Variability in Estimation of Structural Capacity of Existing Pavements from Falling Weight Deflectometer Data

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The calculation of existing pavement structural capacity in terms of 18-kip equivalent single axle load (18-kip ESAL) repetitions by the mechanistic-empirical method is a multistep analysis process. The variability in this calculation process is presented with respect to falling weight deflectometer (FWD) input deflection data in the backcalculation scheme, backcalculated layer moduli, and the number of FWD tests over a section of the road. As can be expected, the sensor readings of FWD on a long section of the road show more variability than on a short section. This variability in sensor readings is magnified when the layer moduli are backcalculated (i.e., small variability in sensor data over a section of a pavement will result in high variability of calculated layer moduli). However, this variability is independent of the length of the section of the roadway over which deflection testing is done. All the layer moduli and their interaction affect the calculated structural capacity. The variation in backcalculated layer moduli is magnified when the number of 18-kip ESALs the pavement can carry before fatigue failure is estimated. Interaction of high asphalt concrete modulus and base modulus tends to produce low asphalt concrete strain and consequently a high number of 18-kip ESAL applications. The opposite is true for interaction of low asphalt concrete modulus and low base modulus. As a result, the computed structural capacity becomes highly variable, especially when the number of tests done on a long section of the pavement is small. The frequency of testing does not affect the estimated 18-kip ESALs over a short section of pavement. However, for long sections it affects the mean estimated 18-kip ESALs. For a mile-long section, five FWD tests were found to be a viable choice for estimation of 18-kip ESALs. However, the coefficient of variation of estimated 18-kip ESALs over a long section may or may not decrease with increasing number of tests.

Nondestructive testing (NDT) is widely recognized as an important tool for pavement structural evaluation. State-of-the-art NDT evaluation measures a pavement's deflection response to a known load. The load generated by an NDT device may be static (Benkelman beam), steady-state vibratory (dynaflect and road rater), or impulse [falling weight deflectometer (FWD)]. Though surface deflection data analysis is a matter of continuing research, NDT for measuring surface deflection is accepted by most highway agencies as standard practice because it is fast and reliable in most cases.

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PROBLEM DEFINITION

Calculation of pavement structural capacity, in terms of the ability to carry 18-kip equivalent single axle load (18-kip ESAL) repetitions, from FWD data is a three-step procedure. First, the layer moduli are backcalculated from the FWD, layer type, and thickness data. Second, the critical pavement response, usually the tensile strain at the bottom of asphalt concrete layer, is calculated. Third, empirical relationships are used for estimating the number of 18-kip ESALs based on the critical pavement response. The relationship estimates the number of 18-kip ESAL repetitions the pavement can carry before fatigue failure. Variability in any stage of the analysis affects the estimation of structural capacity by the mechanistic-empirical method.

OBJECTIVES

The objectives of this study were (a) to find the variability of calculated 18-kip ESALs with respect to the variability in input FWD data and corresponding backcalculated layer moduli over short and long sections of pavements and (b) to find the effect of number of FWD tests over a short section (90 ft) and a long section (1 mi) of pavement on the estimated structural capacity.

DATA COLLECTION

A list of the 16 sites selected for this study is presented in Table 1, and the pavement sections of these sites are presented in Table 2. The sites were selected in the Arizona State University Overlay Study (*1*) for Arizona Department of Transportation (ADOT) on the basis of a number of preselected criteria. All deflection data were collected with a Dynatest model 8002 FWD. The sensors were spaced at 12-in. intervals with the first sensor located at the center of the load. The target load was 9,000 lb. At Sites 1 through 13 deflection data were measured in the outer wheel path at 10 locations at 10-ft intervals. For Sites 14 to 16, deflection data were collected every 0.1 mi.

ANALYSIS METHOD

The analysis process requires backcalculation of layer moduli of the pavements from FWD data and computation of struc-

TABLE 1 Location of Test Sites and Pavement Types

Site/Station	Location	Route	Milepost	Pavement Type	Test Type
1	Benson	I10W	300.07	5-layer	10 tests/90 ft.
2	Winslow	I40E	260.21	4-layer	10 tests/90 ft.
3	Minnetonka	I40E	261.78	4-layer	10 tests/90 ft.
4	Dead River	I40E	317.06	4-layer	10 tests/90 ft.
5	Flagstaff	I17N	337.00	4-layer	10 tests/90 ft.
6	Crazy Creek	I40E	323.78	4-layer	10 tests/90 ft.
7	Sunset Point	I17N	251.41	5-layer	10 tests/90 ft.
8	Seligman	I40W	131.71	4-layer	10 tests/90 ft.
9	Benson East	I10W	303.00	4-layer	10 tests/90 ft.
10	Jacob Lake	US89A	578.00	4-layer	10 tests/90 ft.
11	Morristown	US60W	120.00	4-layer	10 tests/90 ft.
12	McNary	US260E	369.00	5-layer	10 tests/90 ft.
13	Kingman I	I40E	59.00	4-layer	10 tests/90 ft.
14	Yucca	I40W	33.00	4-layer	10 tests/mile
15	Kingman II	I40E	24.00	4-layer	10 tests/mile
16	Tombstone	U80E	316.50	4-layer	10 tests/mile

