. Figure 3 shows the variations of degree of saturation for different gradations of the base material. The excess water dissipates more rapidly over time when the base is composed of coarser aggregates, i.e. greater $D_{10}$, and less fines, i.e. lower P200. On the other hand, when the unbound aggregate material is considerably fine, i.e. $D_{10}<0.3$ mm, the base layer remains over 90% saturated for several days, over 28 days in this case, after flooding. Consequently, the
structural capacity of the base layer decreases due to reduction of its resilient modulus. This is synonymous with severer distresses and premature failure of the pavement.

Table 1. Measured and estimated values of saturated permeability, $k_{sat}$ (m/s)

<table>
<thead>
<tr>
<th>AGGREGATE MATERIAL</th>
<th>MEASURED (Liang, 2007)</th>
<th>MOULTON’S EQ.</th>
<th>MEPDG EQ.</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 57</td>
<td>9.37E-02</td>
<td>4.42E-02</td>
<td>1.44E-07</td>
</tr>
<tr>
<td>304 Fine</td>
<td>7.30E-04</td>
<td>1.50E-07</td>
<td>1.47E-06</td>
</tr>
<tr>
<td>304 Medium</td>
<td>5.00E-03</td>
<td>4.09E-07</td>
<td>4.50E-05</td>
</tr>
<tr>
<td>304 Coarse</td>
<td>1.92E-02</td>
<td>1.29E-02</td>
<td>2.90E-04</td>
</tr>
<tr>
<td>307-NJ Fine</td>
<td>7.88E-03</td>
<td>6.86E-05</td>
<td>3.30E-06</td>
</tr>
<tr>
<td>307-NJ Medium</td>
<td>1.35E-02</td>
<td>1.69E-04</td>
<td>1.59E-05</td>
</tr>
<tr>
<td>307-NJ Coarse</td>
<td>2.80E-02</td>
<td>2.91E-02</td>
<td>6.34E-05</td>
</tr>
<tr>
<td>307-IA Fine</td>
<td>3.08E-03</td>
<td>4.24E-06</td>
<td>6.82E-06</td>
</tr>
<tr>
<td>307-IA Medium</td>
<td>8.03E-03</td>
<td>5.42E-04</td>
<td>3.79E-05</td>
</tr>
<tr>
<td>307-IA Coarse</td>
<td>2.89E-02</td>
<td>2.25E-02</td>
<td>1.08E-04</td>
</tr>
<tr>
<td>307-CE Fine</td>
<td>9.37E-03</td>
<td>5.36E-04</td>
<td>1.48E-05</td>
</tr>
<tr>
<td>307-CE Medium</td>
<td>1.31E-02</td>
<td>4.62E-03</td>
<td>8.05E-05</td>
</tr>
<tr>
<td>307-CE Coarse</td>
<td>3.07E-02</td>
<td>5.60E-02</td>
<td>2.91E-04</td>
</tr>
<tr>
<td>MAE (m/s)</td>
<td>-</td>
<td>0.0110</td>
<td>0.0200</td>
</tr>
<tr>
<td>Mean Percentage Error (%)</td>
<td>-</td>
<td>72.6</td>
<td>99.6</td>
</tr>
</tbody>
</table>

The impact of subgrade plasticity on the post flood behavior of the pavement, characterized as saturation of the base layer, is depicted in Figure 4. Although the base layer’s degree of saturation slightly changes for different plasticity of the subgrade, especially between days of 15 to 25, it is almost the same for different subgrade types at the end of the analysis. This may be attributed to the function of permeable base which makes the role of subgrade insignificant by providing a drainage path through the base layer. According to this observation, it is recommended to use permeable base when there is no control on the subgrade material, or it is of high plasticity, to minimize the distresses associated with post-flood over-saturation period.
Figure 3. The influence of the gradation of base layer’s material, effective diameter (D10) and percentage of fines (bP200) on the variation of degree of saturation.

