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Research

Assessing Lower Impulse Load Levels on Reinforced Asphalt Pavement

Final Report

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16. Abstract

NHDOT installed fiberglass grid reinforcement systems at several locations in an effort to address pavement deterioration, in particular fatigue and reflective cracking. Previous to this investigation, nondestructive testing using a falling weight deflectometer was performed on six test sections, two control and four sections where fiberglass grid was installed at mid-depth of the asphalt. The pavement structure was evaluated at a16 kip load, producing inconclusive results.
This investigation analyzed the effect of lower impulse load levels, specifically the 9 kip load, at three test locations, one control and two reinforced. The intent was to use the light weight deflectometer to determine if reduced load levels demonstrate structural benefit. No obvious distinction due to the presence of the fiberglass grid was detected. Based on this investigation, the benefits are not captured using nondestructive testing methods.

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Unit Conversion Factors

Symbol	When You Know	Multiply By	To Find	Symbol
		Length		
mm	millimeters	3.93701×10^{-2}	inches	in.
cm	centimeters	3.93701×10^{-1}	inches	in.
m	meters	3.28084	feet	ft
m	meters	1.09361	yards	yd
km	kilometers	6.21371×10^{-1}	miles (statute)	mi
		Area		
mm^2	square millimeters	1.55000×10^{-3}	square inches	$In.^2$
m^2	square meters	1.07639×10^{1}	square feet	ft^2
m^2	square meters	1.19599	square yards	yd^2
		Volume		
mL	milliliters	3.38140×10^{-2}	fluid ounces	fl oz
L	liters	2.64172×10^{-1}	gallons	gal
m^3	cubic meters	3.53147×10^{1}	cubic feet	ft^3
m^3	cubic meters	1.30795	cubic yards	yd^3
		Mass		
kg	kilograms	2.20462	pound-mass, avoirdupois (avdp)	lbm
g	grams	3.52740×10^{-2}	ounces (avdp)	oz
		Density		
kg/m^3	kilograms per cubic meter	1.68555	pound-mass (avdp) per cubic yard	lbm/yd^3
kg/m^3	kilograms per cubic meter	6.24280×10^{-2}	pound-mass (avdp) per cubic foot	lbm/ft^3
		Temperature (exact)		
°C	degrees Centigrade	$1.8 \times (°C) + 32$	degrees Fahrenheit	°F
°C days	degrees Centigrade days		degrees Fahrenheit days	°F days
		Pressure or Stress		
MPa	megapascals	1.45038×10^{2}	pound-force per square inch	psi

Introduction

NHDOT installed fiberglass grid reinforcement in several flexible roadways throughout the state in an effort to address fatigue cracking and extend the pavement service life. Fiberglass grid manufacturers have promoted the inclusion of reinforcing grid in flexible pavements as an effective rehabilitation method to either reduce or impede cracking. GlasGrid 8501 was installed in sections of New Hampshire Route 101 as part of a maintenance effort. The inclusion of the fiberglass grid was intended to address pavement deterioration. Nondestructive testing has progressed and is a widely accepted approach to structurally evaluate existing pavements. Nondestructive (NDT) testing was performed on three test sections to collect deflection measurements used in this analysis. The three test sections all have 6 in. of asphalt overlying substrate materials.

The test section at mile marker WB (west bound) 131.5 served as the control and did not have fiberglass grid within the asphalt layer. The other two test sections at mile marker WB 128.0 and EB (east bound) 128.4 both has fiberglass grid at mid-depth in the asphalt. The NDT testing measured the pavement response with a Dynatest 8000 Falling Weight Deflectometer (FWD) and a Dynatest 3031 Light Weight Deflectometer (LWD). FWD deflection measurements were taken at load levels of 6, 9, 12, and 16 kips. An initial evaluation of the deflection data at the 16 kip load were inconclusive that the fiberglass grid provided structural benefit. Details of the field testing and initial structural evaluation are in Barna et al. (2016).

This investigation reviews the lower load levels, specifically the 9 kip load, to determine if reduced load levels will demonstrate structural benefit from the inclusion of fiberglass grid reinforcement in the asphalt layer. Available methods will be used to determine the structural number of the in situ pavement sections and, where possible, the layer coefficient for the asphalt layer. These parameters are used as inputs for pavement and overlay design.

Deflection data

Following the guidance in the 1993 AASHTO Guide, the deflection readings for each of the three test sections were normalized to a 9 kip load and adjusted for the temperature. These corrected deflections were used to backcalculate the material layer modulus values and to estimate the structural number.

The Impulse Stiffness Modulus (ISM) was determined for each drop. The ISM is the load divided by the center deflection. It is used to identify test points with similar stiffness values and used in this study to designate the representative basins from each test section analysis (Table 1).

Table 1. Test section representative basins indicating measured deflection readings at each geophone and calculated ISM value.

