Temperature Considerations in Asphalt-Aggregate Mixture Analysis And Design

JOHN A. DEACON, JOHN S. COPLANTZ, AKHTARHUSEIN A. TAYEBALI, AND CARL L. MONISMITH

The objective was to develop and demonstrate techniques for incorporating the effects of in situ temperature in the mixture-design process without adding significantly to the complexity of testing and analysis. The recommended approach converts the expected traffic, expressed in terms of equivalent single axle loads (ESALs) according to conventional AASHTO practice, into its equivalent value at critical pavement temperature. For both fatigue and permanent deformation, acceptable mixtures are those that resist equivalent laboratory loading in cyclic tests conducted at the critical temperature. Critical temperatures, the points at which most damage occurs in situ, were developed for nine climatic regions throughout the United States using the FHWA's Integrated Climatic Model for determining in situ temperatures. For 20-cm (8-in.) pavements, more than 40 percent of the fatigue damage and more than 64 percent of the permanent-deformation damage occurred within a 5°C (9°F) range centered on the critical temperature. Temperature equivalency factors, used to convert ESALs at one temperature to their equivalent at the critical temperature, were found to depend on mode of distress, the pavement structure, and the asphalt mixture—but to be independent of climate. When multiple temperature laboratory testing is required, for example, when reliability must be unusually high, suitable temperature ranges are 15°C to 30°C (59°F to 86°F) for fatigue testing and 30°C to 45°C (86°F to 113°F) for permanent-deformation testing.

Objectives of the recently completed Strategic Highway Research Program (SHRP) Project A-003A included developing a series of accelerated performance-related tests for asphalt-aggregate mixtures, and systems for analyzing the effects of mixture properties on pavement performance. The hierarchical structure of the analytical process requires a minimum amount of testing and analysis for most routine mixture designs but permits, at the same time, a much more comprehensive approach for specialized requirements, including the evaluation of new types of mixtures and mixture designs for major paving projects. Regardless of the level of detail, however, the analysis seeks to assure, within a level of reliability selected by and suitable to the designer, that the mixture will perform satisfactorily in service. Mixture properties, traffic, temperature, and the pavement structure are among the factors considered in all analyses, regardless of the degree of sophistication.

Distress mechanisms considered in the evaluation process include fatigue cracking, permanent deformation or rutting, and thermal cracking. For both fatigue cracking and permanent deformation, the destructive effects of highway traffic are expressed in terms of equivalent single axle loads (ESALs). Load equivalency factors have proven to be indispensable for expressing the relative destructive effects of a wide variety of over-the-road axle loadings and for determining the number of repetitions of a standard, 80-kN (18,000-lb), single-axle load that is equivalent to the traffic volume anticipated in service (*1*).

Extending the equivalency concept into the temperature domain offers considerable promise for conveniently and accurately treating the complexity of the in situ temperature environment during mixture evaluation. Factors are needed to convert the design ESAL to its equivalent at a single temperature. Use of a single temperature significantly reduces the testing and analysis effort in evaluating mixture performance. Even routine mixture designs can accurately reflect the thermal environment anticipated in situ. Thus the temperature equivalency approach can simplify testing and analysis and thereby increase productivity, reduce costs, and improve predictive accuracy.

A process that has been included in the SHRP A-003A mixture analysis and design system for treating temperature conditions is described, as are the various factors necessary to implement that process.

APPROACH

As originally conceived, the investigation concentrated on the development of a set of temperature-equivalency factors, patterned after the AASHTO load-equivalency procedure, that could be used to easily account for traffic level and environmental temperatures in mixture fatigue and permanent-deformation analyses. The temperature equivalency factor is a multiplicative factor used to convert the number of load applications at one temperature, i, to an equivalent number of load applications at a standard reference temperature, s. Thus,

$$\sum \text{TEF}_i \times \text{ESAL}_i = \text{equivalent ESAL}_s \tag{1}$$

where TEF_i is the temperature equivalency factor for the ith temperature interval, and ESAL_i is the design ESALs accumulating during the ith temperature interval.

The temperature equivalency concept theoretically requires independence of the effects of both multiple temperature levels and the order in which they occur. Because such independence has not yet been validated, the temperature equivalency factors developed here only represent first-order approximations.

As illustrated by the following equation, the computation of temperature equivalency factors requires simulations of pavement life at different temperatures:

J.A. Deacon, University of Kentucky, 202B Transportation Research Building, Kentucky Transportation Center, Lexington, Ky. 40506–0043. J. Coplantz, Bowman and Williams, P.O. Box 1621, Santa Cruz, Calif., 95061. A. Tayebali, Institute of Transportation Studies, University of California at Berkeley, Richmond Field Station, Building 480, Richmond Calif. 94804. C.L. Monismith, Department of Civil Engineering/ITS, University of California at Berkeley, Room 115 McLaughlin Hall, Berkeley, Calif. 94720.

$$\mathrm{TEF}_i = N_s/N_i \quad (2)$$

where N_s is the number of load repetitions to failure at the standard reference temperature, and N_i is the number of load repetitions to failure at the ith temperature. "Failure" is defined as initiation of cracking and a permanent surface deformation of 12.7 mm (0.5 in.) for fatigue and permanent deformation, respectively. Pavement life is modeled using multilayer elastic analysis (ELSYM) to estimate the stress and strain states within hypothetical pavement structures. For fatigue, the maximum principle tensile strain, (ϵ), at the bottom of the asphalt layer is converted to a pavement-life estimate using an $N-\epsilon$ relationship calibrated from laboratory controlled-strain fatigue tests. For permanent deformation, layered-strain procedures are used together with repeated-load triaxial compression test results to estimate the effect of load repetitions on permanent surface deformation. The simulated traffic load in each case was the standard AASHTO 80-kN (18,000-lb), single-axle load.

Temperature equivalency factors must be computed for a specific geographic location (that is, temperature environment) and a specific pavement structure. There are two primary tasks: the first is to estimate temperature profiles throughout the pavement structure; the second is to correlate pavement life (number of repetitions to failure) with the pavement temperature profile. FHWA's Integrated Climatic Model (*2*) was used to compute temperature profiles (depth increments of 5 cm, or 2 in.) for each of 4,380 hr in a typical year. Elimination of one-half the 8,760 hr in each year significantly reduced the computational effort without sacrificing accuracy. The temperature profile for each of the 4,380 hr was then characterized by two quantities, the temperature at the critical pavement location and a temperature gradient. For fatigue, the critical location was considered to be the bottom of the asphalt layer. For permanent deformation, the critical location, which is to be near the pavement surface, was selected somewhat arbitrarily to be a depth of 5 cm (2 in). In each case, the temperature gradient in °C per inch was defined as

$$(T_B - T_{2''})/D \quad (3)$$

in which T_B is the temperature at the bottom of the asphalt layer, $T_{2''}$ is the temperature at a 5-cm (2-in.) depth, and D is the thickness of the asphalt layer less 5 cm (2 in.).

The pavement-life computations sought to relate pavement life to temperature and temperature gradient. Approximately 10 temperature categories and 10 temperature-gradient categories were analyzed—approximately 100 combinations. For each of these combinations, the average temperature profile for those of the appropriate 4,380 hr was analyzed. All pavements were composed of two layers, an upper asphalt layer, and a uniform foundation that supported it. To properly account for temperature effects, the asphalt layer was further subdivided into four sublayers of varying temperatures and, hence, varying stiffnesses. For fatigue, the output variable of interest from the multilayer elastic simulations was the maximum principle tensile strain at the bottom of the asphalt layer. For permanent deformation, the deviator stress was computed at 1 in. deep increments throughout the asphalt layer. To ensure that the critical condition was examined, computations included locations at the center of the dual tire set, at the center of one tire of the dual set, and at an outer edge of one of the tires.