TABLE 2 Layer Type and Thickness at Different Sites

Site/ Sta.	Mat	Layer 1 Thk (in)	Mat	Layer 2 Thk (in)	Mat	Layer 3 Thk (in)	Mat	Layer 4 Thk (in)	Mat	Layer 5 Thk (in)
1/1	AC	7	HB	2.5	AB	2	SB	12	SC-SM*	-
2/1	AC	12	BTB	3	SB	5	SM*	-	-	-
3/1	AC	11.5	BTB	2	SB	3	SM*	-	-	-
4/1	AC	8	CTB	4.5	SB	7	SM*	-	-	-
5/1	AC	9	AB	4	SB	12	-	-	-	-
6/1	AC	8	CTB	6	SB	6	SM*	-	-	-
7/1	AC	6	BS	4	SB	26	SGS	6	CL-CH*	-
8/1	AC	6	AB	6	SB	24	CH*	-	-	-
9/1	AC	6	AB	6	SB	18	SC-SM*	-	-	-
10/1	AC	9	HB	4	AB	4	SC-CH*	-	-	-
11/1	AC	4.25	AB	4	SB	15	-	-	-	-
12/1	AC	4.8	HB	2.2	AB	3	SB	6	-	-
13/1	AC	9.5	AB	4	SB	15	-	-	-	-
14/1	AC	4.0	AB	4	SB	9	-	-	-	-
15/1	AC	4.0	AB	4	SB	9	-	-	-	-
16/1	AC	3.0	AB	4	SB	15	-	-	-	-

* Subgrade Classification based on Unified Method.
Note: AC: Asphalt Concrete, HB: HMAC Base, BTB: Bituminous Treated Base, CTB: Cement Treated Base, AB: Aggregate Base, SGS: Subgrade Seal, SB: Sub Base (Select Material)

tural capacity of the existing pavement through fatigue analysis. Backcalculation of layer moduli was done with the Arizona Deflection Analysis Method (ADAM) developed by Hossain (*2*). ADAM uses the CHEVRON (*3,4*) computer program for pavement response analysis. A robust optimization routine iterates the moduli values to minimize the squared error between the peak measured deflections and calculated deflections at the same offsets. The backcalculated layer moduli were used to determine the tensile strain at the bottom of the asphalt concrete layer. The structural capacity of the pavement in terms of theoretical number of 18-kip ESALs was determined using the following equation for fatigue analysis. Hossain developed the analysis by modifying the ADOT overlay design fatigue relationship (*2*).

$$N = (2.265 \times 10^{-7})\,(1/e_{ac})^{3.84} \qquad (1)$$

where N is the theoretical number of 18-kip ESAL repetitions to fatigue failure and e_{ac} is the tensile strain at the bottom of the asphalt concrete layer (micro inch^2). Equation 1 is valid for an asphalt concrete temperature of 70°F. Figure 1 shows the flow chart of the analysis process.

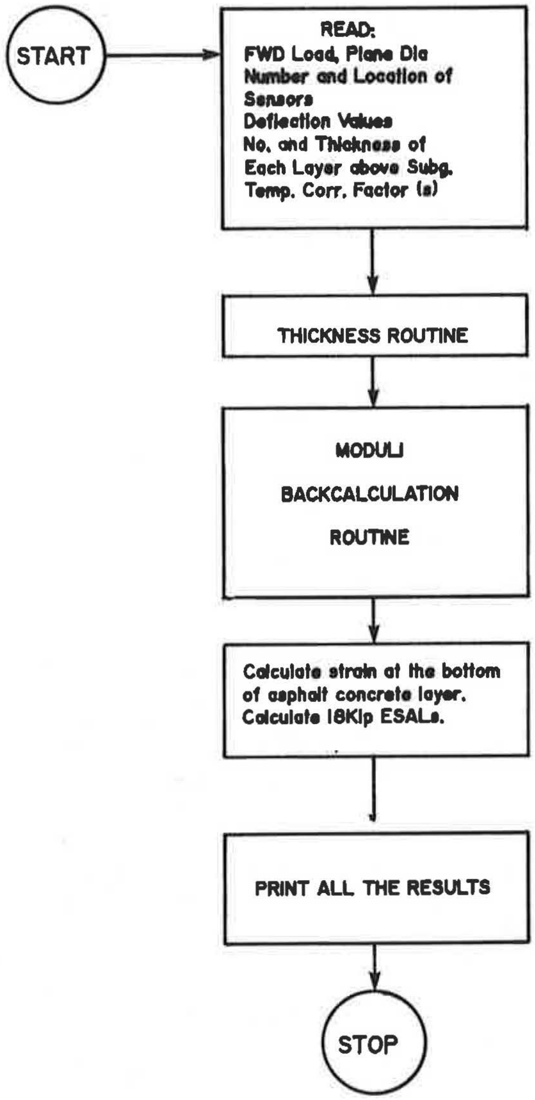

FIGURE 1 Flow chart of the analysis process.

