Alternative Method for Temperature Correction of Backcalculated Equivalent Pavement Moduli

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The Direct Structural Capacity Method as described in the AASHTO *Guide for the Design of Pavement Structures* provides a quick and simple method for analyzing pavement deflections that are collected from nondestructive testing. However, results taken from 13 sites tested bimonthly during an 18-month period showed considerable variation with temperature even after the recommended AASHTO temperature correction procedures were applied. An alternative temperature correction procedure is derived from asphalt cement (AC) concrete temperature-modulus relationships and elasticity equations used to calculate composite pavement modulus. The alternative correction procedure requires knowledge of the ratio of AC bound layer thickness to non-AC bound layer thickness, the estimated average pavement temperature, and the ratio of AC bound layer stiffness to non-AC bound layer stiffness. Although the stiffness ratio is not known before temperature correction in a Direct Structural Capacity Method analysis, it is shown that the correction procedure is relatively insensitive to errors in the estimated stiffness ratio. Therefore, an estimate of the stiffness ratio is sufficiently precise for most pavement analyses. When the alternative procedure is applied to the data from the 13 test sites, it is shown to provide more uniform results with varying temperature.

The AASHTO *Guide for the Design of Pavement Structures* (*1*) introduces the Direct Structural Capacity Method to determine composite pavement stiffness from the analysis of flexible pavement deflections obtained through nondestructive testing (NDT). Integral to the analysis of flexible pavement using NDT data is the correction of the results to account for the temperature sensitivity of asphalt cement (AC) concrete. The AASHTO guide recommends the use of curves derived from studies conducted by Southgate and Deen (*2*) to correct the measured deflection at the test temperature to a deflection at a standard temperature of 70°F. The curves correspond to various pavement types and were determined empirically using AASHO Road Test data for the purpose of correcting Benkelman beam deflections. This correction method was applied to deflections collected with a falling weight deflectometer (FWD) during an 18-month period at various test sites in South Carolina. Even after the AASHTO temperature corrections were applied to the deflection data, the backcalculated pavement stiffnesses were observed to vary considerably in relation to the estimated pavement temperature at the time of FWD testing.

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To achieve more stable backcalculated pavement stiffnesses with varying pavement test temperature and to make such stiffnesses more representative of the standard pavement temperature of 70°F, an alternative temperature correction technique is proposed. The alternative technique is derived from AC concrete temperature-stiffness relationships and the elasticity relations used to calculate composite modulus. This alternative technique applies a correction factor to the calculated composite modulus.

DIRECT STRUCTURAL CAPACITY METHOD

The Direct Structural Capacity Method is derived in Appendix PP of the AASHTO guide (*1*). This method is also referred to as NDT Method 2 in the guide. Presented here is a brief overview of the procedure. The Direct Structural Capacity Method models a flexible pavement structure as a multilayered linear elastic system, having layer characteristics of thickness (h_i), Young's modulus (E_i), and Poisson's ratio (μ_i). The total pavement thickness (h_t) is transformed into one equivalent layer with an equivalent, or composite, Young's modulus (E_e), creating a two-layer (pavement/subgrade) system. Using temperature-corrected underplate surface deflections obtained through field measurements with an FWD or other NDT device, the value of E_e is calculated iteratively using a Bummister linear elastic solution. The values of E_e and h_t are then converted to a structural number (SN) value using a simple transfer function (*1*). The SN value is used to evaluate the pavement's in situ structural capacity using the AASHTO pavement design equations. The recommended AASHTO technique to correct for temperature alters the total underplate (at load center) deflection value before analysis according to the estimated average pavement temperature and pavement type. The temperatures at different depths of the pavement are derived from the pavement surface temperature at the time of the test and the average air temperature for the 5 days preceding the test date using Figure 1. Once the mean temperature of the AC bound layer has been estimated using the average of the surface, midpoint, and bottom layer temperatures, Figure 2 is used to determine the deflection-temperature adjustment factor for the appropriate pavement type. The product of this factor and the observed underplate deflection is an estimate of the deflection at 70°F. The pavement stiffness is then backcalculated using the corrected deflection.