The estimation of pavement life in fatigue applied a laboratory-calibrated equation of the following type:

$$N_f = 10^{(K1+K2\cdot T)} \cdot \epsilon^{(K3+K4\cdot T)} \quad (4)$$

where

N_f = number of repetitions to initiate fatigue cracking under controlled-strain loading
ϵ = maximum principle tensile strain,
T = temperature, and
Ks = experimentally determined constants.

The estimation of pavement life in permanent deformation is slightly more involved. Using layered-strain procedures, permanent surface deformation is estimated as follows:

$$\sum \epsilon_p \cdot \text{(thickness)} \quad (5)$$

where

ϵ_p = vertical permanent strain in a layer increment,
thickness = thickness of the increment, and,
$\sum$ = summation over all the increments within the asphalt layer.

For all computations reported here, 1-in. thickness increments were used. The permanent strain was in turn calculated from a laboratory-calibrated expression of the following type:

$$\epsilon_p = C1 \cdot N_p^{\,C2} \cdot \sigma_d^{\,C3} \cdot T^{\,C4} \quad (6)$$

where

ϵ_p = permanent strain,
N_p = number of load repetitions,
σ_d = deviator stress,
T = temperature, and
Cs = experimentally determined constants.

Because failure is associated with a permanent surface deformation of 12.7 mm (0.5 in.), a trial-and-error process was necessary to determine the appropriate permanent-deformation life.

The objective of the computations is to develop generalized relationships between pavement life and both temperature and temperature gradient, so that a life corresponding to each of the 4,380 simulated hours could be estimated. For both fatigue and permanent deformation, the approximately 100 sets of calculations provided a suitable data base for calibrating regression equations in the following form:

$$Ln(N) = A_1 + A_2 \cdot T + A_3 \cdot G + A_4 \cdot T^2 + A_5 \cdot G^2 + A_6 \cdot T \cdot G \quad (7)$$

where

N = number of load repetitions to failure either for fatigue or permanent deformation,
T = temperature at the critical pavement location (bottom of asphalt layer for fatigue and at a 5-cm (2-in.) depth for permanent deformation),
G = temperature gradient as defined in Equation 3, and
As = regression estimates.

Finally, after determining appropriate temperature categories for which temperature equivalency factors were desired, pavement lives were estimated for each of the 4,380 hr using Equation 7. The 4,380 hr were grouped according to the preselected temperature categories, average pavement lives were computed for each category, and temperature equivalency factors were computed by entering these average lives into Equation 2.

This process was repeated for nine climatic regions spanning the continental United States, for two pavement structures in the fatigue investigation: a "thin" 10-cm (4-in.) structure and a thicker 20-cm (8-in.) structure and also for one 20-cm (8-in. pavement structure in the permanent-deformation investigation. In the latter analysis the 20-cm (8-in.) thickness was considered sufficient to confine most of the rutting to the asphalt layer. A dual-tire load of 40 kN (9,000 lb) with a contact pressure of 585.7 kPa (85 psi) and a 30.5-cm (12-in.) center-to-center tire spacing was used throughout the study.

PAVEMENT TEMPERATURES

The approach taken herein required the simulation of pavement temperature profiles through the 10-cm and 20-cm (4-in. and 8-in.) asphalt surfaces every other hour throughout a typical year. FHWA's Integrated Model of the Climatic Effects on Pavements (2) was well suited to this task.

The FHWA model's computer program can simulate pavement temperatures for any time period up to 1 year. Necessary data for nine climatic regions are built into the program's data base so that, for many purposes, climatological data need not be independently collected and inputted. Unfortunately, output for a specific run is limited to one particular hour of each day. This required 12 runs to analyze a specific site and pavement-structure combination and, at 15 to 20 min per run on 486-based computers with processor speeds of 25 to 33 MHz, the time to complete the temperature simulations for each situation was about 4 hr.

Pavement profiles were produced for both 10-cm and 20-cm (4-in. and 8-in.) surfaces in each of the nine climatic regions. Default characteristics for both material properties and climatic conditions were used throughout. Minimum daily air temperature was assumed to occur at 6 a.m. and maximum daily air temperature at 3 p.m. For the simulations, a time step of 0.125 hr was used, and the node spacing (vertically) was 5 cm (2 in.) in the top 50 cm (20 in.) and 15 cm (6 in.) in the next 320 cm (126 in). Constant deep ground temperatures for the nine regions are shown in Table 1.

The ASCII output files from the temperature simulations served as the input to a series of simple BASIC-language programs that were used for data manipulations and summaries. Tables 2–4, illustrative of the kinds of possible summaries, show the frequency distributions of temperatures at critical locations within the pavement as a function of climatic region.

TEMPERATURE EFFECTS ON PAVEMENT LIFE

For the computations to be tractable, the models must relate pavement life to both temperature and temperature gradient. To develop such models, a number of "standard" temperature profiles, defined by temperature level and by temperature gradient, were developed. For the 10-cm (4-in.) pavement, 110 profiles were identified consisting of all possible combinations of 11 levels of temperature—ranging from −5°C to 45°C (23°F to 95°F) in 5°C (9°F) increments—and 10 levels of gradient—ranging from −1.8°C to 0.9°C/in. in 0.3°C/in. increments. For the 20-cm (8-in.) pavement, 72 profiles were identified consisting of all possible combinations of nine levels of temperature—ranging from −5°C to 35°C (23°F to 95°F) in 5°C (9°F) increments—and eight levels of gradient—ranging from −1.5°C to 0.6°C/in. in 0.3°C/in. increments. For each category of temperature and temperature gradient, the temperature profile to be analyzed was determined by averaging over the applicable 4,380 hr of data and the nine climatic regions. Multilayered elastic analysis was used to estimate the stress and strain conditions within the pavement structure under the 40-kN (9,000-lb) dual-tire load.

Fatigue

A series of laboratory, controlled-strain, flexural-fatigue tests was performed to provide data to support the development of temperature equivalency factors for fatigue. Testing was limited to a single mixture, selected to be representative of a normal paving mixture in the United States. The asphalt cement had a penetration index of approximately 1.0. The aggregate was a dense-graded, partially crushed Greywacke with a 1-in. maximum size. The asphalt content was 5.2 percent by weight of aggregate, and the air-voids content was targeted at 4 ± 1 percent. A total of 23 specimens was tested at four temperature levels ranging from 5°C to 25°C (41°F to 77°F). Test specimens, with dimensions of 6.4 cm by 5 cm by 38 cm (2.5

TABLE 1 Constant Deep Ground Temperatures

Region	Temperature (°C)
IA (Boston, MA, and Chicago, IL)	10.0
IB (Little Rock, AR, and Washington, D.C.)	15.6
IC (Atlanta, GA, and San Francisco, CA)	21.1
IIA (Fargo, ND, and Lincoln, NE)	10.0
IIB (Oklahoma City, OK)	18.3
IIC (Dallas, TX)	21.1
IIIA (Billings, MT, and Reno, NV)	7.2
IIIB (Las Vegas, NV, and San Angelo, TX)	18.3
IIIC (San Antonio, TX)	21.1