		Geophone	0	1	2	3	4	5	6	ISM
Mile Marker		radius (inches)	0	12	24	36	48	60	72	
WB 131.5	TP 31 Drop 5	Deflection (mils)	10.2	6.31	3.83	2.28	1.39	0.95	0.66	883
EB 128.4	TP 23 Drop 5	Deflection (mils)	10.6	6.52	4.1	2.52	1.6	1.12	0.78	849
WB 128.0	TP 48 Drop 5	Deflection (mils)	10.9	6.84	4.36	2.75	1.76	1.18	0.83	826

Backcalculation was performed on the three representative basins to estimate the modulus values for each layer in the structure. The backcalculation was performed using the layered elastic analysis method, WESDEF, within the PCASE software program. Input values for the layer thicknesses and material types were determined from the boring logs collected during the field program. An initial run using a 4-layer structure yielded calculated deflection readings with low errors. However, the calculated layer modulus values varied between the three test sections. In particular, the modulus values for the asphalt layer ranged from 350 ksi (WB 131.5 control), 330.6 ksi (EB 128.4, and 473.2 ksi (WB 128.0). The backcalculation was re-run using a 3-layer structure. For all three test sections, the asphalt layer was 6 in. The thickness of the base layer for each test section was 8.5 in. WB 131.5 (control), 12 in. EB 128.4, and 19 in. WB 128.0. It was assumed there was no bedrock material at a shallow depth, therefore the total thickness of each test section was 240 in. Shallow bedrock may produce interference in the backcalculation method (Janoo 1994). Modulus seed values selected for each layer included, asphalt 350,000 psi (used the WESDEF default values), base material 31,000 psi (Janoo 1994), and subgrade 20,800 psi (Janoo 1999). The backcalculation results are shown in Table 2.

Table 2. Input values used in backcalculation for each representative test basin.

		Geophone Number	Thickness (inches)	Modulus Values Seed (psi)	Minimum (psi)	Maximum (psi)	Calculated (psi)
WB 131.5	TP 31 Drop 5	Asphalt	6	350,000	100,000	1,000,000	665,000
		Base	8.5	31,000	5,000	150,000	16,500
		Subgrade		32,300	28,000	38,000	28,000
EB 128.4	TP 23 Drop 5	Asphalt	6	350,000	100,000	1,000,000	502,000
		Base	8.5	31,000	5,000	150,000	40,000
		Subgrade		25,500	20,500	30,500	23,000
WB 128.0	TP 48 Drop 5	Asphalt	6	350,000	100,000	1,000,000	650,000
		Base	19	31,000	5,000	150,000	28,000
		Subgrade		26,750	21,750	31,750	22,000

Structural number determination from FWD data

AASHTO defines the structural number (SN) of a roadway as an index value based on a traffic analysis, roadbed soil conditions, and the environment used to determine the flexible pavement design thickness. A layer coefficient is used to relate the material type in each layer of the pavement. The SN is used in the design procedure to ensure adequate pavement thickness to withstand the estimated traffic over the service life (AASHTO 1993). The procedure for overlay design described in *Part III Pavement Design Procedures for Rehabilitation of Existing Pavements, Chapter 5 Rehabilitation Methods With Overlays, Section 5.3.3 Structural Evaluation of Existing Pavement* (AASHTO 1993) was followed for this analysis. Backcalculation of the layer moduli is a key component of this method.

Two additional methods to determine the structural number from nondestructive testing were proposed by Noureldin (1993) and Rohde (1994).

The advantage of these two approaches is neither requires the use of a backcalculation procedure to estimate the layer moduli. The backcalculation procedure with the deflection data is complex, but the material layer thicknesses and properties are critical inputs. Layer information may be not available through either a well-documented pavement construction history or core collection. These approaches were formulated from FWD testing conducted on numerous flexible pavements of varying thicknesses and age, and offer an alternative to determine a pavement structural number.

AASHTO

The values determined from backcalculation are used to determine an effective modulus (E_p) and the effective structural number (SN_{eff}). The pavement effective modulus is determined from Equation 1:

$$d_0 = 1.5pa\left\{\frac{1}{M_R\sqrt{1+\left(\left[\frac{D}{a}\right]^3\sqrt{\frac{E_p}{M_R}}\right)^2}} + \frac{\left[1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right]}{E_p}\right\} \quad \text{Equation 1}$$

Where d_0 is the center deflection (adjusted to the standard temperature), p is the load pressure plate (psi), a is the load plate radius (in.), D is the total thickness of the pavement layers above the subgrade (in.), M_R is the resilient modulus of the subgrade (psi), and E_p is the effective modulus of the combined pavement layers above the subgrade (psi).

The effective modulus is used to determine the effective structural number using Equation 2:

$$SN_{eff} = 0.0045\, D\, \sqrt[3]{E_p} \quad \text{Equation 2}$$

The calculated effective structural numbers for each test section are listed in Table 4. It should be noted that the calculation procedure reduces the subgrade modulus (M_R) value using a coefficient multiplier of 0.33. The Guide uses this approach to better align the backcalculated subgrade modulus to the results of the AASHO Road Test (AASHTO 1993) stating that backcalculated moduli values are typically higher than the AASHO Road Test. The resulting effective modulus (E_p) that produced an equivalent center deflection measurement (d_0) for WB 131.5 (control) was 131,000 psi. The E_p values for the two sections with fiberglass grid were 139,000 and 98,000 psi for EB 128.4 and WB 128.0, respectively. The resulting calculated effective structural numbers were 8.2, 8.4, and 5.2 for sections WB 131.5 (control), EB 128.4, and WB 128.0, respectively. A comparison of the SN_{eff} for all of the methods are given in Table 4.

According to the AASHTO method, a structural layer coefficient value (a_i) may be determined using the chart for dense-graded asphalt concrete. However, the chart limits the maximum value of the asphalt elastic modulus to 500 ksi noting that values above 450 ksi used be used with caution. The corresponding layer moduli coefficient at 450 ksi is approximately 0.45. The asphalt modulus values from the backcalculation procedure all exceed 500 ksi making the layer moduli indeterminate using this approach.

