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In Situ Modulus Measurement Using Automated Plate Load Testing for State Wide Mechanistic-Empirical Design Calibration

Introduction

To develop the AASHTOWare Pavement ME Design™ input data needed for typical Iowa foundation layers, the Iowa Department of Transportation selected the Automated Plate Load Testing (APLT) to conduct a state-wide field study. An experimental plan was developed in collaboration with the Iowa Department of Transportation (DOT) pavement design and construction engineering teams.

Objectives and Scope

- Review pertinent project location information provided by the Iowa DOT to select 10-12 project locations that cover a wide range of soil conditions.
- Mobilizing APLT to each project site to conduct field testing.
- Conducting cyclic and static APLTs at each project location to generate a statistically robust dataset.
- Conducting dynamic cone penetrometer (DCP) test at each APLT location to obtain the layer thickness profile for backcalculation analysis.
- Obtaining and conducting the necessary laboratory tests for soil characterization/ classification.
- Developing a data report for each project site with a summary memo for Iowa DOT review.
- Developing a final report and presentation to Iowa DOT on key findings.
- Developing a technical brief.

Experimental Plan

A total of 10 project sites were selected that covered common unbound foundation layer cross-sections used in Iowa highways. Projects consisted of different subbase types (granular subbase and modified subbase), different subbase materials (crushed limestone and recycled concrete aggregate), different subgrade types (select subgrade and embankment cut/fill subgrade).

The goal at each site was to perform cyclic APLTs to determine resilient modulus (M_r) using a 12 in. diameter loading plate and perform static APLTs to determine modulus of subgrade reaction *k*-value using a 30 in. diameter loading plate (Figure 1).

For each project site, an individual data report for each test location summarizing the composite resilient modulus ($M_{r\text{-Comp}}$), layered base and subgrade (SG) resilient modulus analysis results ($M_{r\text{-Base}}$ and $M_{r\text{-SG}}$), the "universal" model parameter values, modulus of subgrade reaction (*k*-values), penetration resistance profile from dynamic cone penetration (DCP) test and a picture were documented. Example results are shown in Figures 2 and 3. Summary statistics of average (μ), standard deviation (σ), and coefficient of variation (C_v) of the different parameters are summarized separately for each project.

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Figure 1. Automated plate load testing setup with 12 in. diameter and 30 in. diameter loading plates setup.

Key Findings from Field Testing

- Typical values provided in the AASHTOWare Pavement ME Design™ guide based on soil classification can significantly under or overestimate the Mr values. Therefore, it is important to perform field measurements for verification of design input parameters.

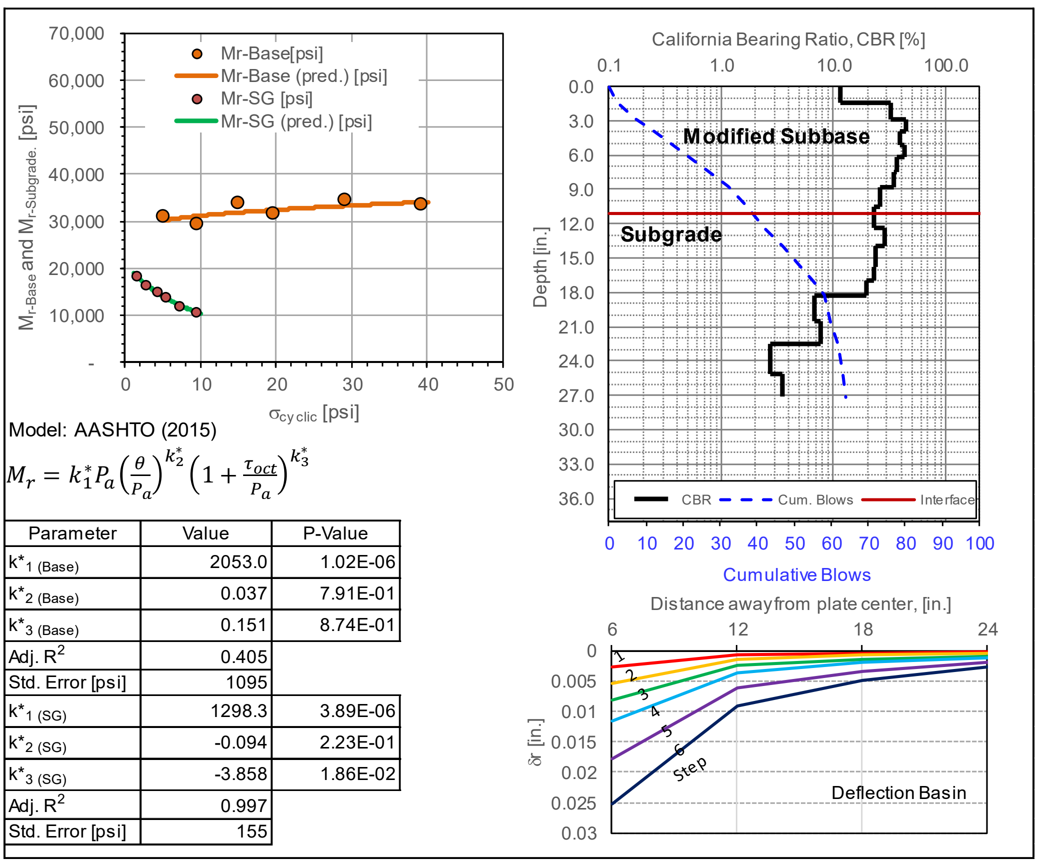
- The cyclic APLTs showed that the Mr values on the unbound layers are variable across the state and within a given project site. The Cv at each site varied from 7% to 70%. For reference, a Cv of about 20% is typically considered a relatively uniform condition. Results from six out of the ten projects yielded Cv > 20%.

