Title: KAY COUNTY SHALE SUBGRADE STABILIZATION REVISITED

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ABSTRACT
This paper presents a re-examination of a research project on US-77 in Kay County, Oklahoma concerning chemical stabilization of an 8-inch compacted shale subbase layer completed in 1983. The primary focus of this paper was to examine the changes in the pavement surface deflection and backcalculated modulus of the stabilized subbase layer within the project test sections over time.

The original research project test sections were identified and tested with a trailer mounted Falling Weight Deflectometer (FWD) unit in August 2010. The FWD deflection survey performed in July 2000 and the original Benkelman beam deflection survey performed in June 1985 following the original pavement construction were used in this pavement analysis.

The pavement had been overlain with two thin (1-inch and 1\(\frac{1}{2}\) inch) asphalt overlays. The FWD deflection data was normalized by ratio of the new thickness to the original pavement thickness so that the original Benkelman beam deflection data could be used. The pavement analysis uses the two-layer Hogg model and three-layer model in the Modulus 6.0 software to estimate the subgrade modulus and 8-inch stabilized subbase layer subgrade modulus respectively.

The conclusions are the following: a) the pavement surface deflections indicate that the stabilized subbase layer sections have relatively stiff subgrade support, b) the three-layer model predicts a higher subbase layer modulus, and c) chemically stabilized subbase layers do not significantly lose strength or degrade with time and can be depended on as part of the structural section.
INTRODUCTION
This paper revisits a research project that was completed 28 years ago dealing with the soil stabilization of shale materials used in a compacted subgrade. The purpose of the original research project was to evaluate the effect that these different chemical soil stabilizers had on the shale subgrade materials. The intent of this original research work was to use a significant amount of chemical soil stabilizer on these problem shale materials and to evaluate the field implementation of shale stabilization. At that time, the Oklahoma Department of Transportation (ODOT) commonly used cement, hydrated lime, and Class C fly ash in their subgrade soil stabilization work.

An exhaustive laboratory research study (Phase I) had been completed prior to the field implementation phase on representative shale samples from the project site to assess the shale properties. Phase I of the research project was completed in September 1984 under ODOT Study No. 79-09-2 (1) supervised by the ODOT Research and Development Division and funded by the Federal Highway Administration. The results of this laboratory study showed that 14 percent cement, 4.5 percent quicklime (equivalent to 6.0 percent hydrated lime), and 25 percent fly ash stabilized the shale subgrade soils. Supporting the use of these chemical soil stabilizers, previous research using qualitative X-ray diffraction studies of Oklahoma shales showed substantial decrease in the activity of the clay minerals as a result of stabilization (2). Phase II of the research project dealt with the analysis of samples prepared during construction and those cored from underneath the pavement section. Phase II of this research project was completed in July 1987 under ODOT Study No. 83-07-2 (3) supervised by the ODOT Research and Development Division and funded by the Federal Highway Administration. The results of Phase II showed significant amelioration of the engineering properties of the stabilized shale subgrade material, a USCS CL material, as manifested by their plasticity, compressive and beam strength compared to the raw (non-stabilized) shale material. Examination of the micrographs of the stabilized shale samples revealed the same dense particle packing and reduction in void space as had been seen in previous research in Oklahoma shale samples (4). Benkelman beam deflection measurements were made following the completion of the finished subgrade and pavement section on June 18, 1985 to verify improvement in deformation resistance.

The project location was on a section of highway US-77 just north of Ponca City in Kay County, Oklahoma, (see Figure 1). The study area (not plotted to scale) is immediately north of Hubbard Road and approximately 1.125 miles north of Prospect Avenue shown in Figure 1. The project typical plan design pavement section is presented in Figure 2. The pavement grade line is at 0 to 1 percent grade. The site location is in a wet freeze-thaw cycling region of the Country.

The preliminary project design plans were initially developed in 1976 and finalized in 1981. The AASHTO Interim Guide for Design of Pavement Structures, 1972, Revised 1981(6) was the pavement design document applicable for this project. The average daily traffic (ADT) in 1976 was 4,900, and the estimated 20 year ADT was 9,900. According to the current 2010 ADT map (5), the ADT is 11,200. The appropriate regional factor for freeze-thaw for Kay County (prescribed in the AASHTO Interim Guide for Design of Pavement Structures, 1972, revised 1981) was applied.