SPATIAL VARIABILITY OF FWD DEFLECTION DATA

The estimated structural capacity of an existing pavement is affected by the spatial variability of the measured deflections. This variability is the result of equipment repeatability and spatial characteristics of the pavement structure and materials. Mamlouk et al. (*1*) concluded that equipment variability is insignificant compared with spatial variability. In this section, the spatial variability of FWD deflection data along a 90-ft section of 13 sites and along a 1-mi section of 3 sites is presented. The variation in sensor readings for the sites listed in Table 1 is presented in Table 3. The coefficients of variation for all the sensors varies from 2.80 percent to 41.7 percent for the 90-ft sections and 27 percent to 57 percent for the 1-mi sections. Variations of sensor readings are higher for the 1-mi section of the road than for the short road section. This spatial variability of deflection measurements reflects the variability of the structural response of the existing pavement sections and varying subgrade support along the roadway.

TABLE 3 Spatial Variability of Sensor Readings at Different Sites

Sensor	Mean Deflection (mils)	Standard Deviation (mils)	C.V. (%)	Average C.V. (%)
Site 1				
1	15.32	4.02	26.3	31.5
2	9.67	2.43	25.1	
3	4.92	1.25	25.5	
4	2.49	0.74	29.6	
5	1.44	0.51	35.4	
6	0.99	0.41	41.7	
7	0.70	0.26	36.9	
Site 2				
1	8.74	1.31	15.0	10.9
2	6.65	0.82	12.4	
3	4.74	0.47	9.90	
4	3.44	0.35	10.1	
5	2.63	0.24	9.20	
6	2.08	0.20	9.80	
7	1.70	0.17	9.90	
Site 3				
1	8.98	0.79	8.80	9.56
2	6.58	0.74	11.2	
3	4.64	0.49	10.60	
4	3.46	0.36	10.5	
5	2.69	0.25	9.40	
6	2.18	0.18	8.30	
7	1.80	0.15	8.10	
Site 4				
1	8.39	0.42	5.00	6.93
2	6.75	0.39	5.70	
3	5.10	0.29	5.60	
4	3.77	0.26	7.00	
5	2.74	0.17	6.20	
6	2.01	0.17	8.30	
7	1.45	0.16	10.7	
Site 5				
1	6.74	0.19	2.80	10.1
2	5.76	0.18	3.20	
3	4.48	0.25	5.60	
4	3.29	0.29	8.90	
5	2.33	0.28	12.1	
6	1.66	0.26	15.9	
7	1.11	0.25	22.2	
Site 6				
1	15.37	2.50	16.3	10.97
2	11.39	1.77	15.5	
3	7.62	0.93	12.2	
4	5.19	0.54	10.5	
5	3.73	0.28	7.40	
6	2.85	0.22	7.60	
7	2.34	0.17	7.30	
Site 7				
1	12.16	0.45	3.70	7.4
2	9.08	0.32	3.50	
3	5.96	0.21	3.50	
4	3.67	0.17	4.70	
5	2.25	0.15	6.70	
6	1.44	0.17	11.7	
7	0.98	0.18	18.1	

(*continued on next page*)

TABLE 3 *(continued)*

Sensor	Mean Deflection (mils)	Standard Deviation (mils)	C.V. (%)	Average C.V. (%)
Site 8				
1	18.93	6.13	32.4	18.9
2	13.71	3.23	23.5	
3	8.09	1.22	15.1	
4	4.96	0.71	14.4	
5	3.22	0.39	12.1	
6	2.36	0.38	16.2	
7	1.80	0.33	18.5	
Site 9				
1	16.96	1.40	8.20	7.5
2	11.23	0.70	6.20	
3	6.20	0.29	4.70	
4	3.32	0.13	3.90	
5	1.91	0.16	8.20	
6	1.31	0.10	7.90	
7	0.98	0.13	13.7	
Site 10				
1	26.70	6.14	23.0	12.5
2	6.31	1.34	21.2	
3	2.01	0.23	11.6	
4	1.18	0.10	8.90	
5	0.92	0.07	8.10	
6	0.72	0.05	7.60	
7	0.57	0.04	7.00	
Site 11				
1	13.98	1.62	11.6	11.1
2	6.47	0.76	11.7	
3	2.35	0.40	17.0	
4	1.25	0.16	13.1	
5	0.92	0.08	8.20	
6	0.74	0.07	8.70	
7	0.61	0.05	7.40	
Site 12				
1	18.29	1.66	9.10	11.2
2	13.31	1.29	9.70	
3	8.57	0.91	10.6	
4	5.45	0.61	11.2	
5	3.52	0.41	11.7	
6	2.54	0.31	12.2	
7	2.00	0.275	13.7	
Site 13				
1	12.28	2.00	16.2	13.84
2	6.91	1.09	15.8	
3	3.05	0.52	16.9	
4	1.48	0.23	15.3	
5	0.91	0.11	11.9	
6	0.68	0.07	10.4	
7	0.55	0.06	10.4	
Site 14				
1	9.56	2.84	29.7	35.44
2	6.81	2.17	31.8	
3	3.61	1.22	33.7	
4	1.99	0.70	34.9	
5	1.19	0.46	38.3	
6	0.82	0.31	38.1	
7	0.58	0.24	41.6	
Site 15				
1	6.62	2.54	38.4	49.30
2	4.62	2.00	43.3	
3	2.57	1.30	50.6	
4	1.48	0.83	56.0	
5	0.94	0.54	57.0	
6	0.68	0.35	52.0	
7	0.52	0.25	47.6	
Site 16				
1	13.71	3.64	26.6	32.20
2	5.01	1.42	28.3	
3	2.33	0.76	32.5	
4	1.47	0.48	32.5	
5	1.10	0.37	33.6	
6	0.88	0.32	36.4	
7	0.72	0.26	35.5	