![Graph showing the influence of D10 and bP200 on degree of saturation over time.]

Figure 4. The influence of subgrade’s weighted plasticity (wPI) on the variations of degree of saturation.

![Graph showing the influence of wPI on degree of saturation over time.]

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Figure 5 shows the variations of base layer’s degree of saturation for different initial heights of water on top of the pavement. The initial height of water seems to be the least significant parameter among those investigated herein. The influence of base layer’s permeability anisotropy on the post-flood variations of saturation is illustrated in Figure 6. More-compacted base layers (i.e. lower $k_v/k_h$) have higher levels of saturation over time, which in turn will result in higher structural damage. This finding is in line with the previous observations and highlights the importance of permeable base layers in post-flood response of pavement structures. This study suggests that consideration of additional measures of compaction may be necessary in order to meet permeability requirements for flood-resilient pavements especially in the areas with more frequent and heavy rainfalls.

![Figure 5. The influence of initial height of water ($h_w$) on the variations of degree of saturation.](image1)

![Figure 6. The influence of base layer’s permeability anisotropy on the variations of degree of saturation.](image2)
5. CONCLUSION

In this paper, post-flood hydraulic behavior of a flexible pavement with unbound aggregate base layer was studied by means of a freeware finite difference code. The pavement was modeled based on the principles of unsaturated flow in porous media for more realistic results. In addition, experimental correlations were utilized for estimation of required material properties. Subsequently, a parametric study was carried out to investigate the impact of key properties of the material, such as gradation and anisotropy, on the post-flood response. It is to be mentioned that severer distresses and even premature failure might occur if the granular layers, particularly the unbound aggregate base, undergo prolonged periods of high saturation. Accordingly, this study revealed that new measures, for instance for compaction control or base layer gradation, may be required to design and construct flood-resilient pavement structures.

REFERENCES


Study on Mechanical Properties of 35T-40T Heavy Haul Railway Lightweight Foamed Concrete Subgrade

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ABSTRACT

Light Weight Foamed Concrete (LWFC) is being investigated by the Railway Engineering Research Institute (RERI) in China as a potential subgrade foundation at bridge transition zones. It has the characteristics of low density, high stiffness, easy access to material that is convenient to construct. The goals of the LWFC are to increase the quality of subgrade structure and improve the stress distribution range and transmission depth, thus solving the problems of difficult compaction and differential settlement of transition section. To understand the LWFC subgrade performance in a 30t-40t heavy-haul railway environment, three types of tests were completed by RERI: indoor mechanical test, full-scale model test, and an in-situ construction test at the Transportation Technology Center (TTC). First, laboratory tests verified that LWFC with wet density of 800 kg/m$^3$ has mechanical properties that are well above the anticipated subgrade stress levels. Second, through full-scale (1:1 scale, which includes the foundation, LWFC, sub-ballast and ballast layers) indoor model testing of the LWFC subgrade structure at RERI, by using mega dynamic pressure machine to simulate the 30t-40t axle load condition, to find the dynamic response characteristics and pressure distribution of each subgrade layer, also to test both stability and serviceability of LWFC subgrade. Finally, the in-situ construction at TTC was completed and is currently being tested, which preliminary analysis has shown that the LWFC structure has reasonably well structural and mechanical stability.