Unique deflection location

Noureldin (1993) proposed an approach to determine the structural number using the deflection measurements from falling weight deflectometer (FWD) testing. The basis of this concept is that there is a unique deflection reading that occurs at the top of the subgrade directly below the surface loading, and that this unique deflection point corresponds to a radial distance from the center plate loading point. The unique deflection reading at the top of the subgrade directly below the load plate is termed D_x, and the radial distance from the center of the load plate is r_x. D_x and r_x are used to determine the subgrade modulus (E_{sg}), the overall pavement modulus (E_p), and the effective total thickness (T_x). The mathematical difference between the center deflection (D_0) and D_x results in the pavement deflection below the center.

A key feature to this approach is it does not necessitate having information on the subsurface layer thicknesses. The underlying assumptions to this approach include the basis of on an idealized two-layer flexible structure, the material layers are homogeneous and isotropic – an assumption typical for backcalculation routines, the lowest modulus value in the pavement system corresponds to the top of the subgrade, and the total pavement thickness is the distance from the top of the pavement to the lowest modulus value (top of the subgrade).

The FWD deflection data was normalized to a 9 kip load level. Use the geophone radial distance (r_x) and the corresponding normalized deflection measurement (D_x) to determine for each geophone the subgrade modulus (E_{sg}), the overall pavement modulus (E_p), and effective total thickness (T_x) were calculated for each geophone location using equations 3 through 7 (Noureldin 1993):

$$geophone\ multiplication\ factor = r_x D_x \qquad \text{Equation 3}$$

$$E_{sg} = \frac{2{,}149}{r_x D_x} \qquad \text{Equation 4}$$

Where E_{sg} is the subgrade modulus (psi), r_x (inches) is the geophone radial distance for each geophone located outside the loading plate (r_1, r_2, r_3, r_4, r_5, and r_6), and is D_x (inches) is the deflection corresponding to the radial distance.

$$E_p = \frac{716 - \frac{2{,}149}{r_x}}{D_0 - D_x} \qquad \text{Equation 5}$$

Where E_p is the overall pavement modulus (psi), D_0 is the deflection under the center of the load plate (inches), and r_x and D_x are the same as above.

Determine the effective total thickness of the pavement

$$T_x = \left[\frac{D_0 - D_x}{D_x\left(\frac{r_x}{3} - 1\right)}\right]^{1/3} * (4r_x^2 - 36)^{1/2} \qquad \text{Equation 6}$$

Where T_x is the effective total pavement thickness, D_o, D_x, and r_x (inches) are the same as defined above.

$$SN_{eff} = \frac{(4r_x^2 - 36)^{1/2}}{17.234\ (r_x * D_x)^{1/3}} \qquad \text{Equation 7}$$

From the geophone multiplication factor, identify the largest value. In this example, this is geophone 2 at a distance of 24 inches. Use the values for geophone 2 to determine the pavement deflection ($D_o - D_2$) and the effective structural number (SN_{eff}). Table 4 summarizes the calculations for the three test sections. The effective structural number for all methods are compared in Table 4.

Table 3. Calculations using the unique deflection location approach (Noureldin 1993) for the three test sections.

a. WB 131.5 (control)

	Geophone Number	0	1	2	3	4	5	6
Radius	r_x (inches)	0	12	24	36	48	60	72
Normalized Deflection	D_x (mils)	10.2	6.31	3.83	2.28	1.39	0.95	0.66
Geophone Distance	$r_x * D_x$		0.08	0.09	0.08	0.07	0.06	0.05
Subgrade Modulus	E_{sg} (psi)		28,371	23,371	26,173	32,198	37,689	45,208
Overall Pavement Modulus	E_p (psi)		138,199	98,408	82,904	76,218	73,558	71,947
Effective Total Thickness	T_x (inches)		13.7	29.5	48.9	71.9	95.9	123.2
Pavement Deflection	$D_0 - D_x$ (mils)			6.4				
Effective Structural Number	SN_{eff}			6.12				

b. WB 128.0

	Geophone Number	0	1	2	3	4	5	6
Radius	r_x (inches)	0	12	24	36	48	60	72
Normalized Deflection	D_x (mils)	10.9	6.84	4.36	2.75	1.76	1.18	0.83
Geophone Distance	$r_x * D_x$		0.08	0.10	0.10	0.08	0.07	0.06
Subgrade Modulus	E_{sg} (psi)		26,193	20,541	21,732	25,416	30,243	36,133
Overall Pavement Modulus	E_p (psi)		132,258	95,822	80,528	73,477	70,032	68,133
Effective Total Thickness	T_x (inches)		13.5	28.5	46.4	67.3	90.6	116.5
Pavement Deflection	$D_0 - D_x$ (mils)			6.5				
Effective Structural Number	SN_{eff}			5.86				

c. EB 128.4

	Geophone Number	0	1	2	3	4	5	6
Radius	r_x (inches)	0	12	24	36	48	60	72
Normalized Deflection	D_x (mils)	10.6	6.52	4.1	2.52	1.6	1.12	0.78
Geophone Distance	$r_x * D_x$		0.08	0.10	0.09	0.08	0.07	0.06
Subgrade Modulus	E_{sg} (psi)		27,447	21,830	23,656	28,008	32,096	38,063
Overall Pavement Modulus	E_p (psi)		131,746	96,404	81,260	74,568	71,718	61,903
Effective Total Thickness	T_x (inches)		13.8	29.0	47.6	69.1	91.7	117.5
Pavement Deflection	D_0 - D_x (mils)			6.5				
Effective Structural Number	SN_{eff}			5.98				