As shown in Table 4, spatial variability for different sites was compared with several factors, including surface condition expressed in terms of percent cracking, coefficients of variation of backcalculated asphalt concrete moduli over the section, and calculated theoretical number of 18-kip ESALs the sections can carry before fatigue failure.

The percent of cracking data was extracted from the ADOT pavement management system inventory data base. These crack data are for 1,000 ft^2 at the milepost location and not for the entire pavement area. The validity of these data with respect to the pavement condition at the point where the deflection measurements were made is questionable. Variability in deflection measurements over a section of the road cannot be explained by the distress condition on the surface only and may depend on the other factors, such as subgrade type and moisture content and properties of other layers.

Because the deflection measurements were used to compute moduli and subsequently the number of allowable applications, variability in deflection measurements produced variability in the computed parameters. However, as shown in Table 4, the variability of the computed parameters actually increased at each step in the process. In every case, the coefficient of variability of the asphalt modulus was greater than for the measured deflections. The coefficient of variability of the computed allowable axle loads was greater than the variability in the moduli values.

EFFECT OF VARIATION IN LAYER MODULI ON STRUCTURAL CAPACITY

The variability in the backcalculated layer moduli affects the estimated structural capacity. In order to study and quantify the effect of layer moduli on the estimated structural capacity, the factorial design shown in Figure 2 with the levels of layer moduli presented in Table 5 was analyzed. Five levels of AC (surface), AB (aggregate base), and SM (select material/subbase) modulus were selected for each of the three predefined pavement categories: weak, medium, and stiff. Figure 3 shows

TABLE 4 Comparison of Spatial Variability of Measured Deflections with Estimated Structural Capacity

Base Type	Average Cracking (%)	Avg. C.V. All Sensors (%)	Avg. C.V. EAC (%)	Avg. C.V. Calculated N18 (%)
AB	2.40	12.2	28.0	69.7
AB (long span)	20.3	39.0	22.3	57.7
HB	0.25	15.6	31.0	56.4
BTB	0.00	10.2	20.0	47.3
CTB	0.00	9.00	31.0	60.4

Note: AC: Asphalt Concrete, HB: HMAC Base, BTB: Bituminous Treated Base, CTB: Cement Treated Base, AB: Aggregate Base

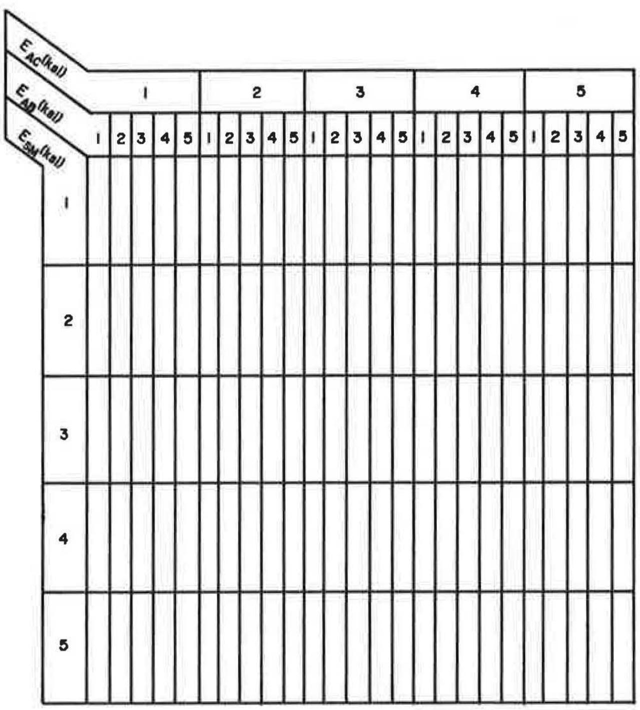

FIGURE 2 Factorial to study the effect of layer moduli on the structural capacity of the pavements.

the cross-section for each type of pavement. The CHEVRON program *(4,5)* was run for each of the $3 \times 5^3 = 625$ pavements. Tensile strain at the bottom of the asphalt concrete layer was calculated for a 9,000-lb wheel load and 100 psi tire pressure.