°F = (°C × 1.8) + 32

TABLE 2 Frequency Distribution of Temperatures at Bottom of 10-cm (4-in.) Pavement (Percent)

Mid-Range Temp. (°C)	Region								
	IA	IB	IC	IIA	IIB	IIC	IIIA	IIIB	IIIC
-12.5				1.19					
-10.0				3.56					
-7.5				5.18			0.04		
-5.0				6.55	0.02		1.30		
-2.5	7.62			7.24	0.55		4.54		
0.0	17.81	1.80		13.10	6.53		17.12		
2.5	2.88	4.50	0.36	2.81	3.45	0.68	5.73	0.84	
5.0	3.90	5.09	3.93	2.90	4.09	2.28	5.09	2.42	0.34
7.5	4.29	6.30	6.16	3.06	4.79	3.81	4.86	3.61	2.24
10.0	4.63	7.12	7.67	3.63	5.78	4.68	4.91	4.57	3.81
12.5	5.18	6.67	9.52	4.02	5.98	5.96	5.39	5.75	5.23
15.0	5.75	6.39	10.39	4.57	6.19	7.10	6.32	6.75	6.74
17.5	6.87	6.76	11.39	5.73	6.37	7.15	7.99	7.08	7.97
20.0	9.04	7.26	11.85	7.62	6.71	7.21	7.15	7.12	8.63
22.5	7.05	9.95	9.34	5.80	8.67	8.17	6.05	7.81	9.43
25.0	5.94	7.56	6.92	5.20	8.45	10.94	4.43	10.62	11.85
27.5	5.20	7.19	6.03	4.70	7.49	8.26	5.00	8.04	9.66
30.0	4.61	5.50	4.79	3.70	4.84	7.74	3.65	7.81	7.74
32.5	3.77	5.07	4.93	3.52	5.68	6.16	3.95	5.34	6.64
35.0	3.42	3.86	4.27	3.22	4.16	5.36	2.92	5.96	6.53
37.5	2.01	4.25	2.44	2.69	3.56	4.38	3.15	4.47	3.95
40.0		3.81			3.84	4.09	0.39	4.00	5.25
42.5		0.91			2.85	3.40		4.16	3.49
45.0						2.60		3.10	0.50
47.5								0.55	

°F = (°C × 1.8) + 32

in. by 2.0 in. by 15 in.), were sawed from slabs prepared by rolling-wheel compaction. Initial flexural stiffness was measured at the fiftieth load cycle, and fatigue life was defined to be the number of repetitions to a 50 percent reduction in flexural stiffness. A work by Tayebali et al. (*3*) provides a detailed description of the test program and its results.

Initial flexural stiffness, serving to characterize the modulus of elasticity in the multilayered elastic analysis, was found to be acutely sensitive to temperature but independent of strain level. The regression equation quantifying the effect of temperature on stiffness is as follows:

$$S_o(mPa) = 1.491 \cdot 10^4 \cdot e^{-0.09385\,T}$$

$$S_o(psi) = 2.1621 \cdot 10^6 \cdot e^{-0.09385\,T} \qquad (R^2 = 0.92) \qquad (8)$$

where

S_o = initial flexural stiffness after 50 load cycles at 10Hz,
e = base of the natural logarithms, and
T = temperature in °C.

As expected in controlled-strain testing, fatigue life decreased with increasing stiffness (decreasing temperature) and with in-

TABLE 3 Frequency Distribution of Temperatures at Bottom of 20-cm (8-in.) Pavement (Percent)

Mid-Range Temp. (°C)	Region								
	IA	IB	IC	IIA	IIB	IIC	IIIA	IIIB	IIIC
-10.0				3.01					
-7.5				8.08					
-5.0				6.76			0.11		
-2.5	2.08			5.02	0.07		1.32		
0.0	20.80	0.18		9.43	1.23		11.92		
2.5	3.38	2.94		2.90	3.33		7.40		
5.0	3.95	7.15	1.05	3.49	6.55	0.87	7.10	0.84	
7.5	4.63	8.40	6.39	4.15	7.76	3.65	6.76	3.54	0.41
10.0	5.41	6.98	10.98	4.41	6.78	6.87	6.44	6.37	3.56
12.5	5.80	6.58	10.00	4.86	6.30	7.19	6.46	7.40	6.96
15.0	6.19	6.32	9.52	5.23	6.07	6.99	6.78	6.92	8.58
17.5	6.83	6.48	9.68	5.68	6.12	6.96	7.15	6.87	8.20
20.0	7.65	7.05	11.39	6.60	6.60	7.24	8.86	6.87	8.38
22.5	10.80	8.04	15.50	9.38	7.53	7.69	10.41	7.42	8.81
25.0	9.43	10.52	10.80	8.04	9.13	8.84	7.42	8.29	10.30
27.5	7.15	11.07	9.11	6.60	12.08	12.03	6.32	10.43	14.29
30.0	5.64	8.70	5.57	5.36	8.33	11.55	5.18	12.62	12.44
32.5	0.27	6.53		0.98	7.33	9.36	0.36	8.97	8.97
35.0		3.04			4.54	6.74		8.06	7.74
37.5					0.23	4.02		5.25	1.35
40.0								0.14	

°F = (°C × 1.8) + 32

creased tensile strain. The slope of the strain-life relationship was found to be highly temperature sensitive. The regression equation quantifying the effects of temperature and strain on fatigue life is as follows:

$$N_f = 10^{(20.0341 - 0.2261\,T)} \cdot \epsilon^{(-5.9138 + 0.1056\,T)} \qquad (R^2 = 0.94) \qquad (9)$$

where

N_f = number of cycles to a 50 percent reduction in flexural stiffness (that is, the fatigue life),
ϵ = maximum principal tensile strain in units of 10^{-6} mm/mm (in./in.), and
T = temperature in °C.

The two hypothetical pavement structures that we analyzed are described in Table 5. The asphalt surface was treated as four layers. Its modulus of elasticity of each layer was determined using Equation 8, based on its midpoint temperature. Fatigue life was calculated using Equation 9.

The computations are used to develop models relating fatigue life to both temperature and temperature gradient in the asphalt layer. Resulting models, containing only statistically significant terms, are summarized.

For the 8-in. pavement and all temperatures ($R^2 = 0.999$)

$$Ln(N_f) = 22.7019 - 0.55674\,T + 1.0481\,G + 0.0088228\,T^2 - 0.024482\,T\,G \qquad (10)$$

where

N_f = cycles to failure in fatigue,
T = temperature (T_B) at bottom of asphalt-bound layer in °C, and
G = temperature gradient in °C/in. [($T_B - T_{2''}$)/depth increment].

