Investigation of Seasonal Load Restrictions in Washington State

JOE P. MAHONEY, JO A. LARY, JAY SHARMA, and NEWTON JACKSON

ABSTRACT

Presented in the paper are the results of monitoring seasonal changes in surface deflections, temperatures, moisture contents, and calculated layer moduli for six test sites in the state of Washington. The goal of the data collection and analysis was to evaluate the Washington State Department of Transportation (WSDOT) load restriction tables used on some state routes during the spring thaw. Further, a criterion was developed to estimate when seasonal load restrictions need to be applied to those pavement sections that require them. Extensive use was made in the study of the WSDOT falling weight deflectometer to obtain pavement surface deflection basins and of multilayered elastic computer programs to analyze the data.

Pavement engineers throughout much of the United States are faced with the recurring problem of weakened pavement structures during the spring thaw. To reduce the pavement deterioration that can occur during this period, load restrictions for truck traffic are often applied. A survey reported in NCHRP Synthesis Report 26 (1,p.77) revealed that slightly less than one-half of the states use seasonal load restrictions. When such restrictions are used, several questions arise and can include the following:

1. Which pavement sections require load restrictions?
2. When should restrictions be applied and removed?
3. For Washington State, are the present load restrictions (developed in 1952) adequate, and how do they affect groups such as freight and timber hauling companies and school buses?

The current load restriction tables used in Washington State are summarized in Table 1 and can be applied at two levels:

1. Emergency load restrictions and
2. Severe load restrictions.

TABLE 1 Current WSDOT Load Restriction Tables[a]

Conventional Tires			Tubeless Tires		
	Allowable Gross Load (lb)			Allowable Gross Load (lb)	
Tire Width (in.)	Emergency	Severe Emergency	Tire Width (in.)	Emergency	Severe Emergency
7.00	1,800	1,800	8-22.5	2,250	1,800
7.50	2,250	1,800	9-22.5	2,800	1,900
8.25	2,800	1,900	10-22.5	3,400	2,250
9.00	3,400	2,250	11-22.5	4,000	2,750
10.00	4,000	2,750	11-24.5	4,000	2,750
11.00[b]	4,500	3,000	12-22.5[b]	4,500	3,000
12.00[c]	4,500		12-24.5[c]	4,500	

[a]Last revised October 1957.
[b]Or more–severe emergency condition.
[c]Or more–emergency condition.

To date, the Washington State Department of Transportation (WSDOT) has applied such load restrictions primarily on the basis of experience and occasionally on the basis of either Benkelman beam or falling weight deflectometer (FWD) pavement surface deflections. Most load restrictions are applied to low-traffic-volume routes such as the Federal Aid secondary system. Further, most counties use identical load restrictions and application periods.

The overall objective of the reported research was to evaluate the effect of freeze-thaw in pavement layers on pavement structural capacity. More specifically the objectives were to

1. Measure the variation of base and subgrade moisture content, frost depth and location, and pavement deflection (surface and in situ);
2. Develop procedures for using easily obtained data or otherwise provide for predicting when load restrictions should be applied on a given pavement structure; and
3. Determine an appropriate load restriction criterion.

To accomplish these objectives it was necessary to

1. Collect data at several test sites, including measurement of
   - Frost depth using frost tubes,
   - Moisture contents using soil cells,
   - Soil temperature using soil cells,
   - Dynamic deflection basins using the FWD,
   - Static deflections using a Benkelman beam, and
   - Dynamic and static deflections using an extensometer permanently buried in the pavement structure.
2. Collect weather data. These data, obtained from National Oceanic and Atmospheric Administration climatic reports or the WSDOT maintenance offices, were used to calculate freezing indices and to estimate depth of freeze using the modified Berggren equation.
3. Obtain pavement samples. Samples of the base and subgrade materials and cores of the asphalt concrete were obtained for laboratory resilient modulus determination. At the time of sampling, the in situ