The tensile strain was substituted in Equation 1 to estimate the number of 18-kip ESALs the pavement can carry before fatigue failure. The correlation coefficients between layer moduli and 18-kip ESAL for weak, medium, and stiff pavements were developed. Significance of correlations were evaluated with the Student's *t*-tests. The moduli for all the layers are significantly correlated with the estimated 18-kip ESALs for each pavement type. The correlation coefficients are higher than 0.5 for the AC and AB layers. For the weak pavement, the AB modulus appears to have a stronger effect than the AC modulus on the calculated 18-kip ESALs, whereas for medium and stiff pavement, the AC modulus appears to affect the calculation of 18-kip ESALs more than any other layer moduli. To study the effect of interaction of layer moduli, the factorial in Figure 4 was developed for medium stiff pavement.

The following model was proposed for the analysis of variance (ANOVA) to describe the variation in calculated 18-kip ESALs:

$$\begin{aligned} N18_{ijk} = {} & \mu + ACE_i + ABE_j + SME_k \\ & + ACEABE_{ij} + ACESME_{ik} \\ & + ABESME_{jk} + ACEABESME_{ijk} \\ & + \varepsilon_{(ijk)} \qquad (2) \\ & i = 1, \ldots, 3 \\ & j = 1, \ldots, 3 \\ & k = 1, \ldots, 3 \end{aligned}$$

where

$N18_{ijk}$ = theoretical 18-kip ESALs calculated at *i*th level of AC modulus, *j*th level of AB modulus, and *k*th level of SM modulus;
μ = overall mean;
ACE_i = effect of *i*th level of (fixed) treatment AC modulus;
ABE_j = effect of *j*th level of (fixed) treatment AB modulus;
SME_k = effect of *k*th level of (fixed) treatment AB modulus;
$ACEABE_{ij}$ = interaction effect between *i*th level of AC modulus and *j*th level of AB modulus;
$ACESME_{ik}$ = interaction effect between *i*th level of AC modulus and *k*th level of SM modulus;
$ABESME_{jk}$ = interaction effect between *j*th level of AB modulus and *k*th level of SM modulus;
$ACEABESME_{(ijk)}$ = interaction effect between *i*th level of AC modulus, *j*th level of AB modulus, and *k*th level of SM modulus; and
$\varepsilon_{(ijk)}$ = random within error. $\varepsilon_{(ijk)}$ is assumed to be normally and independently distributed with mean zero and variance σ^2.

TABLE 5 Levels of Layer Moduli Used to Study Modulus Variability Effect

Pavement Type	Layer Type	Modulus (ksi) at Level 1	2	3	4	5
	AC	325	488	650	819	975
STIFF	AB	20	30	40	50	60
	SM	12.5	19	25	32.5	37.5
	AC	225	337.5	450	562.5	675
MEDIUM	AB	15	22.5	30	37.5	35
	SM	10	15	20	25	30
	AC	125	187.5	250	312.5	375
WEAK	AB	10	15	20	25	30
	SM	5	7.5	10	12.5	15

Note: AC: Asphalt Concrete, AB: Aggregate Base, SM: Select Material

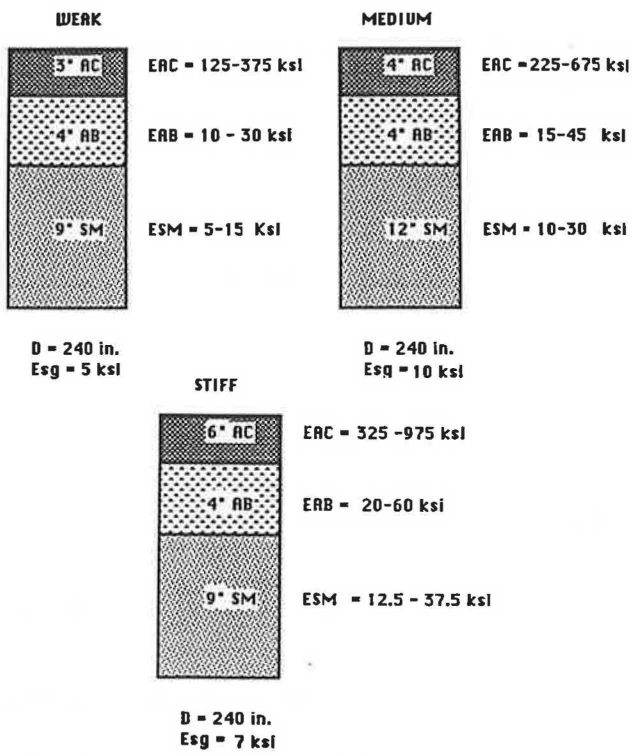

FIGURE 3 Cross-sections for different pavement types.

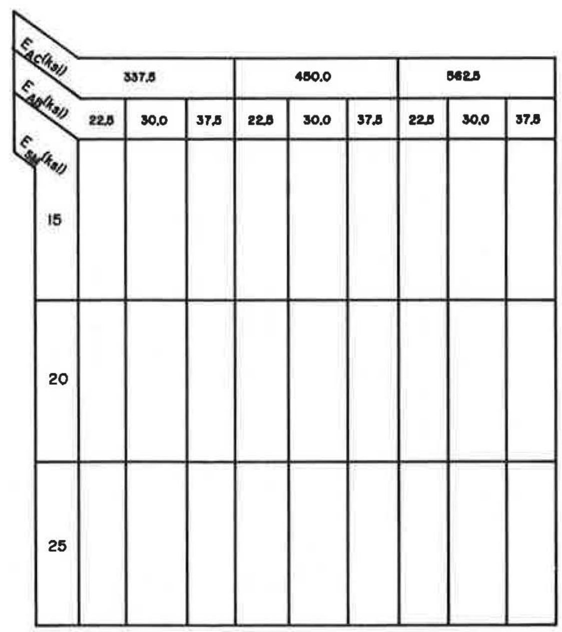

FIGURE 4 Factorial for medium stiff pavement.