TABLE 4 Frequency Distribution of Temperatures at 5-cm (2-in.) Depth in 20-cm (8-in.) Pavement (Percent)

Mid-Range Temp. (°C)	Region								
	IA	IB	IC	IIA	IIB	IIC	IIIA	IIIB	IIIC
-12.5				2.24					
-10.0				3.58					
-7.5				5.68	0.02		0.04		
-5.0	1.96			6.46	0.04		1.10		
-2.5	4.47			4.95	0.11		3.31		
0.0	12.44	1.07		7.78	2.56		10.18		
2.5	5.39	4.63	0.20	3.95	4.45	0.43	7.21	0.55	
5.0	5.20	5.39	3.26	3.86	5.02	2.15	6.39	2.21	0.25
7.5	5.11	6.21	5.94	4.11	5.66	3.49	6.12	3.47	1.78
10.0	5.23	6.94	7.88	4.25	6.37	5.04	6.23	4.73	3.68
12.5	5.34	6.42	9.11	4.45	6.21	5.80	6.23	5.50	5.23
15.0	5.73	6.37	9.93	4.91	6.03	6.78	6.57	6.58	6.46
17.5	6.46	6.42	10.27	5.52	6.05	6.89	7.90	6.85	7.74
20.0	8.47	6.87	12.83	7.17	6.51	7.03	7.99	6.89	8.29
22.5	8.61	8.10	9.47	7.19	7.40	7.51	6.05	7.21	8.74
25.0	6.39	9.93	7.81	5.59	9.68	9.29	6.19	8.74	11.14
27.5	5.71	7.15	6.28	5.00	7.42	10.75	4.54	10.71	11.14
30.0	3.33	6.53	5.02	3.54	6.23	7.51	4.00	7.24	9.02
32.5	4.54	4.54	4.95	3.90	5.48	6.89	3.20	6.76	7.62
35.0	3.63	5.09	5.20	3.20	3.77	5.27	3.56	5.84	4.45
37.5	1.96	3.10	1.83	2.65	4.50	4.61	3.17	3.93	5.91
40.0		4.92			3.17	4.18		4.98	3.31
42.5		0.94			3.31	4.32		3.31	4.66
45.0						2.03		3.93	0.57
47.5								0.57	

°F = (°C × 1.8) + 32

For the 10-cm (4-in.) pavement and temperatures at a depth of 10 cm (4 in.) less than 25°C (77°F) ($R^2 = 0.999$)

$$Ln(N_f) = 18.4110 - 0.39772\,T + 0.45151\,G + 0.0081376\,T^2 - 0.013477\,T\,G \quad (11)$$

For the 10-cm (4-in.) pavement and temperatures at a depth of 10 cm (4 in.) of 25°C (77°F) or more ($R^2 = 0.999$)

$$Ln(N_f) = 16.1578 - 0.21948\,T + 0.28405\,G + 0.0044294\,T^2 - 0.0064224\,T\,G \quad (12)$$

As indicated by the very large R^2s, all the calibrations were highly significant in the statistical sense. Figures 1 and 2 illustrate the relationship between regression estimates and fatigue-life estimates from the calculations described.

Permanent Deformation

The SHRP A-003A approach to calculating permanent deformation in pavement structures uses a finite-element analysis that incorporates nonlinear viscoelastic surface properties and requires a suite of laboratory tests, including constant-height simple shear, uniaxial

TABLE 5 Hypothetical Pavement Structures

Layer	Property	"Thin" Structure	"Thick" Structure
Asphalt Surface	Thickness	10 cm (4 in.)	20 cm (8 in.)
	Number of Layers	4	4
	Modulus of Elasticity	Varies with temperature and temperature gradient	Varies with temperature and temperature gradient
	Poisson's Ratio	0.35	0.35
Subgrade	Modulus of Elasticity	172.3 MPa (25,000 psi)	68.9 MPa(10,000 psi)
	Poisson's Ratio	0.40	0.40

strain, volumetric, and shear frequency sweeps. In lieu of this approach, which had neither been well tested nor sufficiently refined at the time of our investigation, a conventional layered-strain analysis was used to aid development of pavement life versus temperature and temperature gradient models.

Again, the necessary models were calibrated using a mixture that was considered typical of paving mixtures in the United States. Table 6 summarizes mixture properties that were selected as characteristic of typical mixtures. The dynamic modulus-temperature relationship was taken from work reported by Akhter and Witczak (*4*). After adjustment using the mixture properties of Table 6, this relationship is as follows:

$$E = 100{,}000 \cdot 10^{(1.691635 - 0.00815T - 0.0000618\,T^2)} \qquad (13)$$

where E is the dynamic modulus of the asphalt mixture in psi and T is the temperature in degrees Fahrenheit.

Work by Leahy (*5*) provided the basis for relating permanent strain to number of load repetitions, temperature, and deviator stress. For the mixture properties of Table 6, this relationship is as follows:

$$\log \epsilon_p = -12.3469 + 0.408 \log N_p + 6.865 \log T + 1.107 \log \sigma_d \qquad (14)$$

where

ϵ_p = permanent strain in in./in.,
N_p = number of load cycles,
T = temperature in °F, and
σ_d = deviator stress in psi (or 90 psi, whichever is greatest).

The basis for this relationship was repeated-load triaxial testing of 251 specimens, including two aggregate types and two asphalt types. Unfortunately, Leahy's testing was limited to temperatures of 35°C (95°F)) and below. Thus considerable extrapolation was required to reach some of the temperatures at which rutting occurs in hotter climates.

All analysis was limited to one pavement structure having an asphalt surface of sufficient thickness to minimize the likelihood of significant rutting within subsurface layers. The specific section that was evaluated is defined in Table 7. The asphalt surface was treated as four layers, and the modulus of elasticity of each layer was determined using Equation 13 on the basis of its midthickness temperature. Pavement life, the number of repetitions resulting in a permanent deformation at the surface of 1.3 cm (0.5 in.), was calculated using the layered-strain approach of Equation 5 combined with the permanent strain relationship of Equation 14. In this trial-and-error process, eight 2.5-cm (1-in.) layers were used to represent the 20-cm (8-in.) surface course and the midthickness deviator

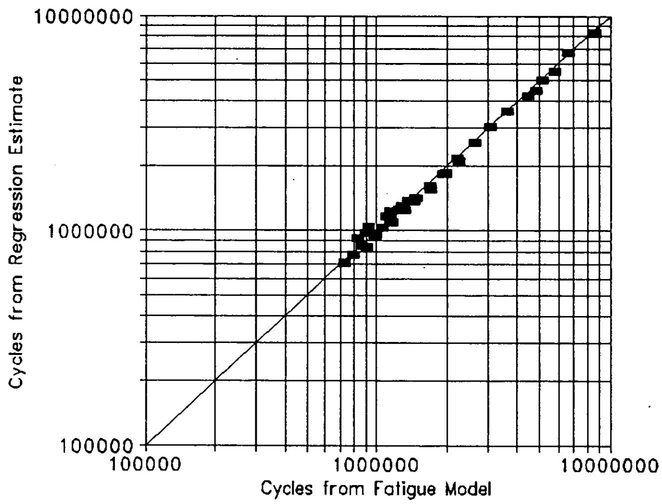

FIGURE 1 Accuracy of fatigue model for 20-cm (8-in) pavement.

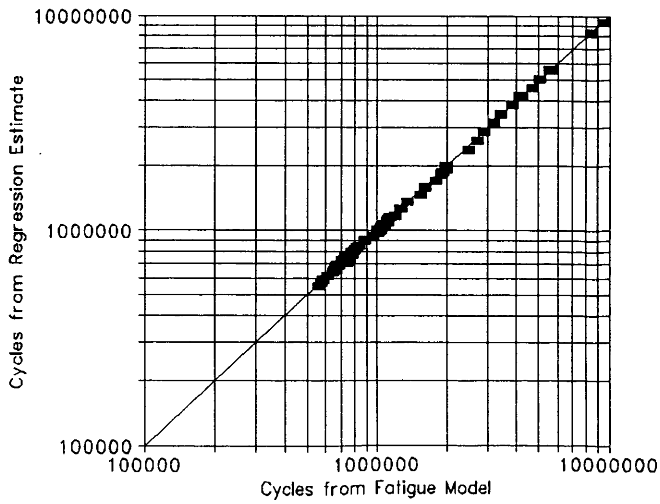

FIGURE 2 Accuracy of fatigue model for 10-cm (4-in.) pavement.