The pavement performed very well over the years with no rutting or other pavement roughness. The surface condition of the pavement at the test section did develop a significant amount of random cracks that had to be sealed and maintained. The pavement maintenance was limited for many years requiring only the sealing of the random occurring cracks. The original pavement surface was overlain in 1998 with 1 inch of Type D asphalt and again in 2006 with a
FIGURE 1 Location of study site.
FIGURE 2  Typical cross-section of stabilized test section.
¾-inch of S6 leveling course and 1½-inch of S4 wearing course for a total overlay thickness of 2¼-inches. ODOT’s purpose of these thin pavement overlays was to maintain the structural integrity due to the increased and future traffic loading.

The objective of this paper is to evaluate the overall pavement performance through the years by looking at the changes in the surface deflection and the backcalculated modulus of the stabilized 8-inch subbase layer by comparing deflection data at three time periods.

PROJECT DESCRIPTION
The fill material which constitutes the pavement subgrade was placed in 1980. It was a plastic weathered shale material which belonged to the lower Wellington Formation of Permian geologic age. This shale varied in color from yellowish gray to gray and grayish brown. The AASHTO classification of the subgrade shale material ranged from A-7-6 (25) to A-7-6 (39), and the plasticity index values varied from 26 to 37. Construction of the stabilized base course test sections started on September 6, 1983.

The research project was set up to have four stabilized sections and one control (non-stabilized) section within a total length of 3000 feet in the southbound lanes of the four lane divided highway on US-77 (see Figure 1). The length of the cement section was 500 feet, and it was stabilized with 14 percent by dry weight of Type 1 Portland cement. The lengths of the quicklime and fly ash sections were each 700 feet, and they were stabilized with 4.5 percent quicklime and 25 percent fly ash respectively. A fourth stabilized section had a length of 700 feet, and it was conjunctively stabilized with 8 percent cement, 3 percent quicklime, and 18 percent fly ash. The reason for using such high percentages of the different chemical stabilizers was to quantify construction problems and to establish practical upper limit of their usage. All chemical stabilizers were incorporated on a dry weight basis. The fifth and final section was a 400 foot length of untreated subgrade control section. The arrangement of the five test sections was controlled by the beginning of the project station on US-77 in conjunction with the control section ending at the juncture of Coleman Avenue. A graphical layout of the research project indicating the order of the stabilized sections and station extents is outlined in Figure 3. ODOT completed a report covering the highlights of physical construction by the test sections for the Federal Highway Administration (7).

PAVEMENT INVESTIGATION
The investigation of the pavement includes the original Benkelman beam deflection survey on June 18, 1985 and FWD (Falling Weight Deflectometer) surveys on March 15, 2001, and August 18, 2010.

The FWD surveys were made at this site in accordance with current ASTM D 4694 and ASTM D 4695 test standards (8) (9) applicable at these dates. The load range for the FWD was 9000 lbf wheel load with four drops were recorded with seven geophones spaced at: 0, 8, 12, 24, 36, 48, and 72 inches. Pavement cores were taken at the site in the August 18, 2010 FWD survey as well as a visual pavement condition record. Four pavement cores were made in the southbound direction on August 25, 2010, and the core locations were arbitrarily selected at the beginning and ending stations and at stations 279+00 and 289+00. This coring plan resulted in an average pavement thickness of 15 inches for the cement, quicklime, and half of the fly ash test sections and 14.5 inches for half of the fly ash, combined, and control test sections to be used in the pavement analysis. These thicknesses included the recent asphalt overlays.
FIGURE 3 Location of test sections.

* C = Cement
QL = Quicklime
FA = Fly Ash
The original Benkelman beam survey was made on June 18, 1985 according to the current AASHTO T256-77 (10) standard at the time. For this original deflection survey only the maximum deflection under the 9000-lbf wheel load was measured.