It is important to note that in Equation 2 the subscripts for *ACEABESME* and ε are the same, indicating the effects due to interaction of all the layer moduli and the error are confounded. This is necessary because of absence of any replication.

From the ANOVA all the main factors or layer moduli and two factor interactions (or interaction between two layer moduli) were significant at 5 percent. The three-way interaction among layer moduli, which was used as the error term, might be significant. Thus the model described in Equation 2 is not adequate to capture all the variation. However, physical interpretation of response of the flexible pavement system to the applied load also supports the assertion that not only the layer moduli but also the interactions between the layer moduli dictate the structural response of a multilayer system. The precision needed for resilient moduli test of pavement materials to determine the effect of test variance on estimated structural capacity should be examined in further research.

In the backcalculation of layer moduli variation in estimated layer modulus from actual values is compensated by the variations in the moduli of other layers in the structure, providing a compensating effect. The resultant capacity of the pavement system with backcalculated layer moduli remains essentially unchanged (*5*). The significant interactions of layer moduli in the ANOVA analysis supports the compensating effect concept in the backcalculation analysis.

SPATIAL VARIABILITY OF BACKCALCULATED LAYER MODULI

Deflection data from each station of the sites listed in Table 1 were analyzed to define the spatial variability of the backcalculated layer moduli. The mean, standard deviation, and coefficient of variation of the layer moduli are presented in Table 6. The average coefficient of variation for layer moduli for the short-span sections varies from 15 percent to 46 percent with a mean value of 29 percent. The average coefficient of variation for layer moduli for three sites with ten deflection

TABLE 6 Spatial Variability of Backcalculated Layer Moduli

Site	Layer	Mean (ksi)	St. Dev. (ksi)	C.V. (%)	Avg. CV (%)	Range	
						Min.	Max.
1	AC	155	79.5	50		110	343
	BS	66	10.5	16		60	87
	AB	24	11.8	49	29	10	38
	SM	11	1.3	12		10	14
	SG	14	2.4	16		12	19
2	AC	200	51.5	26		125	253
	BTB	205	83	40	35	152	433
	SM	21	10.9	51		10	46
	SG	19	4	21		12	23
3	AC	217	28.3	13		186	274
	BTB	189	14.3	7.5	15	171	224
	SM	37	10.8	29		13	46
	SG	19	1.8	9		17	23
4	AC	389	60	15		362	516
	CTB	438	143	33	15	254	692
	SM	29	1.4	5		27	31
	SG	12	0.81	7		11	13
5	AC	349	52	15		287	390
	AB	65	2.7	41	18	61	69
	SM	34	3.0	9		27	38
	SG	14	1.1	7.5		13	15
6	AC	142	65	46		64	271
	CTB	87	67	76	46	60	275
	SM	13	5	42		10	24
	SG	13	3	20		10	14
7	AC	419	11	31		402	436
	BS	101	21	20	19	77	125
	SM	17	2.0	12		15	20
	SG	13	1.4	14		10	15
8	AC	180	100	56		41	302
	AB	38	13	34	32	12	48
	SM	15	4	25		10	23
	SG	9	1.2	14		7	10
9	AC	237	56	23	23	126	309
	AB	32	14	45		17	58
	SM	14	2.5	18		12	18
	SG	14	0.8	6		13	16
10	AC	265	23	9		233	279
	BS	270	38	14		230	358
	SM	12	6	50	24	10	27
	SG	48	10	21		32	60
11	AC	227	38	17		181	307
	AB	66	25	38	29	33	96
	SM	26	11	42		15	52
	SG	47	9	18		31	60
12	AC	199	63	32		141	332
	BS	60	25	42		20	94
	AB	18	13	74	43	10	41
	SM	15	7	44		10	24
	SG	12	3	25		9	16
13	AC	88	26	30		54	125
	AB	31	18	58	45	10	56
	SM	22	13	60		10	55
	SG	44	14	32		26	60
14	AC	1028*	194	19		814	1371
	AB	80	13	16	22	62	1002
	SM	41	17	42		10	78
	SG	32	13	40		19	60
15	AC	1229*	160	13		1059	1523
	AB	77	8	10	22	56	85
	SM	28	15	54		10	48
	SG	24	2.5	10		21	26
16	AC	232*	82	35		175	423
	AB	49	20	40	34	30	915
	SM	38	15	39		22	72
	SG	37	7	20		29	53

*** Modulus was not corrected due to very high correction factor**

Note: AC: Asphalt Concrete, BS: Bituminous Surface, BTB: Bituminous Treated Base, CTB: Cement Treated Base, AB: Aggregate Base, SM: Select Material tests per mile varies from 22 percent to 34 percent with a mean value of 28 percent. Interestingly the average coefficients of variation for the two data sets are almost equal.