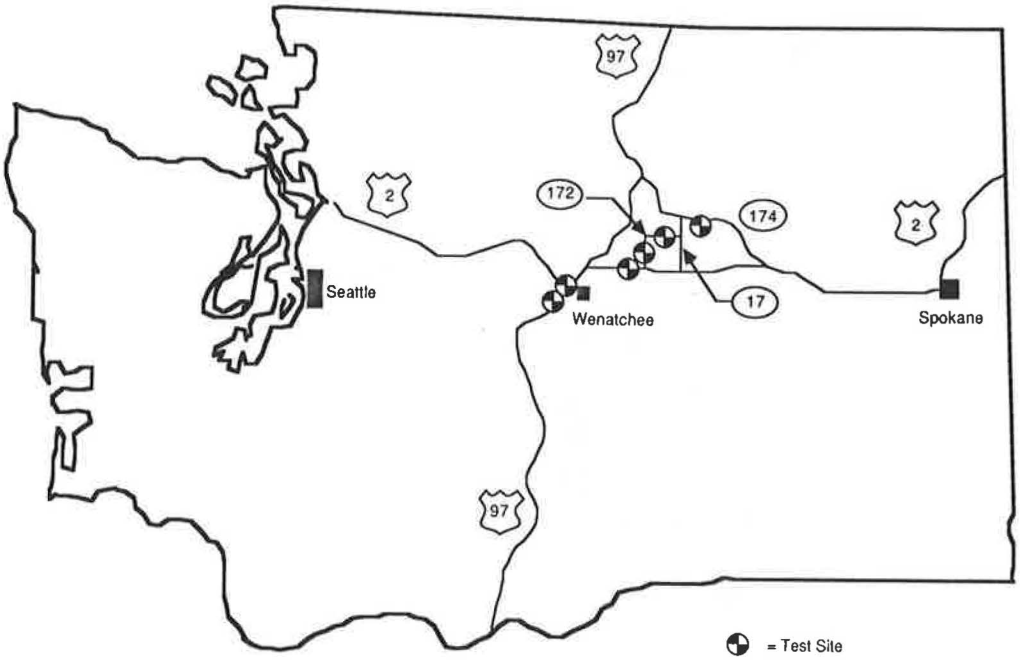

FIGURE 1 Location of field test sites.

density and moisture content of the base and subgrade were determined.

FIELD STUDY

Site Selection

Test sites on existing WSDOT routes were chosen in the central part of Washington (WSDOT District 2). In this part of the state the design freezing index is about 1,000 degree-days (Fahrenheit). Several criteria were used as a basis for selecting the test sites and include

1. The pavement must be located in an area of potential deep frost and be plowed in the winter to keep it free of snow;
2. Each test site should have a reasonable amount of heavy truck traffic (weight but not necessarily volume); and
3. The site locations should encompass a variety of subgrade soil and drainage conditions.

Using these criteria as a partial basis, six 500-ft-long test sites were selected (locations shown in Figure 1) for deflection testing, and four of the sites were instrumented. An overview of the significant test site features is given in Table 2.

Data Collected

Field data were collected at the six test sites during a 15-month period beginning in January 1982, with special emphasis on the spring thaw period. The following data were collected:

1. Pavement surface deflection using the FWD or Benkelman beam, or both;
2. Extensometer readings;

TABLE 2 Principal Test Site Features

State Route and Mile Post	Instrumentation	Pavement Structure[a]	Subgrade Class	Subgrade Percentage Passing 200 Sieve	Traffic, 1982 *(2)* ADT	Traffic, 1982 *(2)* Percentage Trucks
SR 97, MP 184	2 frost tubes 1 moisture tube[b]	4-in. ACP 4-in. CSTC 6-in. ballast	A-1-a(0)	9	3,500	11
SR 2, MP 117	2 frost tubes 2 moisture tubes 1 extensometer	6-in. ACP 17-in. gravel base	A-1-b(0)	16 to 19	11,500	10
SR 2, MP 160	2 frost tubes 1 moisture tube	2-in. ACP 9-in. CSTC	A-1-a(0)	9 to 12	1,000	
SR 172, MP 2	FWD testing	2.6-in. BST 6-in. gravel base			180	6
SR 172, MP 21	FWD testing	2-in. BST 9-in. gravel base			530	6
SR 174, MP 2	2 frost tubes 1 moisture tube 1 extensometer	0.5-in. BST 2-in. ACP 10-in. gravel base	A-1-b(0)	18 to 22	820	16

[a]Nomenclature: ACP = asphalt concrete pavement, CSTC = crushed surfacing top course, and BST = bituminous surface treatment.
[b]Each moisture tube consisted of four moisture cells.

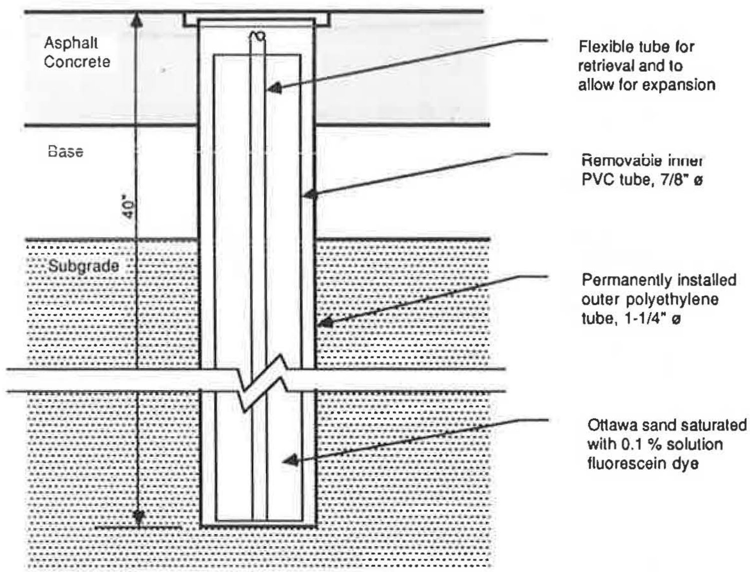

FIGURE 2 Schematic of in situ frost tube.