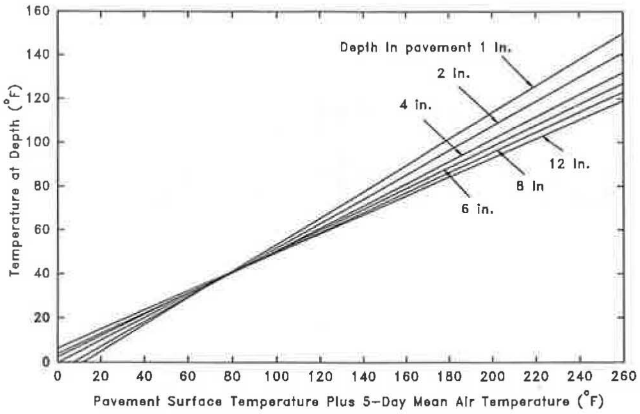
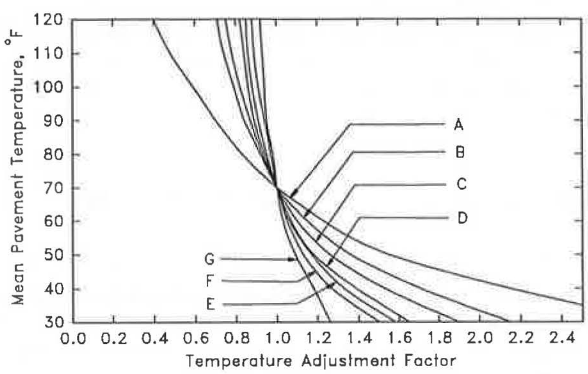


FIGURE 1 Variation of pavement temperature with depth and surface temperature plus 5-day mean air temperature (*1*, Appendix L).



Base Material	Curve (Base Thickness)
Asphalt (Full Depth)	A (All thicknesses)
Asphalt (Deep Strength)	B (4" of granular subbase)*
Portland Cement Concrete	G
Granular (Non-Stabilized)	C (6"); D (12"); E (20"); F(25")
Cement Treated Base Sound Cracked	 D (4"); E (8") C (4"); D (8")

* If more than 4" of granular material present use "Granular (Non-Stabilized)" base material category.

FIGURE 2 AASHTO recommended temperature adjustment factors (*1*, Appendix L).

ALTERNATIVE TEMPERATURE CORRECTION METHOD

It is important to note that Figure 2 does not differentiate between pavements with different AC bound layer thicknesses. For instance, if a pavement has 2 in. of AC concrete over a 6 in. granular base or 8 in. of AC concrete over a 6 in. granular base, Curve C on Figure 2 is the recommended correction curve. To improve the AASHTO procedure an adjustment factor was developed that estimates the change in the overall pavement stiffness from the change in the AC bound layer stiffness and the geometry of the pavement. To calculate this factor the average pavement temperature, the ratio of AC bound layer thickness to non-AC bound thickness, and the modular ratio of AC bound to non-AC bound layers are used.

The composite modulus of an n-layer pavement system is calculated with the following equation (*1*, Appendix PP).

$$E_e = \left[\sum_{n-1}^{i} \frac{h_i}{h_t} \times \sqrt[3]{\frac{E_i(1 - \mu_e^2)}{(1 - \mu_i^2)}} \right]^3 \tag{1}$$

For a three-layer system, (AC bound layer, non-AC bound layer, and subgrade), Equation 1 may be written as

$$E_e = \left[\frac{\sqrt[3]{\dfrac{E_1(1 - \mu_e^2)}{(1 - \mu_1^2)}} + \dfrac{1}{x} \sqrt[3]{\dfrac{\dfrac{E_1}{z}(1 - \mu_e^2)}{(1 - \mu_2^2)}}}{\left(1 + \dfrac{1}{x}\right)} \right]^3 \tag{2}$$

where

μ_1 = Poisson's ratio of AC bound layer,
μ_2 = Poisson's ratio of non-AC bound layer,
μ_e = Poisson's ratio of the equivalent (or composite) layer,
E_1 = Young's modulus of AC bound layer,
x = ratio of AC bound layer thickness to non-AC bound layer thickness (h_1/h_2), and
z = ratio of Young's modulus of the AC bound layer to Young's modulus of the non-AC bound layer (E_1/E_2).