TABLE 6 Mixture Properties for Permanent-Deformation Investigation

Property	Value
f - frequency, Hz	10
Vis - viscosity @ 70°F, × 10^6 poise	1.5
Peff - percent effective asphalt by volume	12
Pair - percent air by volume	4
Pabs - percent of asphalt absorbed by weight of aggregate	0.5
PP200 - percent passing #200 sieve	5
PR4 - percent retained on #4 sieve	50
PR3/8 - percent retained on 3/8" sieve	30
PR3/4 - percent retained on 3/4" sieve	5

TABLE 7 Simulation Parameters

Layer	Property	Value
Asphalt Surface	Thickness	20 cm (8 in.)
	Number of Layers	4
	Modulus of Elasticity	Varies with temperature and temperature gradient
	Poisson's Ratio	0.35
Subgrade	Modulus of Elasticity	138 MPa (20,000 psi)
	Poisson's Ratio	0.40

stresses were calculated both directly and, as necessary, by interpolation.

The purpose of these computations was to develop models relating permanent-deformation life to both temperature and temperature gradient in the asphalt layer. The resulting models, containing only statistically significant terms, are summarized.

For temperatures at a depth of 5 cm (2 in.) less than 25°C (77°F) ($R^2 = 0.993$)

$$Ln(N_{0.5''}) = 28.0936 - 0.82261\,T - 5.0689\,G + 0.0099138\,T^2 + 0.10840\,T\,G \tag{15}$$

where

$N_{0.5''}$ = cycles to 0.5-in. permanent deformation,
T = temperature ($T_{2''}$) in °C at a 2-in. depth, and
G = temperature gradient in °C in. $[(T_{8''}-T_{2''})/6]$.

For temperatures at a depth of 2 inches of 25°C or more ($R^2 = 0.994$)

$$Ln(N_{0.5''}) = 26.6040 - 0.64020\,T - 4.2074\,G + 0.0046575\,T^2 + 0.069677\,T\,G \tag{16}$$

As indicated by the very large R^2s, all of the calibrations were highly significant in the statistical sense just as they were for the fatigue calibrations. Figure 3 illustrates the relationship between the regression estimate and the permanent-deformation–life estimates from the calculations described.

TEMPERATURE FACTORS

The primary attribute of the temperature equivalency approach is that both testing and the bulk of the routine analyses are limited to a single temperature. Mixture analysis and design are thus rather simple, given the proper choice of the testing temperature and the availability of applicable temperature equivalency factors. Atypical mixtures, however, particularly those with atypical temperature

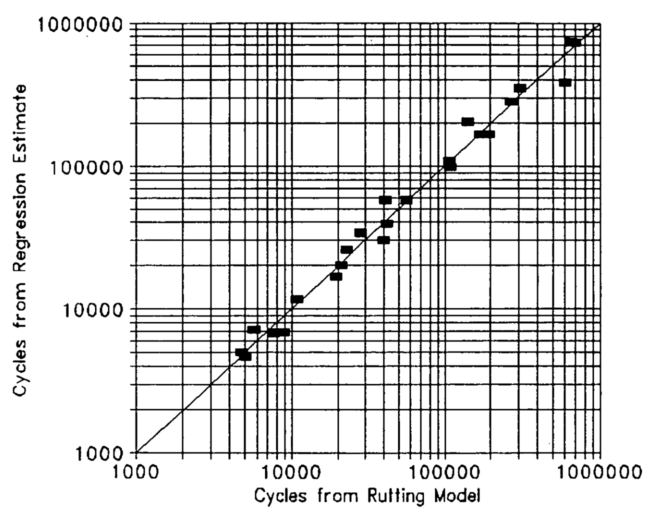

FIGURE 3 Accuracy of permanent-deformation model.

sensitivities, require a more complex routine and testing at multiple temperatures. Multiple-temperature testing also may be desired for large paving projects to increase the accuracy of evaluations.

Testing at a Single Temperature

Some of the factors to be considered by each mixture design agency in selecting the standard reference temperature for single-temperature testing include the following:

- Testing time will be minimized if the test temperature promotes early failure during laboratory testing,
- Accuracy will be maximized if the test temperature is near that to which in situ pavements are most vulnerable to damage,
- Efficiency will be maximized if a single test temperature can be used regardless of structural design and regardless of where the paving project is located within the design agency's jurisdiction, and
- Testing will be easier and less expensive if the test temperature affords comfortable working conditions and can be accurately controlled.

The ideal testing temperature would be one that could be used by any design agency without regard to distress mechanism, specific pavement structure, or project location. Available testing temperatures include, among others, effective temperature, critical temperature, and maximum temperature. Effective temperature is defined as that temperature at which loading damage accumulates at the same average rate in service as in the laboratory. Thus, when testing at the effective temperature, there is a one-to-one correspondence between laboratory and in-service loading cycles. The critical temperature is defined as that temperature at which the largest amount of damage occurs in service. For purposes of this investigation, the critical temperature was defined more specifically to be that temperature integer at the midpoint of a 5°C (9°F) temperature range, within which the largest amount of damage accumulates. Tables 8–10 summarize the effective, critical, and maximum temperatures for nine climatic regions within the continental United States. The tables indicate sizable differences between fatigue and permanent-deformation distress and much smaller but perhaps significant effects of climatic region and pavement structure (for fatigue only).

Critical temperature—or a standard temperature near critical temperature—is considered the optimal temperature for laboratory testing because it minimizes error associated with variations in mixture temperature sensitivity and because of its accelerated rate of damage accumulation. Remaining calculations focus on critical temperature but also include standard temperatures of 20°C (68°F) for fatigue and 35°C (95°F) for permanent deformation.

Temperature Equivalency Factors

Temperature equivalency factors, computed using Equation 2, are summarized in Tables 11–13. The reference temperature for fatigue is 20°C (68°F) (Tables 11 and 12) and, for permanent deformation, 35°C (95°F) (Table 13). Detailed examination of these tables reveals

- Temperature equivalency factors (TEFs) are a function of the type of distress. Regarding fatigue, maximum TEF occurs at an intermediate temperature level, whereas the TEFs increase monotonically with increases in temperature with respect to permanent deformation.
- Concerning fatigue, temperature equivalency factors are a function of the pavement structure. TEFs for thicker pavements are more sensitive to temperature than those for thinner pavements.
- Temperature equivalency factors are relatively independent of climate, especially regarding fatigue.

The apparently inconsequential effect of climate on temperature equivalency factors is of great interest, and potentially has considerable practical significance.