1. INTRODUCTION

Subgrade, as a geotechnical structure, due to its simple and reliable structure, convenient construction and wide selection of filling materials (ZHANG Q.L. et al. 2008, ZHANG Q.L. et al. 2016), which has been widely used in railway construction. Compared with the general axle load trains, the subgrade for the heavy haul railway will face two technical problems. First, the increase of axle load will cause the increase of subgrade load, and the superposition effect of subgrade internal stress will be obvious, which required more strength and transmission depth to maintain the stability and workability of the subgrade. Second, the operation of long marshalling trains will increase the number of continuous action of subgrade, which will make it difficult for the accumulated pore pressure in the subgrade soil to dissipate in time, cause the reduction of the effective stress and the strength of the subgrade (YE Y.S et al. 2012, ZHANG W.T 2017). In addition, the design life of China's heavy-haul railway subgrade is generally 100 years, the requirements of subgrade durability and structural form rationality become much stricter.
The use of LWFC subgrade can effectively solve the problem of difficult compaction and differential settlement in the transition section (CHEN Z.P. et al. 2004), since the LWFC has the characteristics of low density, well stiffness, and convenient construction. In order to comprehensively understanding the structural characteristics of LWFC heavy-haul railway subgrade (YANG C.F. et al. 2016), this paper will focus on the study of the stress-strain relationship and the dynamic response of LWFC subgrade structure under the action of 35t-40t axle load by using in-door mechanics test, full-scale dynamic test and in-situ construction test.

2. INDOOR MECHANICAL TEST OF LWFC

2.1 Fundamental characteristics of LWFC

LWFC is a kind of light material which made from the aqueous solution foaming agent, mixed and stirred with cement-based cementitious material and optional aggregate in a certain proportion (M.R. Jones. et al. 2005), which has the characteristics of adjustable density, light weight and simple construction. Therefore, the LWFC has been widely and successfully used in both railway and highway engineering construction. In this test the water-cement ratio of LWFC specimen is 0.5, the wet density is between 400kg/m$^3$ to 1000kg/m$^3$, and the regression curve of the LWFC compressive strength (28d) is obtained and shown in Figure 1.

![Figure 1. The relationship between compressive strength and density of LWFC](image1)

![Figure 2. The relationship between relative compressive strength and relative density of LWFC](image2)
It can be seen from Figure 1, there is a well correlation between the compressive strength and the wet density of LWFC. The correlation coefficient of the regression curve $R^2=0.9792$, which indicates that the compressive strength of LWFC is largely dependent on the wet density. The 28d compressive strength of LWFC (600 Kg/m$^3$ in wet density) is greater than 2MPa, which basically meet the requirements of subgrade filling material. By the consideration of both reliability and serviceability, the LWFC with wet density as 800 Kg/m$^3$ are recommended to be used for the subgrade of heavy haul railway. In order to give LWFC a certain universality, the relationship between the relative strength and the relative density has been obtained and shown in Figure 2 and the relationship can be written as Eq.(1):

$$\frac{\sigma_f}{\sigma_p} = 0.7241\left(\frac{\rho_f}{\rho_p}\right)^{432}$$

(1)

Where $\sigma_f$ is the strength of LWFC, $\sigma_p$ is the strength of the slurry, $\rho_f$ is the wet density of the LWFC, $\rho_p$ is the density of the slurry.

Figure 2 shows that the strength of the LWFC is proportional to the strength of the slurry. The relative strength in the Eq.(1) is expressed as the strength of the LWFC divided by the strength of the slurry, this can minimize and eliminate the effect of differences in slurry strength. According to the Eq.(1), the strength of LWFC under any wet density can be calculated, when the mixture ratio, strength and density of the slurry are known. Moreover, the strength of LWFC can be adjusted according to the Eq.(1) to the same density, when the density of the LWFC produced by different batches are different.

2.2 Compressive strength test

Figure 3 shows the compressive strength test of the LWFC. The wet density of LWFC specimen is 800kg/m$^3$ with geometry size as 100mm*100mm*100mm. The external load was continuously and uniformly applied to the test specimen until the specimen was close to failure. The process recorded the failure load when the specimen was destroyed. The test process is showed in Figure 3.