Pavement offset

Rohde's (1994) approach to using FWD data to determine a structural number followed the assumption that the peak deflection below the load on a pavement structure is the response of the elastic compression of the pavement structure and a deflection in the subgrade. The stress distribution from the surface deflection in the underlying layers is believed to occur in the subgrade layer at a distance of 1.5 times the pavement thickness. The difference between the peak deflection and the deflection at 1.5 the pavement thickness produces a structural index (SIP). The SIP is related to the pavement structural number using equations 8 and 9.

$$SIP = D_0 - D_{1.5Hp} \qquad \text{Equation 8}$$

Where *SIP* is the pavement structural index (µm), D_0 is the peak FWD deflection measured under a 9 kip load, $D_{1.5Hp}$ is the calculated FWD deflection at an offset of 1.5 times the pavement thickness, and Hp is the total pavement thickness (mm).

$$SN = k_1 SIP^{k2} Hp^{k3} \qquad \text{Equation 9}$$

SN is the structural number, *SIP* and *Hp* are as defined above, and the coefficients k_1, k_2, and k_3 as reported by Rohde for asphalt concrete were 0.4728, –4.4810, and 0.7581, respectively.

Table 4. Calculated effective structural numbers for each test section for each method described.

Mile Marker		Effective SN		
		AASHTO	Noureldin	Rohde
WB 131.5	TP 31 Drop 5	8.2	6.0	6.1
EB 128.4	TP 23 Drop 5	8.4	6.2	6.0
WB 128.0	TP 48 Drop 5	5.2	5.9	4.8

Table 4 shows the difference in the structural numbers for each method. For WB 131.5 (control) and EB 128.4 the AASHTO method resulted in higher SN than both Noureldin and Rohde. For WB 128.0, the SN for all three methods are similar with Noureldin producing the highest value at 5.9. There seems to be more agreement between the Noureldin and Rohde approaches.

Light Weight Deflectometer

A Dynatest 3031 LWD (Light weight deflectometer) testing device was used during the field testing. The LWD is a portable impulse load testing device, similar to the larger versions of the falling weight deflectometer (FWD) or heavy weight deflectometer (HWD) that measure the structural response of a pavement system. Usage of the LWD to measure material stiffness properties of unbound and foundation materials, as well as quality control – quality assurance in highway construction has been gaining acceptance (Livneh and Goldberg 2001, Fleming et al. 2007), and similarly. However, there is little in the literature regarding the use of LWD for the evaluation of existing in-service pavement structures. The LWD is more typically used on unbound pavement materials (Dynatest 2006a), and while it may not be the most suited test instrument for asphalt material, the range of stress levels was under investigation to emphasize the response of the asphalt layer.

LWD testing was conducted at six test points within each test section and coincided, slightly offset, with the location used for FWD testing (Figure

1). At each test point, the drop weight was released four times at each of the four heights for a total of 16 drops (Table 5). This corresponded to load levels ranging from 1,500 to 2,900 lbs. The radius of the loading plate at the base of the unit was 3 in. Deflection readings were measured at three location: at the center of the load plate and at two radial sensors located at 12 and 24 in.

Figure 1. Operation of Dynatest LWD portable impulse loading device during field testing.

Table 5. LWD deflection measurements at each test section.

	Mile Marker		Test Point	Test Time	Drop Number	Load (lbs)	Deflection (mils) D0 0	D1 12	D2 24
WB	131.5	WP	31	15:08:00	1	1,482	2.1	1.3	0.7
					2	1,481	2.0	1.3	0.7
					3	1,472	2.0	1.2	0.7
					4	1,475	1.7	1.3	0.7
					5	1,997	2.8	1.7	1.0
					6	1,979	2.9	1.7	1.0
					7	1,975	2.9	1.7	1.0
					8	1,963	3.0	1.7	1.0
					9	2,440	3.6	2.2	1.3
					10	2,426	3.4	2.2	1.3
					11	2,417	3.2	2.2	1.3
					12	2,422	3.1	2.2	1.3
					13	2,871	3.4	2.6	1.5
					14	2,857	3.3	2.5	1.5
					15	2,894	3.3	2.6	1.5
					16	2,878	3.6	2.6	1.5
WB	128.0	CL	48	12:07:00	1	1,423	1.8	0.8	0.5
					2	1,421	1.7	0.8	0.6
					3	1,438	1.7	0.8	0.5
					4	1,446	1.8	0.8	0.7
					5	1,931	2.8	1.2	0.7
					6	1,917	2.5	1.2	0.7
					7	1,917	2.9	1.2	0.7
					8	1,921	2.6	1.2	0.7
					9	2,371	2.9	1.6	1.0
					10	2,359	3.1	1.6	1.0
					11	2,354	3.1	1.6	1.0
					12	2,344	3.8	1.6	1.1
					13	2,813	3.7	1.9	1.0
					14	2,814	3.9	2.0	1.2
					15	2,797	3.2	2.0	1.2
					16	2,790	3.4	2.0	1.2
EB	128.4	WP	23	13:18:54	1	1,469	2.0	1.1	0.6
					2	1,460	1.8	1.0	0.6
					3	1,457	1.9	1.0	0.6
					4	1,454	1.9	1.0	0.6
					5	1,945	2.4	1.4	0.8
					6	1,931	2.2	1.4	0.8
					7	1,935	2.3	1.4	0.8
					8	1,930	2.2	1.4	0.8
					9	2,384	3.1	1.8	1.0
					10	2,377	3.2	1.8	1.0
					11	2,385	3.3	1.8	1.0
					12	2,384	3.3	1.8	1.0
					13	2,813	3.8	2.2	1.3
					14	2,829	4.0	2.2	1.3
					15	2,825	3.7	2.2	1.3
					16	2,813	3.8	2.2	1.3

Given the modulus values for the asphalt layer backcalculated from the FWD data, the LWD data was examined to corroborate the backcalculated results. The deflection data was analyzed using the licensed software program Dynatest LWDMod version 1.2.23 (Dynatest 2006b) to backcalculate the material modulus values. Limited field testing has been done using the LWD testing apparatus on thin-layered asphalt roadways, yet satisfactory correlation with FWD data has been observed (Fleming et al. 2007).