- The use of 2 ft of special backfill to improve subgrade in one of the project sites (Projects 4 and 10), provided higher Mr values than other projects, and the special backfill material layer (contained of RAP material) increased its stiffness between test periods.

- The modulus of subgrade reaction k-values obtained across the state varied between 35 pci to 300 pci. 11 out of the 14 tests performed across the state showed k values < 150 pci – the typically assumed (conservative) design input target value by Iowa DOT for PCA (1984) design. At one site (Project 7), two tests performed on the compacted modified subbase layer about 420 feet apart, showed k-values of 39 and 284 pci.

- The kcomp values obtained over granular subbase/modified subbase layers were on average lower than the k-values obtained directly on the underlying subgrade layer. This finding suggests that the subbase layers were relatively loose/uncompacted at the surface, which is also evidenced by the relatively high re-load to initial load k-value ratio (k2/k1). 6 out of 7 tests on subbase layers produced ratios > 3. For reference, Swedish specifications require the ratio of reload to initial moduli values to be < 2.8 for base/subbase layers within the top 0 to 10 inches as an indicator of compaction quality.

- Permanent or plastic deformations occurring from repeated traffic loading is a recognized cause of pavement distresses. δp was monitored and reported for the cyclic and static APLTs. The average δp from each site varied between 0.01 in. and 0.26 in., and the Cv at each site varied between 14% and 123%. δp values at the end of static APLTs show the values varied between 0.05 and 0.4 in. 11 out of the 14 static APLTs showed δp > 0.05 in., which is considered the critical limit to develop LOS beneath pavement.



Model: AASHTO (2015)

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

Parameter	Value	P-Value
$k^*_{1\ (Base)}$	2053.0	1.02E-06
$k^*_{2\ (Base)}$	0.037	7.91E-01
$k^*_{3\ (Base)}$	0.151	8.74E-01
Adj. R^2	0.405	
Std. Error [psi]	1095	
$k^*_{1\ (SG)}$	1298.3	3.89E-06
$k^*_{2\ (SG)}$	-0.094	2.23E-01
$k^*_{3\ (SG)}$	-3.858	1.86E-02
Adj. R^2	0.997	
Std. Error [psi]	155	

Figure 2. **Example test results from cyclic APLT performed at different stress levels showing layered Mr results ($M_{r\text{-Base}}$ and $M_{r\text{-SG}}$) versus cyclic stress on each layer, DCP-CBR and cumulative blows profile, AASHTO (2015) model parameters for each layer, and deflection basin for each load step (Project 3 – 11 in. modified subbase over select subgrade).**

Figure 3. Example test results from static APLTs at two test locations (Project 7 – 12 in. modified subbase over compacted select subgrade).

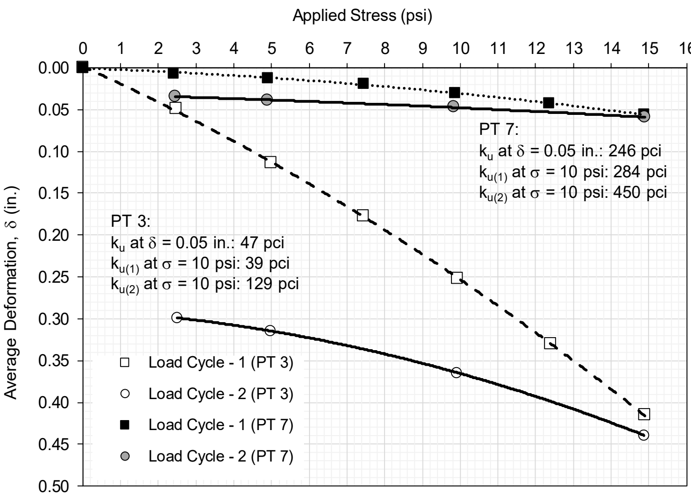


Figure 5. Box plots of k-values for different material/cross-section types.