![Figure 3. The compressive strength test of LWFC](image)

The compressive strength of the concrete cube should be calculated accurately to 0.01mpa with Eq.(2):

$$f_{cc} = \frac{F}{A}$$

(2)

Where $f_{cc}$ is the compressive strength of concrete cube specimen (MPa), $F$ is the failure load of specimen (N), $A$ is the pressure area of the specimen (mm$^2$).
In order to satisfy the requirements of both Chinese National Design Code and Technical Code of Cast-In-Situ LWFC for Heavy-Haul Railway Construction, the compressive strength of the LWFC specimen should be measured and satisfied the standard requirements after curing for 7d and 28d respectively. In this test, the compressive strength of LWFC at 7d has been measured as 1.55 MPa, which is greater than 0.8 MPa, and the compressive strength of LWFC at 28d was 2.63 MPa which is greater than 1.5 MPa.

2.3 Freeze-thaw cycle test

Specimen with different wet densities were prepared in this laboratory freeze-thaw cycle test. The size of specimen was 100mm*100mm*100mm, with the wet density between 400kg/m$^3$ to 1000kg/m$^3$. After curing for 28 days, the unconfined compressive strength test was conducted at first. Then the compressive strength has been measured after 25 freeze-thaw cycles, shown in Figure 4. As the wet density increases, the compressive strength of the LWFC gradually increases. When the wet density changes from 400kg/m$^3$ to 1000kg/m$^3$, the compressive strength increases from 0.37MPa to 6.79MPa.

![Figure 4. The compressive strength of LWFC with different wet density](image)

The compressive strength loss of LWFC with different wet density before and after 25 freeze-thaw cycles is shown in the Figure 5. The maximum loss rate of compressive strength is 15% for the specimen with wet density of 800kg/m$^3$. The maximum loss rate of compressive strength of 900kg/m$^3$ and 1000kg/m$^3$ after 25 freeze-thaw cycles is 4%. The 400kg/m$^3$~600kg/m$^3$ specimen has a large mass loss after 15 freeze-thaw cycles, hence it is impossible to carry out the compressive strength test. Therefore, the wet density of the LWFC should be greater than 800kg/m$^3$ to ensure the strength and the serviceability of LWFC subgrade can maintain in a well acceptable status.
2.4 Dry-wet cycle test

The influence of moisture on the strength and other mechanical properties of LWFC is obvious. In practical engineering, LWFC is greatly affected by the dry-wet cycle. With the same specimen density and size as freeze-thaw cycle test. Again, the unconfined 28d compressive strength should be checked firstly. Then, the compressive strength was measured after 25 dry-wet cycles (WANG L.J. et al. 2017).

Figure 6 shows the loss of compressive strength of LWFC after 25 dry-wet cycles with different wet densities. The maximum compressive strength loss of LWFC after 25 dry-wet cycles was about 15%, which is reasonable and acceptable for the design requirements. Therefore, LWFC has strong resistance to the dry-wet cycle. The overall physical properties and durability of the LWFC with a wet density of 800kg/m$^3$ can be safely used as a subgrade filling material for heavy haul railway.
3. FULL-SCALE MODEL TEST OF LWFC

3.1 Test objective

This full-scale model test will mainly focus on the dynamic response characteristics and pressure distribution of LWFC subgrade structure under the action of 35t-40t dynamic load. Figure 7 shows the LWFC subgrade structure for the full-scale test which used for the heavy-haul railway. The thickness of LWFC was 1m in this test, which replace partial bottom layer of original subgrade. The ballast of 0.45m thick is overlaid on the graded gravel layer (0.2m). Then 0.2m sand cushion layer (with composite geomembrane) has been laid which can improve the overall permeability properties of LWFC subgrade structure (FENG W. et al. 2015, LV W.Q. et al. 2016).

![Figure 7. Layout of sensors in this full-scale model test machine](image)

In order to simulate the train dynamic load in this LWFC subgrade structure full-scale model test. The mega pulse fatigue test machine would be used to add the dynamic load on the rail track (Figure 8).
3.2 Test programme

In order to test the dynamic deformation resistance and mechanical stability of LWFC subgrade structure, the settlement observation marks with Linear Variable Differential Transformer (LVDT), the dynamic pressure cells and the pressure distribution sensors and will be used. The layout of all the dynamic sensors is shown in Figure 7. The basic parameters of LWFC subgrade structure are shown in Table 1.