The average coefficient of variation of the AC modulus for nine sites is 28 percent, whereas for the base modulus the average coefficient of variation is 36 percent. These variations strongly affect the structural capacity determined from fatigue analysis. The interaction of high asphalt concrete modulus and base modulus tends to produce low asphalt concrete tensile strain and consequently a high number of 18-kip ESAL applications. The opposite is true for the interaction of low asphalt concrete modulus and low base modulus. As a result, the computed structural capacity is highly variable.

Table 7 compares average coefficient of variation of layer moduli to the estimated structural capacity for the 1-mi section sites listed in Table 1. Variation of asphalt concrete and base moduli is highly magnified in the calculation of structural capacity.

It is to be noted that there are inherent sources of errors in the backcalculation of the moduli itself. Layer thicknesses are variable and sometimes spatially unknown, as are depths to effective rigid layers. Both affect the relative magnitudes of the backcalculated layer moduli. However, in this analysis, actual layer thicknesses determined from the cores were used and the depths to rigid layer were estimated (*6*).

EFFECT OF TESTING FREQUENCY ON ESTIMATED STRUCTURAL CAPACITY

Researchers differ on the issue of required number of FWD tests needed for structural characterization of existing pavements. The AASHTO Guide for the Design of Pavement Structures (*7*) recommends a spacing of 300 to 500 ft when accurate historic data for a section are unavailable. When accurate historic data are available, the guide recommends 10 to 15 test points per mile. No analysis was presented in the guide to support this recommendation. ARE Inc. (*8*) recommended dynaflect tests every 100 ft when the subgrade is nonuniform. For uniform subgrade, the spacing can be extended to 250 ft. Karan et al. (*9*) used a spacing of 6 deflection tests per kilometer (roughly 10 per mile). Koole (*10*) proposed a spacing of 66 ft for an overlay design method. ADOT studied the variability of dynaflect deflection data and concluded that one measurement per mile is required for network level pavement management system (*11*). Shell Research (*12*) recommends one FWD test per 85 to 165 ft. Lytton et al. (*13*) concluded that a minimum of 5 tests per mile is required at the network level to rank pavement sections. Project level evaluation requires one test every 100 to 300 ft in each wheel path.

TABLE 7 Effect of Variation of Backcalculated Layer Moduli on Estimated Structural Capacity

	Average Coefficient of Variation (%)				
Site	EAC	EAB	ESM	ESG	N18
14	13	10	54	10	23
15	19	16	42	40	44
16	35	40	39	20	106

Effect of Testing Frequency Over a Short Section

Of the projects listed in Table 1, FWD data collected on 9 projects were analyzed to determine the effect of the number of FWD tests over a 90-ft section of the pavement. The number of tests were sorted in the following fixed fashion.

Number of Tests	*Location*
10	Beginning of project and 10-ft intervals
5	Beginning of project and 20-ft intervals
3	Beginning of project and 30-ft intervals
1	Beginning of project

It was assumed that 10 FWD tests per 90 ft represented the standard or truth for this particular experiment. It was also inherently assumed that the samples were a random selection from the population of pavements.

The backcalculated asphalt concrete modulus (*EAC*) and the structural capacity in terms of estimated number of 18-kip ESALs (*N*18) the sections can carry were selected as the response parameters. Table 8 summarizes the mean, pooled standard deviation, and coefficient of variation for the above parameters for each sample size.

The size of the coefficients of variation for the backcalculated asphalt concrete moduli is much smaller than anticipated. Linear regressions were conducted between the mean 18-kip ESALs derived by taking different sample sizes. The coefficient of determination R^2 for the linear regression between estimated 18-kip ESALs from one test per 90 ft and average estimated 18-kip ESALs from ten tests per 90 ft was 0.94. It appears that estimated 18-kip ESALs from 1 FWD test per 90 ft closely approximate the average of estimated 18-kip ESALs from 10 tests per 90 ft.

The assumed linear relationship between backcalculated asphalt concrete modulus and 18-kip ESALs from one FWD test per 90 ft and from ten FWD tests per mile was verified by the ANOVA. The relationship appeared to be significant at a 5 percent level of significance. In addition, paired *t*-tests were conducted between the values of this parameter from 1 test per 90 ft and 10 tests per 90 ft. No significant difference was detected at a 5 percent level of significance. It appears

TABLE 8 Summary Statistics of Backcalculated AC Modulus and Structural Capacity Corresponding to Different Testing Frequency

No. of Tests per Site	EAC (ksi)	N18 (millions)
Mean		
10	222	41.7
5	221	42.8
3	212	42.9
1	234	42.4
Pooled Standard Deviation of Group		
10	48.3	65.2
5	57.2	17.0
3	55.4	21.8
Coefficient of Variation		
10	21.7	156.3
5	25.9	39.70
3	26.1	50.80

NOTE: $N = 9$.

that the number of tests does not affect the estimated structural capacity for a short section of the pavement (up to 90 ft).