3. Pavement surface temperature;
4. Base and subgrade temperature;
5. Soil cell resistivity (for moisture content determination); and
6. Depth of frost penetration (using frost tubes).

Instrumentation

A primary objective of the study was to measure changes in pavement strength over the project duration. To this end, four of the six test sites were instrumented (as indicated in Table 2) with frost tubes to measure depth of freezing and soil cells (Soiltest MC 310A) to measure subgrade and base course moisture contents and temperatures. Extensometers were installed at two sites to measure pavement surface deflections. Paint marks were placed on the pavement surface to facilitate repeating deflection testing at the same locations. Sketches of typical frost tube and extensometer construction and soil cell layout are shown as Figures 2-4.

Deflection Measurements

With the advent of computer programs that provide for using pavement deflections to estimate in situ pavement layer moduli, the usefulness of devices that provide such measurements continues to increase. Three approaches were used in the study to measure pavement deflections:

1. Benkelman beam,
2. FWD, and
3. Extensometer.

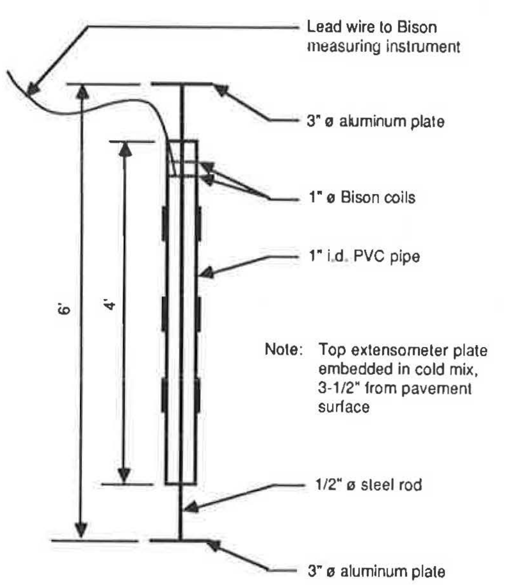

FIGURE 3 Schematic drawing of extensometer.

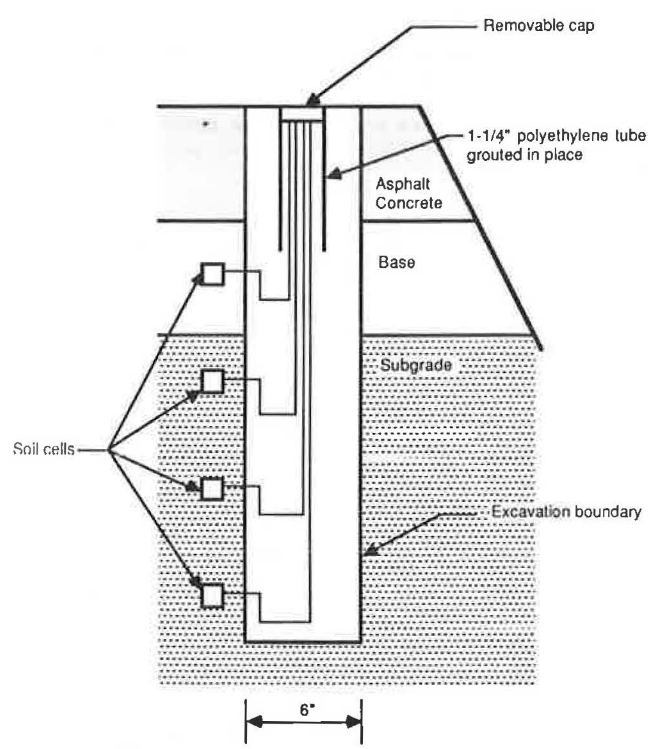

FIGURE 4 Typical soil cell layout.

The Benkelman beam was used to measure single-point rebound deflections along with a single axle, dual tire 18,000-lb axle load (quasi-static loading). The FWD was used to measure deflection basins (with seven sensors). The FWD can apply a dynamic load ranging from 3,000 to 24,000 lb and simulates a vehicle moving at speeds greater than 30 mph. Finally, the extensometer was used to measure deflections under various axle and tire loadings. The Benkelman beam and FWD measurements were taken at 50-ft intervals at each test site.