Test points selected for evaluation were the same as the FWD test point for the representative basin (WB 131.5 test point 31, For each of the three NH 101 test sections, a two-layer structure was input consisting of 6 in. of asphalt overlying the base layer. Two trials were attempted, the first used the measured deflection data without any adjustments. The second scenario normalized the deflection data to a 9 kip load level and adjusted them for temperature. In the first scenario the backcalculated modulus values returned were low with an average of 157,000 psi. For the second scenario, the program was unsuccessful in converging on a deflection basin. The resulting backcalculated modulus values were unreasonably low and not considered further in the analysis.

Test points selected for evaluation were the same as the FWD test point for the representative basin (WB 131.5 test point 31, For each of the three NH 101 test sections, a two-layer structure was input consisting of 6 in. of asphalt overlying the base layer. Two trials were attempted, the first used the measured deflection data without any adjustments. The second scenario normalized the deflection data to a 9 kip load level and adjusted them for temperature. In the first scenario the backcalculated modulus values returned were low with an average of 157,000 psi. For the second scenario, the program was unsuccessful in converging on a deflection basin. The resulting backcalculated modulus values were unreasonably low and not considered further in the analysis.

Estimating stresses and strains in the asphalt layer

The stresses and strains within the asphalt layer were calculated using a linear elastic analysis module, called WinJULEA, for a multi-layered structure. The WinJULEA module is a utility application within the PCASE software program. module applicable for the module WinJULEA (Jacob Uzan Layered Elastic Analysis). Figure 2 illustrates the input screen containing the layer thicknesses and material values for the control section. The evaluation points (lower left corner of the Figure 2) correspond to the FWD geophone spacing and the surface values are calculated as indicated by the depth of 0 (under the Calculation Depths box).

As an initial test, the vertical deflections for all surface geophone locations were calculated and compared to the measured deflections for each of the three representative basins. Figure 2 illustrates reasonable agreement. However, there is a difference of nearly 2 mils between the calculated and vertical deflection at the center of the plate. Vertical deflections were calculated and shown below for the two grid sections, EB 128.4 and WB 128.0, respectively. There is good agreement for EB 128.4 and WB 128.0, however a greater difference for WB 131.5.

Figure 2. Screen shot of layered elastic analysis module.

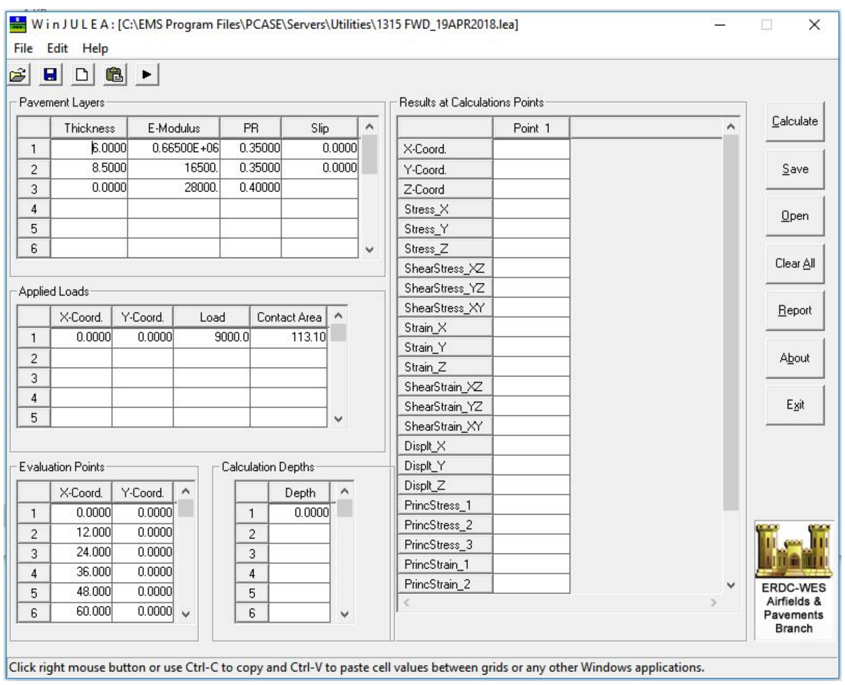


Figure 3. Calculated deflections compared to measured deflections using layer thicknesses and backcalculated material modulus values for each representative basin.