Several functions have been developed to estimate the variation of the modulus of AC concrete with temperature. An approximation taken from the Asphalt Institute (*3*) and recommended for correction of NDT results (*4*) is

$$\begin{aligned} \text{Log}E_{\text{std}} = {} & \log E_{\text{field}} + 0.028829 P_{200} \left[\frac{1}{(f_o)^{\lambda}} - \frac{1}{(f)^{\lambda}} \right] \\ & + 0.000005 \sqrt{P_{\text{ac}}} \left[(t_o)^{r_o} - (t)^r \right] \\ & - 0.00189 \sqrt{P_{\text{ac}}} \left[\frac{(t_o)^{r_o}}{(f_o)^{1.1}} - \frac{(t)^r}{(f)^{1.1}} \right] \\ & + 0.931757 \left[\frac{1}{(f_o)^n} - \frac{1}{(f)^n} \right] \end{aligned} \tag{3}$$

where

λ = 0.17033,
n = 0.02774,
t = test temperature (degrees Fahrenheit),
f = loading frequency (hertz),
t_o = standard temperature,
f_o = standard frequency,
P_{ac} = percent AC by weight of the mix,
E_{field} = AC concrete modulus at the standard temperature and frequency,
r_o = 1.3 + 0.49825 log(f_o),
r = 1.3 + 0.49825 log(f), and
P_{200} = percent aggregate passing the No. 200 sieve.

Equation 3 may be simplified. A value of P_{ac} = 5.7 percent was selected as typical (*5*). Typical FWD load duration is approximately 30 to 40 msec (*6*). Using the approximation

$$f = \frac{1}{2t} \tag{4}$$

where t = FWD load duration, an approximate typical frequency of 15 Hz is indicated for both test and standard conditions (*4*). Using these values and a reference temperature of 70°F, Equation 3 can be reduced to the following equation.

$$\text{Log}E_{\text{std}} = \log E_{\text{field}} - 0.0002175[(t_o)^{1.886} - (t)^{1.886}] \tag{5}$$

Rewriting Equation 5 results in the following:

$$\frac{E_{\text{std}}}{E_{\text{field}}} = 10^{-(.0002175[(t_o)^{1.886} - (t)^{1.886}])} \tag{6}$$

Another correction, developed by Ullidtz (*7*), is based on backcalculation of moduli from AASHO Road Test deflections. This relationship for asphalt temperatures above 35°F is given by

$$E(t) = 2.18 \times 10^6 \text{ psi} - 1.15 \times 10^6 \text{ psi} \times \log\left(\frac{(t°\text{F} - 32)}{1.8}\right) \tag{7}$$

where $E(t)$ is the AC concrete modulus (pounds per square inch) at the test temperature (t) (degrees Fahrenheit). After solving for $E(70°\text{F}) = E_{\text{std}}$ and setting $E(t) = E_{\text{field}}$, Equation 7 may be rewritten as

$$\frac{E_{\text{std}}}{E_{\text{field}}} = \left[3.319 - 1.751 \times \log\left(\frac{(t°\text{F} - 32)}{1.8}\right)\right]^{-1} \tag{8}$$

Ullidtz (*7*) notes that some AC concrete modulus-temperature relationships developed from laboratory tests indicate unrealistically low modulus values at high temperatures, whereas in the field the lower limit of AC concrete modulus is determined by the unbound modulus of the aggregate. A comparison of modular ratios derived from Equations 6 and 8 is shown in Figure 3. The values derived from Equation 6 are considerably lower than those predicted by Equation 8 at temperatures higher than 100°F. However, as the pavement temperature approaches 32°F, the asphalt modulus predicted by Equation 8 approaches infinity. The Ullidtz formula (*7*) is not recommended when the pavement temperature is below approximately 35°F. Based on the authors' experience in the backcalculation of pavement stiffness, Equation 8 appears to provide the best prediction of in situ pavement stiffness for this type of analysis for AC bound layer temperatures above 35°F. Lytton et al. (*4*) discuss several other AC modulus temperature correction relationships that could also be used with this procedure.

The ratio of AC bound layer modulus at the reference temperature (70°F) to AC bound layer modulus at field temperature may be applied in a dimensionless form of Equation 2. To derive this ratio, Equation 2 is evaluated twice. The first evaluation sets $E_e = E_{e\ \text{std}}$ = the composite pavement modulus at the reference temperature and $E_1 = E_{\text{std}}$ = AC bound layer modulus at 70°F. The second evaluation sets $E_e = E_{e\ \text{field}}$ = the composite pavement modulus at the field temperature and $E_1 = E_{\text{field}}$ = AC bound layer modulus at the field temperature. The ratio of these two evaluations of Equation 2 may be written as

$$\frac{E_{e\ \text{std}}}{E_{e\ \text{field}}} = \left[\frac{1 + \sqrt[3]{\frac{E_{\text{std}}}{E_{\text{field}}} zx^3 \left(\frac{1 - \mu_2^2}{1 - \mu_1^2}\right)}}{1 + \sqrt[3]{zx^3 \left(\frac{1 - \mu_2^2}{1 - \mu_1^2}\right)}}\right]^3 \tag{9}$$

The derivation of Equation 9 assumes that neither the base course stiffness (E_2) nor the Poisson's ratios of the layers (μ_1 and μ_2) vary with temperature.