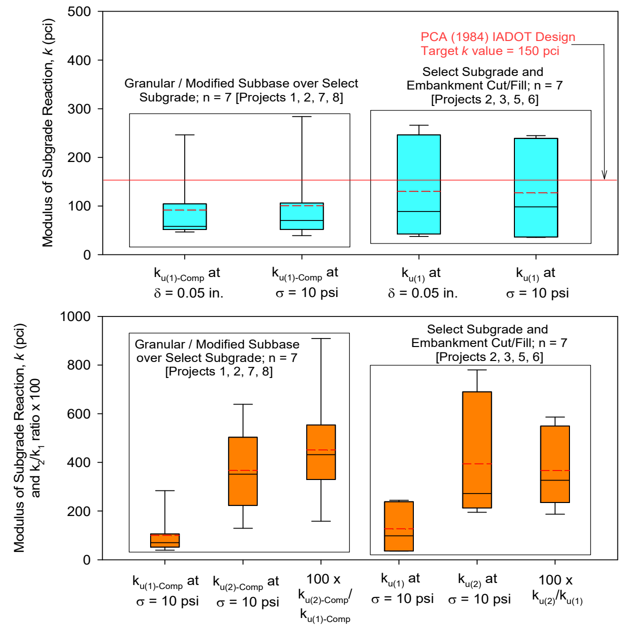
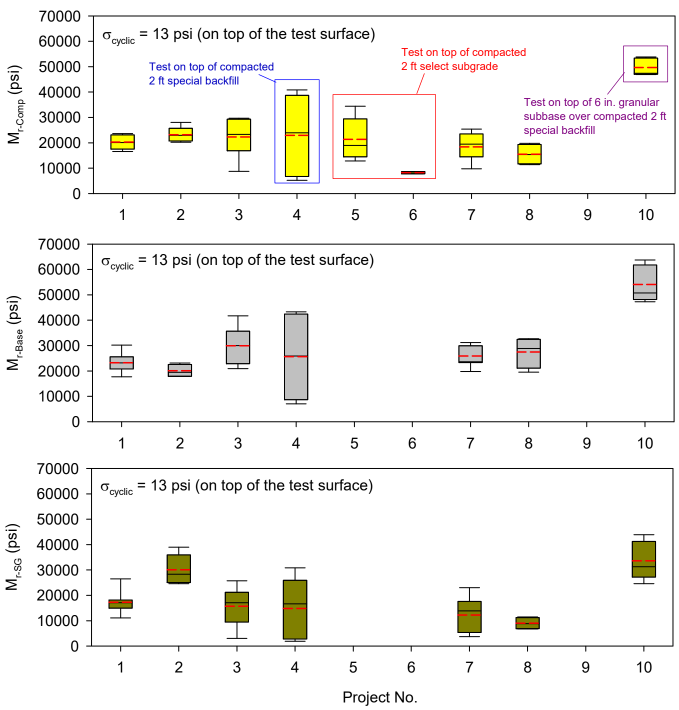


Figure 4. Box plots of $M_{r\text{-Comp}}$, $Mr_{\text{-Base}}$, and $M_{r\text{-SG}}$ results from each project location.



Mechanistic Analysis Using APLT Results

The APLT results were utilized to perform mechanistic analysis of a rigid pavement system and assess the pavement performance characteristics. A few example cases are demonstrated in this report using Kenslabs 2D FE analysis. The main objective of the FE analysis was to assess the influence of k-value, LOS condition, and pavement thickness on the bending stresses in the pavement layer. The stress ratio (SR) values were calculated for each case as the ratio of the maximum principal stress in the pavement layer and the modulus of rupture of the concrete (assumed as 660 psi).

FE analysis results showed that there were no significant differences in the bending stresses between the low and high k-value cases for LOS = 0 condition, but there are significant differences when LOS = 0 versus 1 cases are compared. For the LOS = 1 cases, the peak stresses occurred in a distribution corresponding to a typical corner break observed in distressed concrete pavements. For LOS = 0 condition, the SR values are < 0.45 for all three thicknesses evaluated and k-values evaluated. For LOS = 1 and 2 conditions, the SR values increased and the associated number of allowable load repetitions per PCA (1984) are decreased, with no significant differences between LOS 1 versus 2 conditions. The SR's were either similar or lower for LOS 3 compared to LOS 2 condition.

This analysis demonstrates that during pavement design, simply changing the k-value without accounting for LOS that can potentially occur due to plastic deformations under repeated loading, the calculated bending stresses can be misleading.