**PAVEMENT ANALYSIS**

The goal of this pavement analysis is to see how well the pavement condition has performed structurally over the course of 27 years of in-place service. For the pavement analysis, the original June 18, 1985 Benkelman beam, July 31, 2000 FWD, and August 18, 2010 FWD surveys were used to compare the maximum surface deflection and then estimate the modulus of the stabilized 8-inch subbase layer.

The pavement analysis consists of two phases: a) the pavement surface deflections were measured, and b) the 8-inch stabilized subbase layer subgrade modulus was estimated from the surface deflection bowls.

**Deflections**

In a comparison of the surface deflections, the theoretical relationships developed from the Boussinesq and two-layer elastic systems were employed (11). It was decided that assuming a straight line relationship between loads and deflections would be satisfactory. Thus, a proportional relationship such as:

\[
\frac{L_1}{d_1} = \frac{L_2}{d_2}
\]

where: \(L_1, L_2\) = the axle loads, pounds  
\(d_1, d_2\) = Benkelman beam or FWD deflections, inches

can be used. This allows the computation of an expected deflection for any load once a deflection for a specific load has been established. The July 31, 2000 and August 18, 2010 FWD measurements each had succeeding asphalt overlays respectively 1-inch and 2\(\frac{1}{4}\)-inch, respectively. Therefore, the deflections measured in the July 31, 2000 and August 18, 2010 FWD surveys shown in Figure 4 were normalized by the ratios 12.75/11 and 15/11 so that the deflections would be comparable to the original 11 inch pavement thickness in the 1985 Benkelman beam deflection measurements. The maximum center deflections comparing the June 18, 1985 Benkelman beam survey and the July 31, 2000 FWD, and the August 18, 2010 FWD surveys are presented in Figure 4.

**Stabilized subgrade modulus**

Since the Benkelman beam deflection survey in June 18, 1985 recorded only a maximum centerline deflection, the initial analysis of this data is limited to a two-layer model, such as the Hogg model (13). The Hogg model is based on a thin plate on an elastic foundation and has been shown to produce satisfactory estimates of the subgrade modulus and is used in this analysis. In the Hogg model, a method by Wiseman and Greenstein (13) allows for the use of the Benkelman beam deflection to estimate the subgrade modulus. The Hogg model used in the estimate of the subgrade modulus, however, requires that two deflection measurements be made, which in this case in 1985 was not done. A further review of the original Benkelman beam maximum deflection under the 9000-lbf wheel load shows that all of these deflections recorded were consistently very low. It was decided to estimate the second required deflection at a
FIGURE 4 Pavement deflection at center of load, normalized to 9000 lbf, 11-in. asphalt thickness.
FIGURE 5  Backcalculated moduli using 2-layer model, semi-infinite depth.
FIGURE 6  Statistical distribution of modulus values.

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Figure 7 Comparison of moduli calculated using 2-layer versus 3-layer model, August 2010.
location within the characteristic length of 100 cm (39.4 in.) at 40 percent of the maximum deflection in order to evaluate the stabilized subgrade modulus by using the Hogg model. The rational for using 40 percent of the maximum deflection was based on Benkelman procedures that would have a second deflection measurement to be located between 40 and 80 cm (14).

The subgrade modulus was first estimated for a two-layer model by two methods: a) the Hogg model used for the original June 18, 1985 Benkelman beam deflection survey and b) the Modulus 6.0 software (12) by TTI was used for the July 31, 2000 and the August 18, 2010 FWD deflection surveys. The subgrade modulus versus the test section station extents comparing the June 18, 1985 Benkelman Beam deflection survey with the July 31, 2000 FWD, and the August 18, 2010 FWD deflection measurements are presented in Figure 5. The statistical distribution of the backcalculated subgrade moduli is shown in Figure 6.

The subgrade modulus shown in Figure 5 includes both the subgrade modulus of the 8-inch stabilized subbase layer along with the underlying subgrade modulus. Again the two-layer Hogg model was used because the original Benkelman beam deflection measurements cannot estimate multilayer subgrade moduli.

Since the subgrade modulus profiles based on the two FWD surveys for the two-layer model presented in Figure 5 are very similar, the August 18, 2010 FWD deflection profile was selected for use in a three-layer model in the Modulus 6.0 software to further identify the subgrade modulus of the 8-inch stabilized subbase layer. Note that the subgrade modulus profile using the Benkelman beam deflection data is not as accurate in estimating the subgrade modulus in a two-layer model as the two FWD surveys and therefore is not considered in further analysis.