<table>
<thead>
<tr>
<th>Subgrade layer</th>
<th>Density (g/cm$^3$)</th>
<th>EVD (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LWFC</td>
<td>800</td>
<td>152</td>
</tr>
<tr>
<td>Graded gravel</td>
<td>2.1</td>
<td>44.7</td>
</tr>
</tbody>
</table>

The maximum dynamic test force of the mage pulse fatigue test machine is 500kN, and the frequency of dynamic load is 5Hz. The geometric size of the model test tank is 6.8m*5m*3.3m. In order to determine the stability and reliability of the LWFC subgrade structure, the dynamic stress with 30t, 35t, 40t, 45t axle load was applied to the test structure respectively (under natural drying conditions). Then, both 35t and 40t axle loads with 2 million times were applied to the LWFC subgrade structure under the water immersion condition respectively, which verify the mechanical stability and impermeability of the structure. The loading scheme of full-scale model test is shown in Table 2.

<table>
<thead>
<tr>
<th>Water immersion situation</th>
<th>Loading level /kN</th>
<th>Loading cycles /ten thousand times</th>
</tr>
</thead>
<tbody>
<tr>
<td>Naturally dry</td>
<td>300</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>360</td>
</tr>
<tr>
<td></td>
<td>450</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>200</td>
</tr>
<tr>
<td>Fully immersion</td>
<td>400</td>
<td>200</td>
</tr>
</tbody>
</table>
3.3 Pressure distribution in vertical direction

Figure 9(a) and Figure 9(b) show the vertical pressure of each layer under the 30t-40t axle load condition. It can be seen that the vertical pressure of each layer presents a linear rule with the increase of external load. Under the same load condition, the vertical pressures are highly depends on the depth and the position of the point. The vertical pressure at the same location decreases by 35% with the depth increases by 0.4m, under the dynamic load of 350kN. Moreover, the vertical pressure decreases by 30% with the depth increases by 0.4m under the 400kN dynamic load.

a) LWFC surface

![Graph showing pressure distribution in LWFC surface](image)

b) Graded gravel surface

![Graph showing pressure distribution in graded gravel surface](image)

Figure 9. Vertical pressure of each layer under 30t-40t axle load
3.4 Deformation status of dynamic load test

Figure 10 shows the dynamic deformation at the surface of ballast, graded gravel and LWFC under the condition of 300kN-450kN dynamic load. It can be seen that the overall dynamic deformation of LWFC is less than 0.1mm, which is relatively low. The LWFC subgrade has reasonably well dynamic stability under the dynamic load of 450kN.

Figure 11 shows the dynamic deformation at the surface of both graded gravel and LWFC, under the condition of naturally dry and fully immersion. It shown that the LWFC subgrade has well stable mechanical properties and well serviceability performance, since the overall dynamic deformation is less than 0.075mm.

![Figure 10: The dynamic deformation status under 30t-45t axle load](image)

![Figure 11: The dynamic deformation of each layer under dynamic load conditions](image)
4. IN-SITU CONSTRUCTION TEST OF LWFC

4.1 TEST OBJECTIVE

The LWFC has the characteristics of low density and convenient construction. It can be used as a transition section structure to effectively solve the problem of partially compaction. The site for this project is located at the High Tonnage Loop (HTL), at the west approach of the East Steel Bridge (ESB), shown in Figure 12.