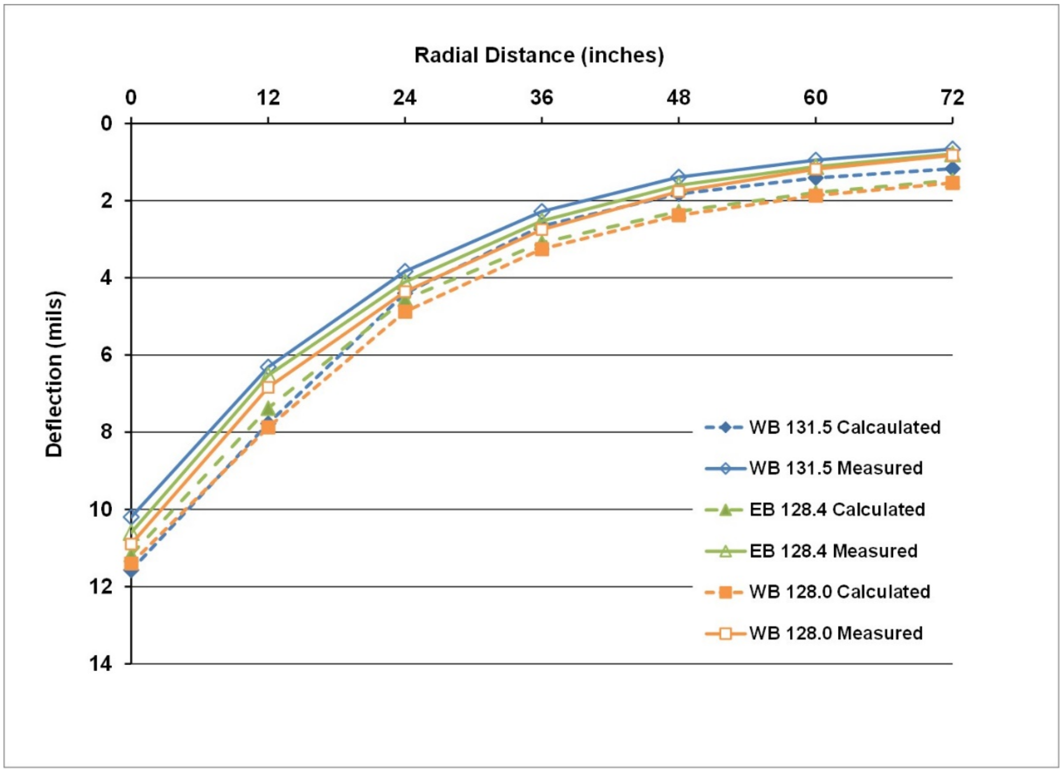


Table 6. Vertical center deflection calculations used in layered elastic analysis.

	Geophone number	1	2	3	4	5	6	7	Calculated Basin Area
	Radial distance from plate center	0	12	24	36	48	60	72	(in^2)
WB 131.5 (control) TP 32 Drop 5	Measured vertical deflection	10.20	6.31	3.83	2.28	1.39	0.95	0.66	0.24
	Calculated vertical deflection	11.57	7.77	4.40	2.65	1.82	1.41	1.17	0.29
	difference (%)	13%	23%	15%	16%	31%	48%	77%	21%
EB 128.4 TP 23 Drop 5	Measured vertical deflection	10.60	6.52	4.10	2.52	1.60	1.12	0.78	0.26
	Calculated vertical deflection	11.17	7.38	4.55	3.09	2.28	1.79	1.47	0.30
	difference (%)	5%	13%	11%	22%	42%	60%	88%	18%
WB 128.0 TP 48 Drop 5	Measured vertical deflection	10.90	6.84	4.36	2.75	1.76	1.18	0.83	0.27
	Calculated vertical deflection	11.40	7.87	4.87	3.25	2.38	1.87	1.54	0.32
	difference (%)	5%	15%	12%	18%	35%	58%	86%	17%

The calculated vertical deflections tend to over predict using layered elastic analysis. However, the center deflection is chiefly of interest to estimate the stresses and strains within the asphalt layer, therefore the difference between the measured and calculated deflections are not considered unreasonable for further analysis.

The layered structures were analyzed to estimate the stresses and strains only within the asphalt layer. The values were calculated at the surface (0 in.) and at inch depth intervals (1, 2, 3 [depth of the fiberglass grid], and 4), with the final location just above the bottom of the layer at 5.99 inches. Negative values indicate tension. Figure 4 and Figure 5 show the calculated stress in the horizontal and vertical directions, respectively. The horizontal and vertical strains are shown in Figure 6 and Figure 7.

Figure 4. Calculated horizontal stress within asphalt layer for each representative basin.

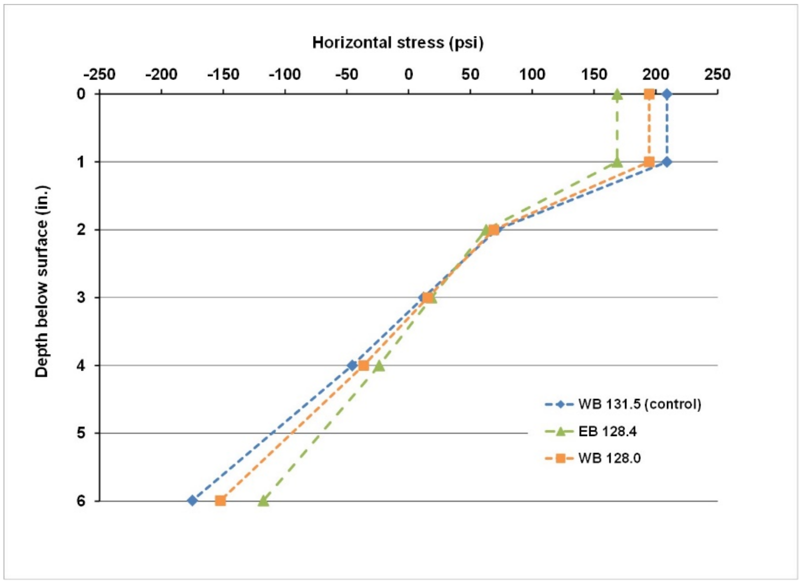


Figure 5. Calculated vertical stress within asphalt layer for each representative basin.