Recommendations

- The Iowa DOT is currently either performing or in the process of considering state-wide calibration for AASHTOWare Pavement ME Design™ input parameters. AASHTO (2010) provides guidance on how to perform this calibration work, with the primary objectives of reducing bias and increasing precision of the empirical models used in the design software for predicting performance indicators (i.e., distresses, ride quality). The end-result of this process is developing local calibration-based regression factors that can be updated in the design software. The AASHTO guide document details an approach consisting of 10 steps for the local calibration process. Detailed procedures for developing an experimental plan, estimating the sample size, selecting the roadway segments, collecting the required field data, and assessing bias/standard error in the global calibration factors for local conditions, are discussed in the AASHTO (2010) guide document.
- Selection of appropriate design input parameters should be based on project specific materials and conditions considering the variability and potential post-construction changes in saturation. For rehabilitation design projects, foundation layers can be tested directly to determine in situ k or Mr values. The variability aspect can be addressed by determining the mean (□) and standard deviation (□) of the data and calculating the target value as equivalent to □ - 2□. The moisture aspect must be addressed, especially if field tests are conducted when material is relatively dry. Moisture corrections can be performed via laboratory Mr testing on a given material type at different moisture contents and determining the correction factors for the design moisture content. Alternatively, empirical procedures established based on local historical data or some provided in the AASHTOWare ME design guide can be utilized.
- Field verification of Mr values reduces risk of not meeting the design the pavement design performance criteria and increase quality, thus helping to insure long-term performance. A field quality assurance (QA) protocol and specifications that requires measurement and reporting of in situ Mr values is recommended. The specification should address the test frequency (1 every 500 to 1,000 feet, depending on in situ conditions) required for QA. Specification options with reduced QA testing frequency with implementation of intelligent compaction technologies should also be considered.

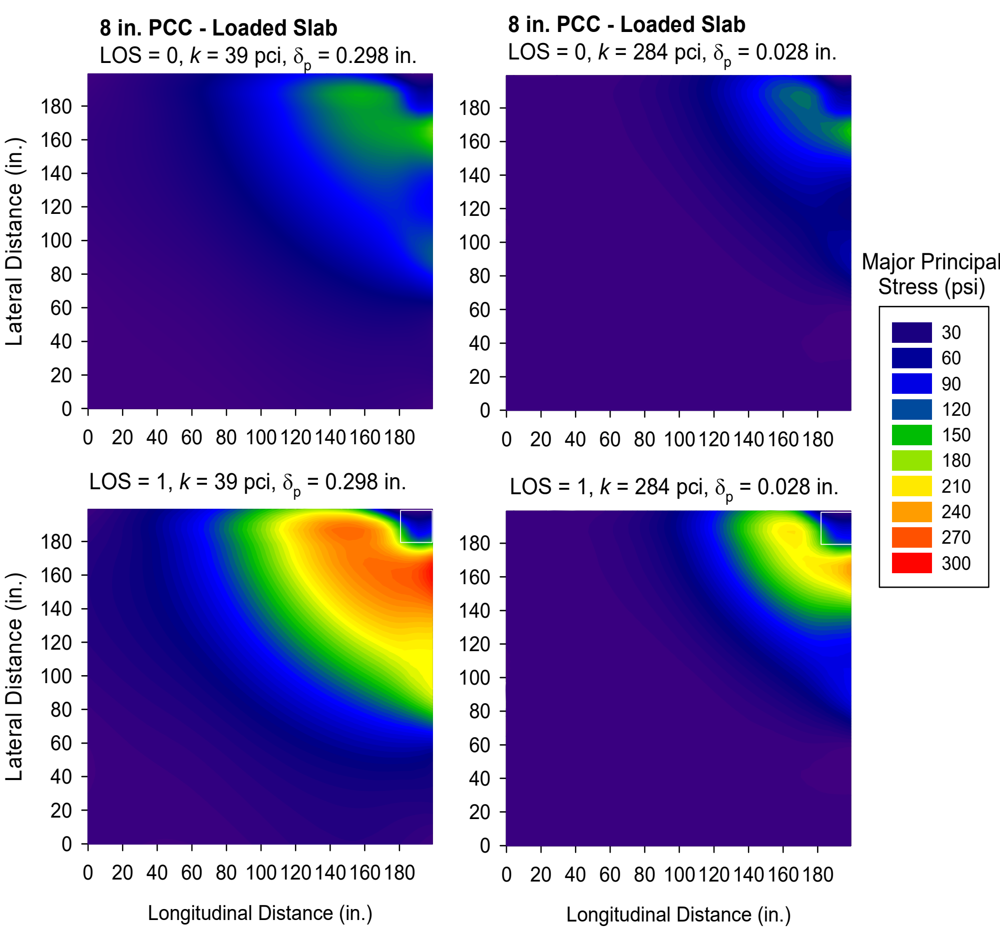


Figure 6. **Spatial contour plots of major principal stresses in the pavement layer for k = 39 pci and δp = 0.298 in. (left) versus for k = 284 pci and □p = 0.028 in., for LOS = 0 and 1 cases.**