The use of Equation 9 with the Direct Structural Capacity Method requires that the modular ratio of the AC bound layer to the non-AC bound layer be specified. However, the Direct Structural Capacity Method (*1*) combines all layers before analysis and therefore provides no information on modular ratio. Fortunately, Equation 9 can be shown to be rather insensitive to variation in modular ratio. Figure 4 shows that at the bounds of thickness ratio (x) typically encountered in the field, estimations of z will be inaccurate by a factor of 10 and will result in errors in $E_{e\ \text{std}}$ of approximately 10 percent.

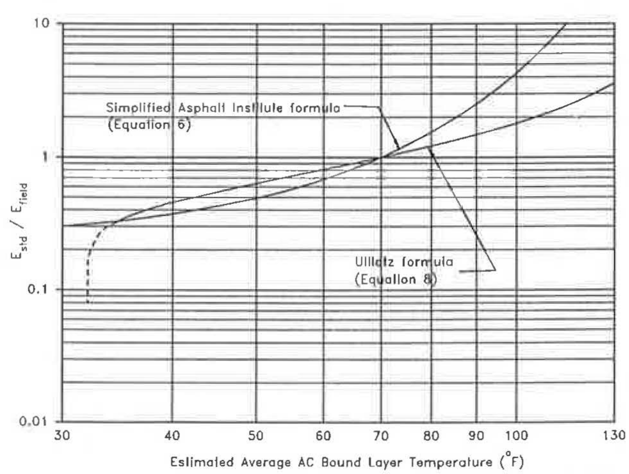


FIGURE 3 Variation of AC concrete modular ratio with temperature.

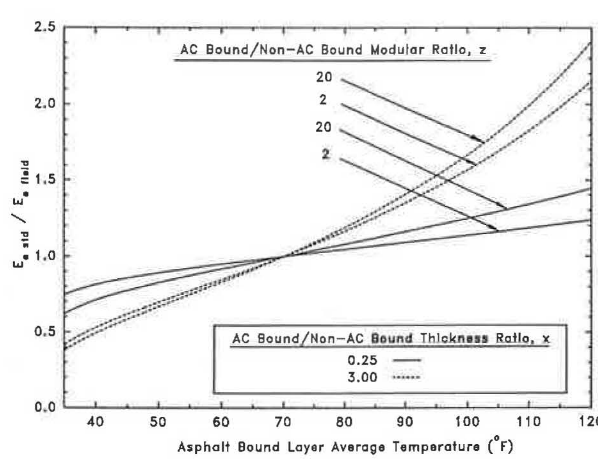


FIGURE 4 Variation of $E_{e\ std}/E_{e\ field}$ ratio calculated using Equation 9 with temperature.

Therefore, it is suggested that a reasonable estimate of modular ratio is adequate for most pavement deflection analyses.

To simplify the use of Equation 9, the ratio of AC bound to non-AC bound layer stiffness (z) may be estimated at 70°F and adjusted for temperature using Equation 8. Rewriting Equation 9 using this refinement gives

$$\frac{E_{e\ \mathrm{std}}}{E_{e\ \mathrm{field}}} = \left[\frac{1 + \sqrt[3]{\left(\frac{E_{\mathrm{std}}}{E_{\mathrm{field}}}\right) z_{\mathrm{std}} x^3 \left(\frac{1-\mu_2^2}{1-\mu_1^2}\right)}}{1 + \sqrt[3]{\frac{E_{\mathrm{std}}}{E_{\mathrm{field}}} z_{\mathrm{std}} x^3 \left(\frac{1-\mu_2^2}{1-\mu_1^2}\right)}} \right]^3 \tag{10}$$

where $z_{\mathrm{std}} = z$ at 70°F. The variation with temperature in composite modulus temperature adjustment factor ($E_{e\ \mathrm{std}}/E_{e\ \mathrm{field}}$) calculated using Equation 10 is shown in Figure 5. Equation 10 may also be written as

$$\frac{E_{e\ \mathrm{std}}}{E_{e\ \mathrm{field}}} = \left[\frac{1 + \sqrt[3]{z_{\mathrm{std}} x^3 \left(\frac{1-\mu_2^2}{1-\mu_1^2}\right)}}{1 + \sqrt[3]{\frac{E_{\mathrm{field}}}{E_{\mathrm{std}}} z_{\mathrm{std}} x^3 \left(\frac{1-\mu_2^2}{1-\mu_1^2}\right)}} \right]^3 \tag{11}$$