In the backcalculation of the subgrade modulus of the 8-inch stabilized subbase layer, the pavement thickness varies as noted earlier as required by the AASHTO/Modulus segmented station core location criteria over the length of the test sections. In Figure 7 the three-layer model using in the Modulus 6.0 software for data derived from the August 2010 FWD deflection profile indicates the subgrade modulus of the 8-inch subbase layer. Also shown in Figure 7 is the two-layer subgrade modulus of the 8-inch stabilized subbase layer plus the underlying subgrade from Figure 5.

**Data analysis**
Considering the surface deflections shown in Figure 4, the pavement surface deflections measured in the original 1985 Benkelman beam survey and in the July 31, 2000 and August 18, 2010 FWD surveys have not deviated considerably. What is of note pertaining to the deflection profiles shown in Figure 4 are the following: a) for the cement, quick lime, and cement test sections the average deflection ranges approximately between 8 and 12 x 10^{-3} inches discounting the spikes, b) for the combined test section the deflection ranges approximately between 0.5 and 2.5 x 10^{-3} inches, and c) for the control test section the deflection ranges between 9 and 22 x 10^{-3} inches. The stiffness of the stabilized 8-inch subbase layer is contributing to a lower pavement deflection within the study extent.

Considering the backcalculated subgrade modulus (8-inch stabilized subbase layer along with the underlying subgrade modulus) profiles for the two-layer model presented in Figure 5 for the two FWD surveys are the following: a) for the subgrade modulus profiles for the cement, quick lime, and cement test sections have a very narrow range between 20 and 35 ksi and b) for the subgrade modulus profile for the combined test section, the range is approximately between 70 and 120 ksi. For the control test section, the subgrade modulus range is between 10 and 25 ksi in the two FWD surveys. The backcalculated subgrade modulus estimated by the Hogg two-layer model for the Benkelman beam survey falls short of the FWD surveys except only in the...
combined test section where the subgrade modulus is comparable. The subgrade modulus estimated by the 1985 Benkelman beam survey does serve as a base line.

In Figure 7 the subgrade modulus of the 8-inch subbase layer is further distinguished from the underlying untreated subgrade modulus in a three-layer model. Plotted also in Figure 7 is the subgrade modulus for the August 2010 two-layer model which includes the 8-inch stabilized subbase layer modulus plus the underlying subgrade modulus from Figure 5. Some variation can now be seen in Figure 7 between the cement, quick lime, and fly ash test sections with the fly ash test section showing the larger range in the 8-inch stabilized subbase layer modulus in the three-layer model. Again the combined test section indicates the highest range in the 8-inch stabilized subbase layer modulus.

CONCLUSION
Based on the results of the surface deflections measured and the estimated backcalculated subgrade modulus the following conclusions can be made:

– The pavement surface deflections indicate that the cement, quick lime, and fly ash test sections have a relatively stiff subgrade support. In the combined test section the deflections indicate simply a very rigid subgrade support. The control test section shows much higher and greater variability in deflections which were anticipated. It can be concluded that the pavement surface deflections have not changed much with time.

– Based on a comparison between the two and three-layer models presented in Figure 7 August 2010 data, the subgrade modulus for the 8-inch stabilized subbase layer shows a higher modulus for the cement, quick lime, fly ash, and combined test sections for the three-layer model compared with the two-layer model. Splitting the fly ash section in two subsections based on segmented station core location requirement resulted in only a slight deviation in increased modulus trend. The subgrade modulus for the 8-inch stabilized subbase layer is substantially higher than the untreated subgrade in the three-layer model.

– A reasonable conclusion in the data analysis is that the subgrade modulus of the 8-inch stabilized subbase layer based on the results shown in Figures 5 and 6 has not changed significantly with time. This study supports the theory that chemically stabilized subbase layers do not significantly lose strength or degrade with time and can be depended on as part of the structural section. The subgrade modulus estimated by the 1985 Benkelman beam survey in the two-layer Hogg model represents a minimum baseline.

REFERENCES