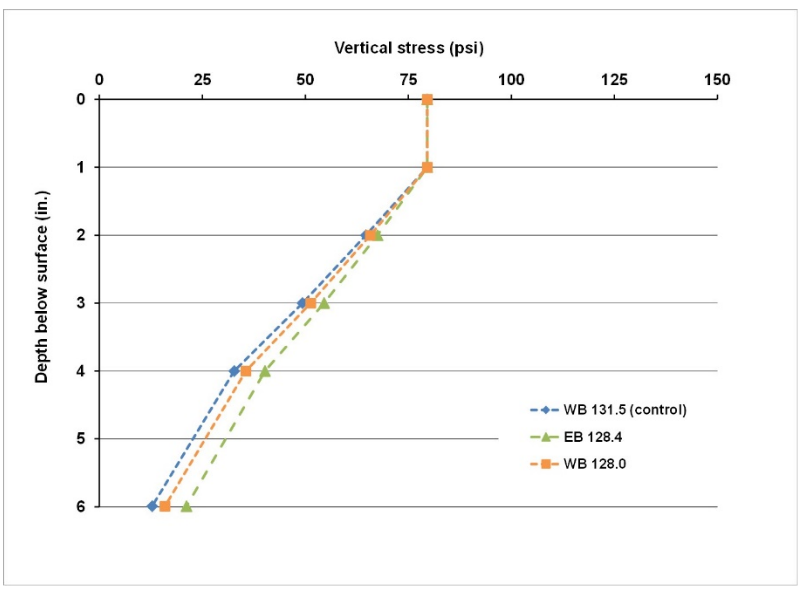


Figure 6. Calculated horizontal strain within asphalt layer for each representative basin.

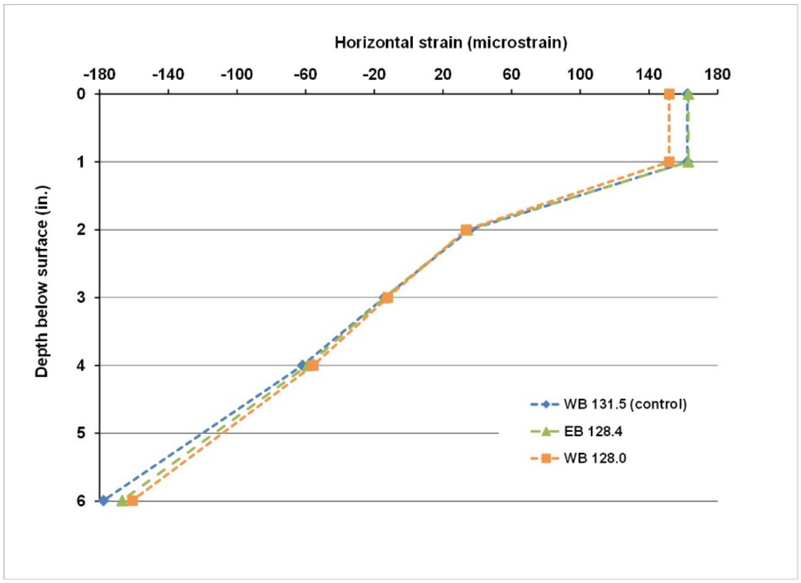
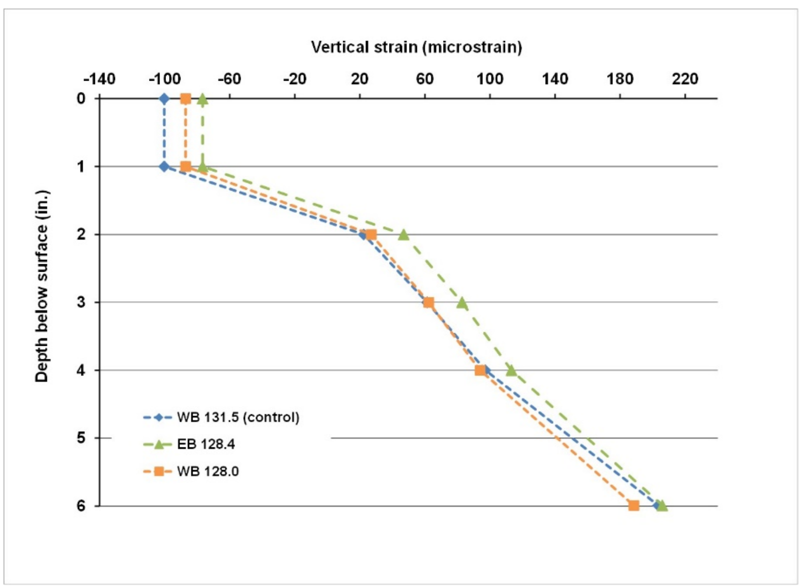


Figure 7. Calculated vertical strain within asphalt layer for each representative basin.



The different asphalt modulus values and the differing structure base layer thicknesses yield somewhat different horizontal stress values at the asphalt surface. The calculated horizontal stress for sections WB 131.5 (control) and WB 128.0 closely compare given the comparable asphalt modulus values obtained from backcalculation. Of interest is the convergence of the horizontal stress for all three sections at a depth of 2 in. below the surface. At 3 in., or the depth of the fiberglass grid for the test section EB 128.4 and WB 128.0, the calculated horizontal stress is nearly identical to the control section. At the bottom of the asphalt layer the calculated horizontal stress is in a tensile state, as would be expected.