The use of Equation 10 in lieu of the AASHTO method for the adjustment of composite pavement modulus has the advantage of computational ease. Because Figure 2 is presented in graphical form only, using it in pavement deflection analysis software requires either interpolation from data files of values read from the figure or the creation of regression equations to simulate Figure 2. Figure 2 also requires some judgement to select the proper curve for deflection adjustment. Equation 10 may be integrated easily into pavement deflection analysis software and has clearly defined inputs.

Although Equation 10 theoretically may be used at any temperature, when used with Equation 8 it is valid only for pavement test temperatures above 35°F. In warm climates, this restriction is not a serious problem. In climates in which high pavement temperatures are not typically encountered, the use of Equation 6 or another AC concrete modulus-temperature relationship may be more appropriate.

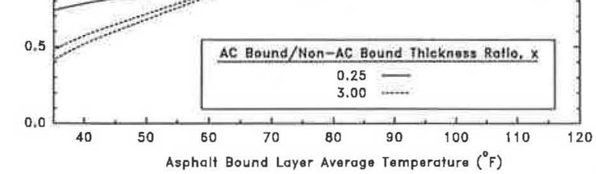


FIGURE 5 Variation of $E_{e\ std}/E_{e\ field}$ ratio calculated using Equation 10 with temperature.

COMPARISON OF RESULTS USING AASHTO AND PROPOSED METHODS

Theoretically, a perfect temperature correction technique should yield identical corrected pavement moduli ($E_{e\ \mathrm{std}}$) when the same site is tested repeatedly at different temperatures. Unfortunately, inaccuracy in the estimation of the actual pavement temperature, complex pavement and subgrade material constitutive properties, variations in as-built pavement thickness, and other factors prevent totally uniform results from being achieved in the field. The removal of the component of variation in backcalculated pavement stiffness and, subsequently, structural number due to the variation of AC bound layer modulus with temperature should reduce the overall observed variation seen with varying temperature. Therefore, to judge the effectiveness of a temperature correction procedure, the overall variation of the results can be analyzed. A superior correction procedure should reduce variation of deflection-based backcalculated pavement stiffness with test temperature.

In order to achieve satisfactory results, any temperature correction procedure must rely on accurate estimation of pavement temperature. Bissada and Guirguis (*8*) noted that the deflection characteristics of flexible pavements are not only controlled by average pavement temperature, but also by the temperature gradient within the pavement, especially at high temperatures. Dynaflect tests were performed during a 2-year period on test sections in Kuwait, and the most uniform results were achieved under conditions of zero temperature gradient. The difficulty of estimating temperature gradient in uninstrumented pavements illustrates one of many difficulties that practicing pavement engineers must consider before the results from NDT may be used to accurately design overlays for flexible pavements. It should be noted that Figure 1 is based on the typical temperature gradient at 1 p.m. The actual temperature gradient may vary significantly during the day, decreasing the accuracy of the pavement temperature estimates.

Thirteen sites at various locations throughout South Carolina were tested using a Dynatest FWD. Drop heights were selected to give peak impact loadings of approximately 6,000, 9,000, 12,000, and 15,000 pounds. Each test section was 500 ft long and was tested at 50 ft stations on 8 to 10 different dates between January 1989, and June 1990. Most tests were conducted between 9 a.m. and noon. Testing was not conducted under rainy conditions or when the road surface was visibly moist. Pavement surface temperatures were collected using an infrared temperature sensor mounted on the FWD trailer. A summary of the pavement structure at the test sites is presented in Table 1.