Effect of Testing Frequency Over a Long Section

FWD deflection measurements were taken at the beginning of the project and at nine locations at a uniform interval of 0.1 mi for Sites 14 through 16 listed in Table 1. To determine the effect of testing frequency, 7, 5, and 3 tests were randomly selected out of 10 tests. The current ADOT practice is to take three tests per mile for overlay design.

The FWD data were used to backcalculate the layer moduli and estimate the number of 18-kip ESALs based on fatigue criteria. Table 9 shows the mean, standard deviation, and coefficient of variation of the number of 18-kip ESALs (N18) for the three sites for each random selection. The coefficient of variation is nearly equal for seven and five tests per mile. However, for three tests per mile the variation is high.

It is apparent that the Kolmogorov-Smirnov (K-S) (*14*) test verified that the data were normally distributed. Student's *t*-tests were conducted to detect the difference in means of 18-kip ESALs calculated at each site corresponding to different testing frequency. No significant difference was detected between means of 18-kip ESALs calculated from 10, 7, 5, or 3 tests. It may be noted that because of the high standard deviation associated with each mean, the pooled standard deviation during the *t*-tests was also high. This high standard deviation was responsible for the low *t*-statistic during mean testing.

The results of the *t*-test suggest that 3 tests per mile are as good as 10 tests per mile to characterize the pavement structurally. However, the coefficients of variation of 18-kip ESALs for three tests per mile appeared to fluctuate widely compared with those for five tests per mile. Student's *t*-tests were also conducted between means of 18-kip ESALs from seven, five, and three tests per mile. Significant difference was observed between means of 18-kip ESALs computed from five and three test results for a single run for Site 15. However, in two other runs for Site 15, no difference was detected.

Results suggest that 18-kip ESALs estimated from five tests per mile would be a viable choice for a project level decision on structural capacity estimation for the existing pavements. The failure of the Student's *t*-test to detect a statistically significant difference between three and five tests per mile is due to the variability associated with three tests per mile. Hence, this level of testing was rejected in favor of five tests per mile. However, FWD data collected on a larger number of sections should be analyzed to further support or reject this conclusion. Other statistical methods can also be employed in the data analysis.

CONCLUSIONS

In this paper the variability of the structural capacity determination by the mechanistic-empirical method was presented with respect to FWD input deflection data in the backcalculation scheme, backcalculated layer moduli, and the number of FWD tests over a section of the road.

As can be expected, sensor readings on a longer section of the road show more variability than on a shorter section. This variability in sensor readings was magnified when the layer moduli were backcalculated (i.e., small variability in sensor data over a section of a pavement would result in high variability of calculated layer moduli). However, this variability was independent of the length of the section of the roadway over which deflection testing was done.

All the layer moduli and their interaction affect the calculated structural capacity. The variation in backcalculated layer moduli was magnified when the number of 18-kip ESALs the pavement can carry before fatigue failure was estimated. Interaction of high asphalt concrete modulus and base modulus tends to produce low asphalt concrete strain and consequently a high number of 18-kip ESAL applications. The opposite is true for interaction of low asphalt concrete modulus and base modulus. As a result, the computed structural capacity becomes highly variable, especially when the number of tests done on a long section of the pavement is small.

TABLE 9 Effect of Testing Frequency on Variation of Structural Capacity

		N18 (millions)								
		Run 1			Run 2			Run 3		
Site	No. of Tests/ Mile	X	σ	CV (%)	X	σ	CV (%)	X	σ	CV (%)
14	10	1862	810	44	1862	810	44	1862	810	44
	7	1858	805	43	2180	686	32	2179	688	32
	5	2184	826	38	2126	904	43	1598	696	44
	3	1574	1155	73	2729	184	7	2061	1008	49
15	10	1197	275	23	1197	275	23	1197	275	23
	7	1170	292	25	1159	300	26	1232	306	25
	5	1008	195	19	1205	332	28	1166	300	26
	3	1460	134	9	1450	145	10	1283	408	32
16	10	18	19	106	18	19	106	18	19	106
	7	18	23	128	18	23	128	17	23	135
	5	26	25	96	15	8	53	10	5	50
	3	31	33	107	8	3	38	13	5	39

The frequency of testing did not affect the estimated 18-kip ESALs over a short section of the pavement. However, for long sections it affects the mean estimated 18-kip ESALs. For a mile-long section, five FWD tests were found to be a viable choice for estimation of 18-kip ESALs. However, the coefficient of variation of estimated 18-kip ESALs over a long section may or may not decrease with an increasing number of tests.

ACKNOWLEDGMENTS

The authors wish to acknowledge the financial support for this study provided by the ADOT's Arizona Transportation Research Center. The computing facilities were provided by Arizona State University. Thanks are due to Larry Scofield and John Eisenberg of ADOT and Dwayne Rollier and Rohan Perera of Arizona State University for their help in this study.

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Publication of this paper sponsored by Committee on Strength and Deformation Characteristics of Pavement Sections.