TABLE 8 Key Temperatures at 10-cm (4-in.) Depth for Fatigue Analysis, 10-cm (4-in.) Pavement

Region	Temperature in °C		
	Effective	Critical	Maximum
IA	15.2	22	37.7
IB	17.1	25	41.8
IC	17.9	21	37.6
IIA	14.1	27	38.6
IIB	17.0	26	43.7
IIC	18.9	28	45.6
IIIA	15.0	25	39.1
IIIB	18.6	27	46.5
IIIC	20.2	27	44.2
Mean	17.1	25.3	41.6

°F = (°C × 1.8) + 32

TABLE 9 Key Temperatures at 20-cm (8-in.) Depth for Fatigue Analysis, 20-cm (8-in.) Pavement

Region	Temperature in °C		
	Effective	Critical	Maximum
IA	19.6	25	31.3
IB	21.8	28	34.9
IC	20.6	26	30.9
IIA	19.2	27	31.9
IIB	22.0	28	36.3
IIC	23.2	30	38.1
IIIA	19.3	27	31.4
IIIB	23.3	30	38.8
IIIC	23.7	29	37.0
Mean	21.4	27.8	34.5

°F = (°C × 1.8) + 32

TABLE 10 Key Temperatures at 5-cm (2-in.) Depth for Permanent-Deformation Analysis

Region	Temperature in °C		
	Effective	Critical	Maximum
IA	27.7	35	37.6
IB	33.0	40	41.8
IC	29.3	35	37.5
IIA	28.3	36	38.4
IIB	34.2	42	43.7
IIC	36.0	43	45.7
IIIA	30.1	36	38.6
IIIB	37.2	44	46.6
IIIC	35.1	42	44.3
Mean	32.3	39.2	41.6

°F = (°C × 1.8) + 32

Temperature Conversion Factors

Equation 1, using temperature equivalency factors such as those in Tables 11–13, is used to convert the design ESALs to their equivalent at the standard reference temperature. Before the computation can be performed, however, the designer must estimate the number (or proportion) of the design ESALs that accumulate during each temperature interval. In the absence of detailed frequency distribution data, it appears reasonable to assume that damaging truck traffic is distributed uniformly over time. Thus, in the context of mixture evaluation, this suggests that a daytime hour is reasonably similar to a nighttime hour in terms of truck traffic, that a wintertime hour is reasonably similar to a summertime hour, and so forth.

The assumption that truck traffic is uniformly distributed through time permits the computation of a temperature conversion factor as follows:

$$TCF = \sum f_i \times TEF_i \tag{17}$$

where

TCF = temperature conversion factor,
f_i = frequency associated with the ith temperature interval, and
TEF_i = temperature equivalency factor for the ith temperature interval.

TABLE 11 Temperature Equivalency Factors in Fatigue for 10-cm (4-in.) Pavement; Referenced to 20°C (68°F) at 10-cm (4-in.) Depth

Mid-Range Temp. (°C)	Region								
	IA	IB	IC	IIA	IIB	IIC	IIIA	IIIB	IIIC
-12.5				2.4e-05					
-10.0				7.2e-05					
-7.5				2.9e-04			4.9e-04		
-5.0				1.1e-03	1.2e-03		1.3e-03		
-2.5	3.4e-03			3.5e-03	4.6e-03		3.7e-03		
0.0	0.011	0.014		0.011	0.012		0.012		
2.5	0.029	0.027	0.039	0.029	0.026	0.034	0.028	0.033	
5.0	0.068	0.063	0.067	0.067	0.063	0.063	0.065	0.061	0.077
7.5	0.140	0.137	0.141	0.139	0.133	0.136	0.133	0.134	0.135
10.0	0.258	0.259	0.261	0.258	0.255	0.250	0.248	0.245	0.251
12.5	0.425	0.429	0.437	0.430	0.424	0.423	0.413	0.423	0.413
15.0	0.636	0.636	0.649	0.635	0.636	0.638	0.618	0.636	0.629
17.5	0.848	0.846	0.860	0.848	0.844	0.845	0.816	0.844	0.846
20.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
22.5	1.071	1.056	1.106	1.066	1.051	1.060	1.062	1.057	1.058
25.0	1.246	1.296	1.378	1.221	1.283	1.324	1.376	1.330	1.270
27.5	1.731	1.708	1.814	1.712	1.697	1.668	1.731	1.660	1.683
30.0	1.566	1.544	1.580	1.593	1.541	1.560	1.570	1.546	1.550
32.5	1.398	1.383	1.424	1.385	1.376	1.328	1.411	1.326	1.347
35.0	1.106	1.120	1.138	1.124	1.084	1.116	1.103	1.119	1.101
37.5	0.927	0.869	0.952	0.882	0.862	0.854	0.894	0.832	0.852
40.0		0.627			0.624	0.613	0.723	0.612	0.621
42.5		0.495			0.429	0.439		0.419	0.413
45.0						0.298		0.271	0.329
47.5								0.213	

°F = (°C × 1.8) + 32

Computation of the equivalent ESALs at the standard reference temperature then requires only a single multiplication, as follows:

$$\text{equivalent ESAL}_s = TCF \times \text{design ESAL} \qquad (18)$$

where equivalent ESAL_s is the equivalent number of ESALs at the standard reference temperature, *s*, and design ESAL is the design traffic loading.

Temperature conversion factors for the pavements evaluated herein are summarized in Tables 14–16. For comparative purposes, reference temperatures include both the critical temperature for each condition as well as standard temperatures of 20°C (68°F) for fatigue and 35°C (95°F) for permanent deformation. The percentage of damage that occurs within a range of 5°C (9°F) centered on the reference temperature is illustrated in Tables 14–16. A large percentage is desirable because it minimizes potential error associated with atypical mixture temperature sensitivities.

Examination of Tables 14–16 reveals that temperature conversion factors are affected by mode of distress, pavement structure (at least for fatigue), and climatic region. A mixture effect, although also ex-

TABLE 12 Temperature Equivalency Factors in Fatigue for 20-cm (8-in.) Pavement; Referenced to 20°C (68°F) at 20-cm (8-in.) Depth

Mid-Range Temp. (°C)	Region								
	IA	IB	IC	IIA	IIB	IIC	IIIA	IIIB	IIIC
-10.0				8.7e-07					
-7.5				3.4e-06					
-5.0				1.7e-05			2.9e-05		
-2.5	1.3e-04			9.0e-05	1.5e-04		1.4e-04		
0.0	3.4e-04	5.6e-04		3.4e-04	4.4e-04		4.0e-04		
2.5	1.6e-03	2.3e-03		1.6e-03	1.9e-03		1.6e-03		
5.0	5.7e-03	5.8e-03	9.7e-03	5.5e-03	5.6e-03	9.3e-03	5.4e-03	9.2e-03	
7.5	0.018	0.017	0.019	0.017	0.017	0.018	0.017	0.018	0.030
10.0	0.049	0.049	0.050	0.048	0.047	0.050	0.046	0.050	0.052
12.5	0.123	0.124	0.123	0.122	0.122	0.125	0.119	0.123	0.128
15.0	0.275	0.278	0.277	0.273	0.277	0.279	0.266	0.277	0.278
17.5	0.557	0.555	0.556	0.554	0.552	0.561	0.549	0.561	0.557
20.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
22.5	1.628	1.616	1.576	1.648	1.605	1.614	1.528	1.604	1.595
25.0	2.169	2.330	2.224	2.193	2.321	2.301	2.232	2.298	2.293
27.5	2.885	2.843	2.764	2.823	2.914	2.966	2.886	2.951	2.921
30.0	3.220	3.221	3.234	3.319	3.284	3.258	3.288	3.288	3.185
32.5	3.359	3.356		3.232	3.292	3.271	3.623	3.334	3.344
35.0		3.137			3.033	3.028		2.964	2.988
37.5					2.718	2.510		2.404	2.628
40.0								2.059	

°F = (°C × 1.8) + 32

pected to be significant, was not among the variables investigated. Approximately 64 to 77 percent of the permanent-deformation damage occurs within a 5°C (9°F) interval centered on the critical temperature. The critical temperature is thus an effective temperature for permanent-deformation testing. For fatigue, approximately 26 to 46 percent of the damage occurs within a similar 5°C (9°F) interval. Choice of the test temperature thus seems to be somewhat less critical for fatigue than for permanent deformation.