Using both the AASHTO and the proposed alternative (Equations 8 and 10) temperature correction methods, values of $E_{e\ \mathrm{std}}$ were computed for each station at each date using the Direct Structural Capacity Method. All Direct Structural Capacity Method computations here are based on FWD loadings of approximately 9,000 pounds. The value of z_{std} was

TABLE 1 Pavement Structure of Test Sites

Site Number	Road and County	Pavement Structure
1	I-26, Orangeburg Co.	11.3 inches AC Bound 14.0 inches Uniform Earth Base
2	SC-31, Charleston Co.	3.2 inches AC Bound 11.5 inches Fossiliferous Limestone Base
3	US-17, Charleston Co.	3.5 inches AC Bound 6.2 inches Fossiliferous Limestone Base
4	US-17, Charleston Co.	4.9 inches AC Bound 7.4 inches Fossiliferous Limestone Base
5	US-321, Fairfield Co.	6.2 inches AC Bound 3.5 inches Unbound Granular Material 12 inches Cement Stabilized Earth Base
6	SC-9, Chester Co.	10.8 inches AC Bound 6 inches Uniform Earth Base
7	I-26, Newberry Co.	9.0 inches AC Bound 16 inches Unbound Macadam Base
8	I-77, Richland Co.	18.1 inches AC Bound 6 inches Cement Stabilized Earth Subbase
9	S-1623, Lexington Co.	1.3 inches AC Bound 6 inches Unbound Macadam Base
10	I-20, Lexington Co.	12.4 inches AC Bound
11	US-76/378, Sumter Co.	6.6 inches AC Bound 12 inches Uniform Earth Base
12	US-76, Marion Co.	10.2 inches AC Bound
13	US-76/301, Florence Co.	7.0 inches AC Bound 4.5 inches Cement Stabilized Earth Base 8 inches Uniform Earth Base

assumed to be eight for all non-AC bound base pavements tested. The $E_{e\ \mathrm{std}}$ values were converted to structural number using the transfer function in the AASHTO guide (*1*). This function is

$$SN = 0.0043h_t \sqrt[3]{\frac{E_{e\ \mathrm{std}}}{(1 - \mu_e^2)}} \tag{12}$$

The equivalent Poisson's ratio (μ_e) equals 0.35 and $E_{e\ \mathrm{std}}$ is in pounds per square inch. Examples of E_e backcalculation results are shown in Figures 6 and 7.

Figure 6 compares backcalculated site average uncorrected and corrected composite pavement moduli (E_e and $E_{e\ \mathrm{std}}$) for Site 9. Each data point on this figure represents an estimated average pavement temperature or average backcalculated composite pavement modulus on a testing date. Figure 6 shows a case for which the proposed temperature correction yields $E_{e\ \mathrm{std}}$ values that remain much more stable with fluctuations in temperature. Interestingly, at Site 9, use of the AASHTO temperature correction significantly overcorrects composite pavement modulus. That is, overly high $E_{e\ \mathrm{std}}$ values are predicted for high pavement temperatures (for example, see testing date in June 1989, when the estimated average pavement temperature was higher than 100°F) and overly low $E_{e\ \mathrm{std}}$ values are predicted for low pavement temperatures (for example, see testing date in October 1989, when the estimated average pavement temperature was less than 50°F). Backcalculated site average uncorrected and corrected composite pavement moduli for Sites 8 and 12 are compared in Figure 7. Data from these sites clearly show that the proposed pavement temperature correction tends to yield $E_{e\ \mathrm{std}}$ values less

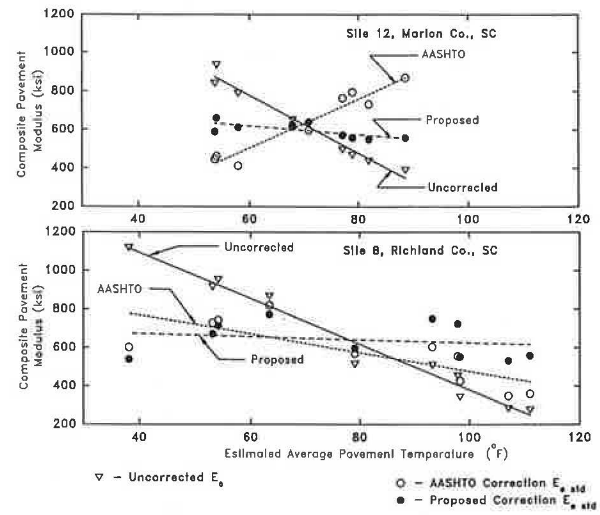


FIGURE 6 Variation of average backcalculated corrected and uncorrected composite pavement modulus and estimated average pavement temperature with date for Site 9.