![Figure 12. East Steel Bridge (ESB) location of the Bridge approach subgrade test.](image)

The transition section is divided into three steps with a height change of 0.8m. The thinnest part of lightweight concrete should be no less than 0.8m. Metal mesh is set 0.1m away from the top surface of the LWFC. Deformation joints with a width of 2cm and 0.2m depth will be made at each step. The whole structure of the LWFC subgrade is completely wrapped with HDPE film, and the sand cushion is laid on the top surface of the LWFC. The wet density of LWFC should be controlled around 800 kg/m³, the 7days compressive strength should be greater than 0.8MPa and the 28days compressive strength should be greater than 1.5MPa. The transverse-section and the vertical-section of LWFC subgrade structure have been shown in Figure 13 and Figure 14.

![Figure 13. LWFC subgrade transverse-section](image)
4.2 Test content

The main test contents of this test include earth pressure test, dynamic deformation test and moisture content test. The surface cracks and erosion status of LWFC should be observed and measured before and after the test. The deformation of LWFC subgrade and the differential settlement of transition section should be measured for each 10MGT. The sensors arrangement have been detailed shown from Figure 15 to Figure 18.
4.3 Track Settlement

The test vehicle consists of three locomotives and 110 carriages, each weighing 35.4 tons. The maximum operation speed is 64 km/h and the accumulated driving weight for each night is about 2 MGT. Figure 19 shows the settlement and deformation of the track structure (rail) along the longitudinal direction, the overall settlement of test section tends to be stable with no significant increase after 40MGT (101.5 MGT has been completed so far). Starting from the initial settlement of track structure: the total settlement of track (rail) is 28mm (after 100 MGT), and the settlement of subballast is about 9.5mm. Figure 20 shows the average stratified settlement of each layer after passing the weight of 100MGT, in which the compressive deformation of LWFC subgrade layer is about 4.6mm. Overall, the performance was well and in accord with expectations.
4.4 Dynamic Testing and Results

4.4.1 Earth pressure

Figure 21 and Figure 22 show the test results of the earth pressure cells in section 2 and section 5, wherein the average surface pressure of LWFC is basically stable at about 60kPa, and the surface pressure of graded gravel is about 110-140kpa, which is related to the driving speed and axle load, hence the service performance is basically well.
4.4.2 Dynamic displacement

Figure 23 and Figure 24 show the dynamic displacement test of section 2 and section 5, in which the dynamic displacement of graded gravel surface is about 0.6mm, and the dynamic displacement of LWFC surface is about 0.4mm. Since the position of section 2 has two steps, where the dynamic displacement is smaller than that of section 5 (single step).
Figure 23. Dynamic displacement at section 2

Figure 24. Dynamic displacement at section 5

4.4.3 Moisture

Figure 25 and Figure 26 show that the water content on the surface of LWFC increases slightly compared with that on the bottom, and the water content in section 1 is slightly larger than that in section 2, but there is no obvious abnormality, and the overall performance is basically stable.

Figure 25. Moisture status at LWFC bottom
5 CONCLUSIONS

The LWFC has stable mechanical properties with relatively light weight and small compressive deformation, which is suitable for construction of special railway subgrade section and bridge transition section, where the deformation requirements are strict. It can effectively reduce the differential settlement between different structures and provide sufficient bearing capacity.

For the LWFC with a wet density of 800kg/m$^3$, the laboratory test results show that it has high compressive strength and well mechanical stability, which can be used for the new design of heavy-haul subgrade structure.

The structure of LWFC subgrade has well impermeability and water stability, which can avoid the subgrade diseases caused by the influence of water. The results show that the overall dynamic deformation of LWFC under the fully immersion condition is still less than 0.075mm, which indicates that the structure of the LWFC subgrade has relatively high stability to resist deformation and excellent serviceability performance for the dynamic load.

The LWFC bridge transition section structure with wet density of 800kg/m$^3$ is designed and tested with in-situ construction test. The results shown that the LWFC subgrade has reasonably well workability and stability to resistant the heavy axle load, the structure can also reduce the differential settlement of bridge transition section and redistributed the stress uniformly.

6 ACKNOWLEDGMENTS

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7 REFERENCES