The calculated vertical stress for all three of the test sections indicate a similar response to the horizontal stress. The vertical stress decreases, yet remains in compression with increasing depth through the asphalt layer. The control section appears to have the lowest vertical stress value

To observe if the calculated stresses and strains within the asphalt layer varied a distance away from the center of the plate, another analysis was run at a distance of 3 in. and 6 in. linear distance away from the plate center. The results are shown in Figure 8 and Figure 9.

It is worth noting that none of the stress or strain calculations for the sections with fiberglass grid show any observable difference in the values. All of the sections converge at approximately the mid-point of the asphalt layer.

Figure 8. Horizontal stress for each test section at 3 in. linear distance from center of plate.

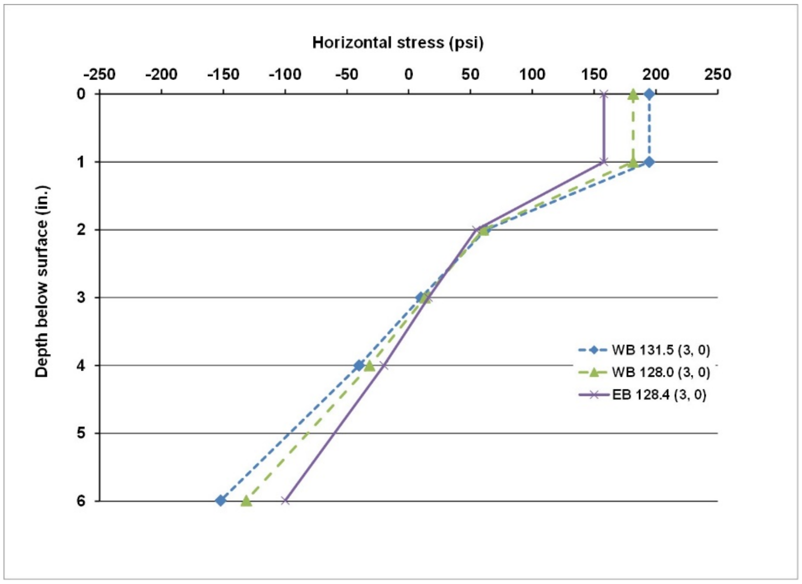
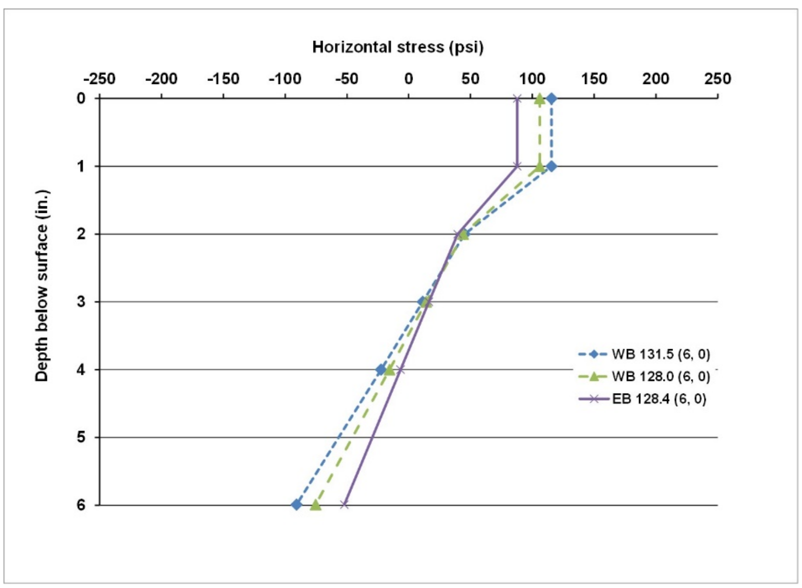


Figure 9. Horizontal stress for each test section at 6 in. linear distance from center of plate.



Conclusions

Representative basins for the three 6 in. thick asphalt layered test sections were selected for this analysis. The measured FWD deflection data obtained during nondestructive field testing was used to select the most similar test points for comparison. The test sections are identified as WB 131.5 (control), EB 128.4, and WB 128.0. Both EB 128.4 and WB 128.0 test sections had fiberglass grid installed at mid-depth within the asphalt layer while WB 131.5 was the control section without fiberglass grid. Three approaches were applied to determine an effective structural number for each test section: AASHTO 1993 Design Guide, Noureldin, and Rohde. The calculated effective structural number for each test section was compared to determine if the addition of the fiberglass grid provided structural benefit to the roadway, and if so, could the benefit be quantified.

The results of the structural number varied depending on the calculation method used. The AASHTO method produced the highest SN values and given the point in the service life, this result is questionable. There was greater similarity of the SN between the Noureldin and Rohde methods that used the deflection readings, but not the backcalculation results. Given that the structural numbers were similar, it is concluded that no benefit is gained with the inclusion of fiberglass grid in the asphalt layer through the application of nondestructive testing.

The deflection measurements collected from the Light Weight Deflectometer (LWD) were also analyzed through means of backcalculation. The intent was to use the LWD data to more accurately backcalculate the modulus of the asphalt layer. The backcalculation results for the asphalt layer were unreasonably low.

Layered elastic analysis was used to calculate the stresses and strains with depth in the asphalt layer and compare these results between the control and fiberglass grid test sections. The results showed strong similarity between the stresses and strains for all of the test sections. No obvious distinction due to the presence of fiberglass grid within the asphalt layer was detected.

Fiberglass grid reinforcement within the asphalt layer may provide some benefit to a flexible roadway and overall pavement performance. Based on the methods and results obtained through this investigation, the benefits of fiberglass grid reinforcement are either imperceptible or are not adequately captured using nondestructive testing methods and existing evaluation tools.

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