Mode of distress, pavement structure, and climatic effects on the temperature conversion factor are similar for both the critical temperature and a standard reference temperature of 20°C (68°F) for fatigue, and 35°C (95°F) for permanent deformation. By definition, however, more damage is concentrated nearer the critical temperature than any other alternative. As a starting point for establishing testing norms on a nationwide basis, the average critical temperatures listed in Tables 8–10; that is, approximately 25°C (77°F) for fatigue and 40°C (104°F) for permanent deformation could be used. Other testing-temperature norms, reflecting more accurately the local climate, would likely be more appropriate than national norms for use by local and state design agencies.

Testing at Multiple Temperatures

Testing and analyzing mixtures at multiple temperatures are necessary when evaluating mixtures of atypical temperature sensitivity for which standard temperature equivalency and conversion factors are not applicable. It is also desirable for important paving projects, those designs demanding the utmost accuracy. The objective of such testing is to recalibrate Equations 8 and 9 for fatigue investi-

TABLE 13 Temperature Equivalency Factors in Permanent Deformation; Referenced to 35°C (95°F) at 5 cm (2-in.) Depth

Mid-Range Temp. (°C)	Region								
	IA	IB	IC	IIA	IIB	IIC	IIIA	IIIB	IIIC
-12.5				3.0e-11					
-10.0				1.2e-10					
-7.5				1.0e-10	1.3e-07		5.7e-09		
-5.0	4.8e-08			2.3e-10	4.4e-08		4.2e-08		
-2.5	6.5e-08			1.5e-09	5.0e-08		1.1e-07		
0.0	8.4e-08	6.3e-07		1.8e-08	3.1e-07		2.2e-07		
2.5	1.5e-07	2.6e-06	1.5e-05	1.3e-07	9.8e-07	1.3e-05	4.8e-07	1.5e-05	
5.0	3.1e-07	5.3e-06	3.3e-05	3.2e-07	2.1e-06	2.0e-05	1.2e-06	2.3e-05	9.4e-05
7.5	7.0e-07	9.9e-06	1.0e-04	7.3e-07	2.5e-06	6.5e-05	5.0e-06	7.2e-05	1.8e-04
10.0	1.5e-06	1.8e-05	1.2e-04	2.4e-06	1.6e-05	7.2e-05	2.1e-05	7.9e-05	3.5e-04
12.5	1.3e-05	7.4e-05	2.5e-04	7.5e-05	5.7e-05	1.7e-04	8.5e-05	1.9e-04	6.6e-04
15.0	4.4e-04	3.1e-04	5.6e-04	2.7e-04	2.6e-04	2.8e-04	3.3e-04	2.4e-04	1.2e-03
17.5	1.6e-03	1.1e-03	2.4e-03	1.1e-03	1.0e-03	1.1e-03	1.7e-03	7.9e-04	1.7e-03
20.0	7.0e-03	4.3e-03	9.1e-03	5.2e-03	3.9e-03	3.8e-03	4.9e-03	3.1e-03	4.8e-03
22.5	0.021	0.017	0.021	0.018	0.015	0.013	0.013	0.011	0.017
25.0	0.042	0.049	0.041	0.038	0.053	0.045	0.035	0.036	0.061
27.5	0.102	0.108	0.091	0.095	0.122	0.144	0.080	0.136	0.164
30.0	0.179	0.239	0.177	0.181	0.270	0.245	0.165	0.228	0.344
32.5	0.475	0.434	0.402	0.443	0.566	0.527	0.494	0.510	0.630
35.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
37.5	2.364	2.012	2.007	2.324	2.561	1.912	2.956	1.702	2.464
40.0		4.419			4.967	3.714		4.015	4.559
42.5		7.197			11.391	8.585		7.597	9.876
45.0						17.069		16.594	15.991
47.5								24.673	

°F = (°C × 1.8) + 32

gations and Equations 13 and 14 for permanent-deformation investigations. Following these recalibrations, temperature equivalency or conversion factors could be determined by procedures such as those we've employed here.

Testing temperatures should be carefully chosen so that they are compatible with the capabilities of laboratory test equipment and so that they span the range within which much of the damage occurs in situ. The calculations reported in Table 17 suggest that most damage occurs within a temperature range as small as 15°C (59°F). An even smaller range may eventually prove sufficient for permanent deformation. At the present, however, it appears that testing in the ranges of 15°C to 30°C (59°F to 86°F) for fatigue and 30°C to 45°C (86°F to 113°F) for permanent deformation is likely to be sufficient for most locations in the continental United States. In any case, lower-temperature testing is unproductive for both fatigue and permanent-deformation investigations, because very little fatigue and permanent-deformation damage occurs at such temperatures.

TABLE 14 Temperature Conversion Factors for Fatigue; 10-cm (4-in.) Pavement

Region	Reference Temperature: 20°C		Reference Temperature: Critical	
	Temperature Conversion Factor	Percent Damage Within 5°C Range	Temperature Conversion Factor	Percent Damage Within 5°C Range
IA	0.6587	25.1	0.6234	26.4
IB	0.8261	17.9	0.6332	28.9
IC	0.8748	24.9	0.8507	25.8
IIA	0.5720	24.2	0.3288	26.2
IIB	0.8083	17.0	0.4750	29.6
IIC	0.9407	15.6	0.5740	31.8
IIIA	0.6215	22.0	0.4528	25.7
IIIB	0.9258	15.4	0.5502	31.0
IIIC	1.0091	17.0	0.6065	33.0
Mean	0.8041	19.9	0.5661	28.7
Coefficient of Variation (%)	18.0	19.4	24.1	9.2

°F = (°C × 1.8) + 32

TABLE 15 Temperature Conversion Factors for Fatigue; 20-cm (8-in.) Pavement

Region	Reference Temperature: 20°C		Reference Temperature: Critical	
	Temperature Conversion Factor	Percent Damage Within 5°C Range	Temperature Conversion Factor	Percent Damage Within 5°C Range
IA	0.9147	18.5	0.4036	45.8
IB	1.4254	10.5	0.4788	43.6
IC	1.1457	23.8	0.4374	45.5
IIA	0.8531	16.7	0.2883	44.3
IIB	1.4817	9.5	0.4987	42.1
IIC	1.7928	8.3	0.5519	41.4
IIIA	0.8586	22.6	0.3180	42.5
IIIB	1.8268	7.9	0.5563	40.1
IIIC	1.9276	9.1	0.6135	42.6
Mean	1.3585	14.1	0.4607	43.1
Coefficient of Variation (%)	30.0	42.5	22.5	4.1

°F = (°C × 1.8) + 32

APPLICATION OF TEMPERATURE FACTORS IN MIXTURE ANALYSIS AND DESIGN

Temperature equivalency or conversion factors are applied in the SHRP A-003A mixture analysis and design process to convert in-service traffic loading (expressed in ESALs) to its equivalent at the preselected reference temperature. These equivalent traffic loading cycles are compared with mixture resistance as measured by laboratory repeated-load testing at the same reference temperature to determine the acceptability of specific mixtures. The SHRP A-003A system uses controlled-strain, flexural beam testing for fatigue. For basic-level, permanent-deformation analysis, a constant-height, simple-shear device operated in a repeated-load mode is used. The laboratory environment is calibrated to in situ