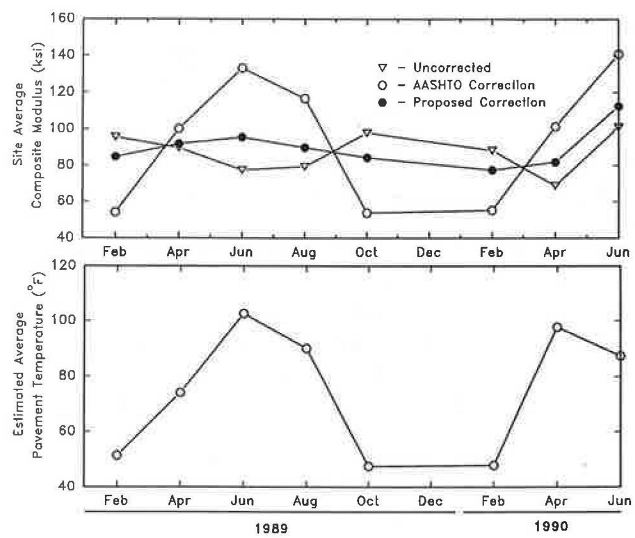


FIGURE 7 Variation of average backcalculated corrected and uncorrected composite pavement modulus with estimated average pavement temperature for Sites 8 and 12.

dependent on the average pavement temperature at the time of FWD testing.

For statistical evaluation, the structural number results for each deflection test at all testing dates were grouped according to the site and temperature correction method used and variances were computed. Each group of data was checked for normality using the Shapiro-Wilk statistic; all variances were then combined to compute an overall variance value. A summary is presented in Table 2 of the computed variation in structural number for each site and the results of statistical inferences described here. The assumption was made that the structural number of the sites remained constant during the 18-month test period. In reality, some slight reduction may have occurred as a result of traffic loading at all sites except

TABLE 2 Summary of Site Structural Number Variances

Site	s_2^a	s_1^b	Number of Tests Analyzed	s_1^b / s_2^a	Percent Probablity $s_1^b < s_2^a$
2	0.18900	0.22351	99	1.18261	79.605
3	0.01478	0.01336	99	0.90383	30.885
4	0.01086	0.00994	99	0.91565	33.179
5	0.04999	0.04582	99	0.91661	33.366
6	0.14491	0.15038	88	1.03778	56.845
7	0.21878	0.23076	88	1.05476	59.789
8	0.13925	0.11994	99	0.86131	23.062
9	0.25595	0.73767	110	2.88209	100.000
10	0.00675	0.03912	88	5.79612	100.000
11	0.06349	0.51971	88	8.18567	100.000
12	0.22503	0.23625	88	1.04989	58.955
13	0.14799	0.26223	99	1.77195	99.749
14	0.14605	0.15554	88	1.06499	61.515
All Sites	0.12356	0.21316	1232	1.72516	100.000

[a]Proposed method site structural number variance
[b]AASHTO method site structural number variance

Site 8, which was closed to traffic during the test period. Because the visual condition of the sites remained constant throughout the test period, it was concluded that the reduction in structural number during the test period was negligible compared with the observed structural number variation due to temperature effects.

Once all variances were computed, the *F* test for the comparison of the variances of two populations was used to test the null hypothesis that the variance of the proposed method is greater than or equal to the variance of the AASHTO method versus the alternative hypothesis that the proposed method's variance is less than the AASHTO method's variance. The percent confidence that the null hypothesis should be rejected in favor of the alternative is shown in the sixth column of Table 2. The site standard deviations are compared graphically in Figure 8.

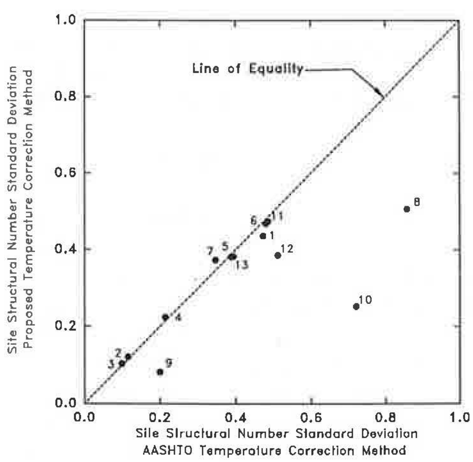


FIGURE 8 Comparison of site average structural number standard deviation using proposed correction method versus AASHTO correction method.