TABLE 16 Temperature Conversion Factors for Permanent Deformation; 20-cm (8-in.) Pavement

Region	Reference Temperature: 35°C		Reference Temperature: Critical	
	Temperature Conversion Factor	Percent Damage Within 5°C Range	Temperature Conversion Factor	Percent Damage Within 5°C Range
IA	0.1262	70.2	0.1262	70.2
IB	0.4800	20.6	0.0993	69.8
IC	0.1517	74.2	0.1517	74.2
IIA	0.1466	56.2	0.1006	76.6
IIB	0.7486	16.4	0.0707	67.8
IIC	1.4443	9.4	0.1224	64.2
IIIA	0.1817	47.4	0.1218	76.2
IIIB	1.1627	7.0	0.1337	68.5
IIIC	1.0486	13.4	0.1162	67.3
Mean	0.6100	35.0	0.1158	70.5
Coefficient of Variation (%)	78.6	72.8	18.9	5.7

°F = (°C × 1.8) + 32

conditions by applying an empirically determined shift factor, and a suitable multiplier is applied to satisfy reliability requirements. The procedure has been fully documented elsewhere (*6,7*).

SUMMARY AND CONCLUSIONS

The primary objective was to demonstrate the efficacy of the temperature equivalency concept for use in mixture analysis and design. For both fatigue and permanent-deformation investigations, techniques have been developed by which traffic loading in situ can be expressed in terms of its equivalent at a single reference temperature. This provides a simple effective way to accurately account for both traffic and environmental effects in mixture analysis and design. Specific conclusions include the following:

- Single-temperature testing is sufficient for routine mixture analysis of mixtures with typical temperature sensitivity. Setting the testing temperature to correspond with the critical temperature (appropriate to the geographical location and structural section) assures optimum results.

TABLE 17 Extent of Damage Accumulation in Suggested Temperature Ranges

Region	Percent Damage Within Indicated Temperature Range		
	Fatigue [15° through 30°C (59° through 86°F)]		Permanent Deformation [30° through 45°C (86° through 113°F)]
	10 cm (4-in.) Pavement	20-cm (8-in.) Pavement	
IA	71.6	92.4	89.6
IB	66.9	71.7	95.4
IC	71.5	95.1	89.3
IIA	70.3	88.6	91.3
IIB	65.8	67.3	97.4
IIC	67.8	61.2	92.3
IIIA	67.8	92.0	94.4
IIIB	66.3	56.5	78.7
IIIC	68.8	64.1	96.3
Mean	68.5	76.5	91.6

- Temperature equivalency factors are a function of the type of distress, pavement structure, and, presumably, mixture characteristics. However, equivalency factors appear to be relatively independent of climate especially for fatigue.
- Temperature conversion factors provide a simple, convenient way to convert traffic loading to its equivalent at a fixed temperature level. As a result, direct comparisons can be made between traffic loading in situ and single-temperature repeated-load testing in the laboratory.
- Particularly for permanent deformation, testing and analysis at the critical temperature are effective in reducing the influence of climatic variations throughout the continental United States.
- Test temperatures of 25°C (77°F) for fatigue and 40°C (104°F) for permanent deformation appear to be promising levels for establishment of national norms. Fatigue testing at 20°C (68°F) is an acceptable alternative if available testing equipment is unsuitable for testing at 25°C (77°F). However, other temperatures are likely to permit more accurate analyses by local and state design agencies.
- When multiple-temperature testing is necessary, the range of test temperatures can be reasonably small. On average, within nine climatic regions spanning the continental United States, temperatures in the range of 15°C to 30°C (59°F to 86°F) accommodate about 68 percent of the fatigue damage in 10-cm (4-in.) pavements and 76 percent in 20-cm (8-in.) pavements. Permanent-deformation testing from 30°C to 45°C (86°F to 113°F) encompasses the range within which an average of about 92 percent of the rutting occurs. These findings are expected to depend on mixture type, and they may be refined based on future work with a greater variety of mixture types.

The temperature equivalency and conversion factors reported here are considered to be first-order approximations that, if applied with care, provide an effective way to account for traffic and climatic effects in mixture analysis and design. They form an integral part of the system that has been developed by SHRP Project A-003A.

The next important requirement is to replace the layered-strain analysis with a more accurate model of permanent deformation, coupled with a range of appropriate laboratory test data to support its application. A more suitable permanent-deformation model also would allow investigation of the independence of the effects of multiple temperature levels, as well as the order in which they occur, and spur refinement if needed. Another objective would be to incorporate the effects of climate on subgrade support and to identify the variety of mixtures for which standard temperature equivalency and conversion factors are applicable. Finally, an investigation of the temporal (hourly, daily, and seasonally) variation in traffic loading is necessary to validate the assumption that damaging traffic loads are uniformly distributed through time.

ACKNOWLEDGMENTS

Research reported here was conducted as part of Project A-003A of the Strategic Highway Research Program, "Performance Related Testing and Measuring of Asphalt-Aggregate Interactions and Mixtures." The project was conducted by the Institute of Transportation Studies of the University of California, Berkeley.

REFERENCES

1. *AASHTO Guide for Design of Pavement Structures*—1986. AASHTO, Washington, D.C., 1986.
2. Lytton, R. L., D. E. Pufahl, C. H. Michalak, H. S. Liang, and B. J. Dempsey. *An Integrated Model of the Climatic Effects on Pavements.* FHWA-RD-90-033. Texas Transportation Institute, College Station, Tex., 1990.
3. Tayebali, A. A., J. A. Deacon, J. S. Coplantz, F. N. Finn, and C. L. Monismith. *Fatigue Response of Asphalt-Aggregate Mixtures: Part II—Extended Test Program.* SHRP Project A-003A, Institute of Transportation Studies, University of California, Berkeley, 1993.
4. Akhter, G. F., and M. W. Witczak. Sensitivity of Flexible Pavement Performance to Bituminous Mix Properties. In *Transportation Research Record 1034,* TRB, National Research Council, Washington, D.C., 1985, pp. 70–79.
5. Leahy, R. B. *Permanent Deformation Characteristics of Asphalt Concrete.* Ph.D. dissertation, University of Maryland, College Park, 1989.
6. Deacon, J. A., A. A. Tayebali, J. S. Coplantz, F. N. Finn, and C. L. Monismith. *Fatigue Response of Asphalt-Aggregate Mixtures: Part III—Mixture Design and Analysis.* SHRP Project A-003A, Institute of Transportation Studies, University of California, Berkeley, 1993.
7. Sousa, J. B., et al. *Permanent Deformation Response of Asphalt-Aggregate Mixtures: Part III—Mixture Design and Analysis.* SHRP Project A-003A, Institute of Transportation Studies, University of California, Berkeley, 1993.

This paper represents the views of the authors only and does not necessarily reflect the views of the National Research Council, SHRP, or SHRP's sponsor. The results reported here are not necessarily in agreement with the results of other SHRP research activities. They are reported to stimulate review and discussion within the research community.

Publication of this paper sponsored by Committee on Characteristics of Bituminous Paving Mixtures To Meet Structural Requirements.