It may be inferred from the results in Table 2 that the proposed temperature correction method yielded a lower variance than the AASHTO correction method at 9 of the 13 sites. The confidence level that the proposed method is superior exceeded 99 percent at four of the sites. When all sites are considered together, the variance in structural number values computed using the proposed correction method is clearly lower than those computed using the AASHTO method. However, the proposed method led to greater variances at 4 of the 13 sites—Sites 2, 3, 4, and 7. Examination of the data shows that the variances at Sites 2, 3, and 4 are very low for both methods, so the increase in variance caused by the new method is extremely small.

The observed standard deviation of corrected SN values was found to increase with increasing SN for both correction methods. This relationship is shown in Figure 9 for both the AASHTO and proposed methods. The coefficient of variation (CV) of calculated SN, which expresses the standard deviation as a percentage of the mean, is shown in Figure 10. Although the data are relatively scattered, the trends shown indicate that the average CV values for both methods tend to remain

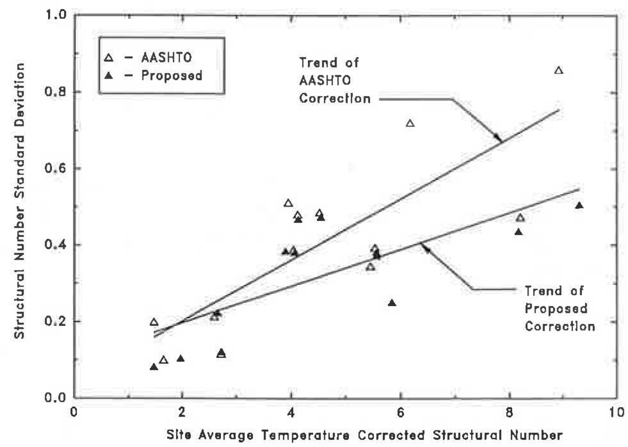


FIGURE 9 Site average structural number standard deviation versus temperature-corrected backcalculated site average structural number.

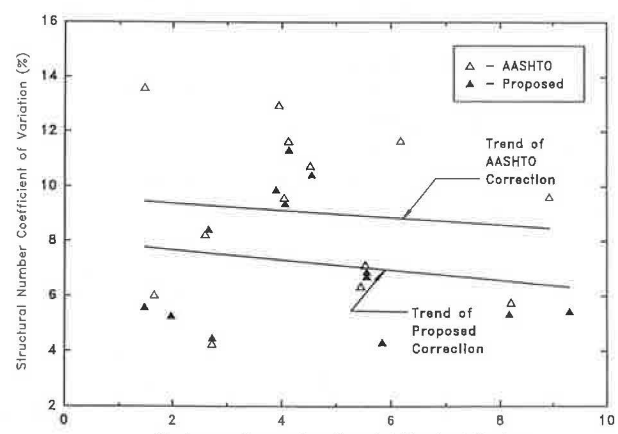


FIGURE 10 Site average structural number coefficient of variation versus temperature-corrected backcalculated site average structural number.

roughly constant with increasing SN. Additionally, it is clearly shown that the proposed alternative correction method yields a lower average CV value than the AASHTO correction method.

CONCLUSIONS

The proposed method to correct composite modulus and structural number values calculated using the Direct Structural Capacity Method for the effect of temperature has been shown to provide more consistent results than the method currently recommended by the AASHTO pavement design guide (*1*) for conditions observed in South Carolina. The proposed technique requires an estimate of the ratio of AC bound layer stiffness to non-AC bound layer stiffness. Although the AASHTO Direct Structural Capacity Method does not provide any information on individual pavement layers, it was shown that an estimate of the AC/non-AC layer modular ratio is sufficient for most pavement analysis applications.

The equation used for the estimation of AC concrete modulus at varying temperatures, Equation 8, is not recommended at estimated average pavement temperatures of 35°F and below. Although curtailing FWD operations during periods of low pavement temperature is not a problem in warm climates such as South Carolina, in many areas this may not be an acceptable limitation. Additional difficulties may ensue if freezing of water in the base layer alters overall pavement stiffness.

It is likely that the accuracy of either correction procedure would be increased if more accurate estimates of pavement temperature could be made. Although destructive methods of measuring pavement temperature allow more accurate pavement temperature estimates, such tests tend to offset the advantage of the FWD to test large sections of pavement rapidly. As a compromise between rapid testing and accuracy, it may be advisable to perform a single destructive temperature measurement at the beginning of a test section. This direct measurement of temperature could then be compared with the temperature calculated from Figure 1 to determine the estimation procedure's accuracy for that particular pavement.

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