Typical Results

The type and range of the data collected from the test sites will be illustrated by use of two of the six test sites [SR 2, milepost (MP) 160 and SR 172, MP 2].

FWD Deflections

For the two selected test sites, the maximum pavement deflection (first sensor) averaged over each test site and normalized to a 9,000-lb load is plotted versus time in Figures 5 (SR 2, MP 160) and 6 (SR 172, MP 2). According to these data, spring was the period of highest deflection, as expected (similar trends were observed for the four other test sites). In general, maximum deflections were reached in late February or early March. During the January 1984 site visits, the measured deflections ranged from 9 to 20 percent of the previous summer values, which illustrates the increased stiffness of frozen pavement layers.

Figures 5 and 6 were developed on the basis of several site visits and corresponding data. These trends (as plotted) mask the actual variations that occurred at other times for which data are not available.

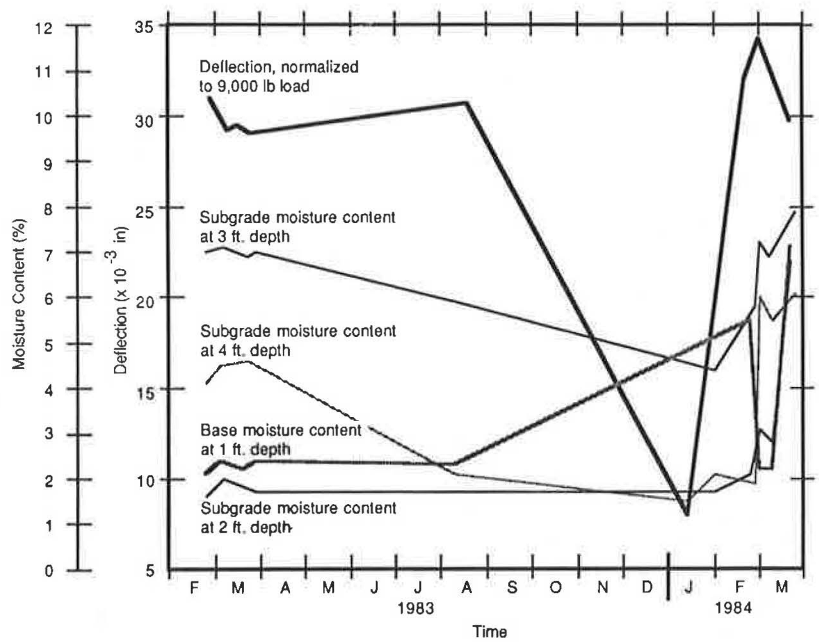

FIGURE 5 SR 2, MP 160—plot of FWD first sensor deflection and base and subgrade moisture contents versus time.

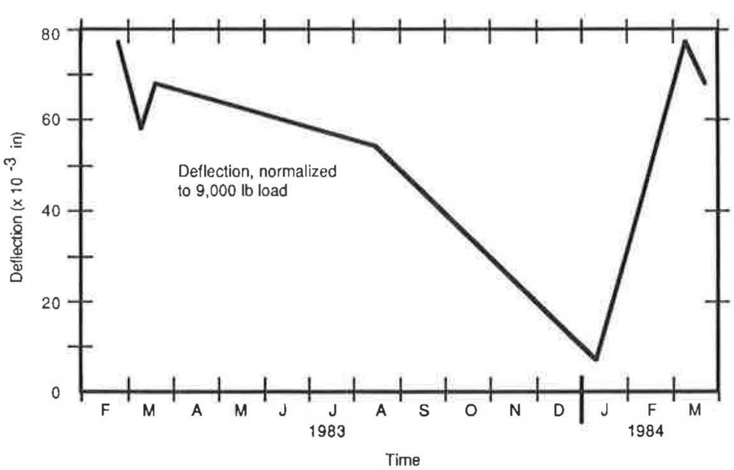

FIGURE 6 SR 172, MP 2—plot of FWD first sensor deflection versus time.

Base and Subgrade Moisture Contents

It is generally accepted that soils (and asphalt concrete) exhibit a decrease in strength with increasing moisture content. To this end, soil resistivity was measured and later converted to soil moisture content when the appropriate laboratory calibrations were completed. An example of the annual change in soil moisture content is shown in Figure 5. Moisture content tends to increase with depth (as expected) and is most variable during the thawing period (winter-spring 1984). Of significance is the high moisture content of the base course, which was essentially as high as the moisture content in the subgrade located 2 ft below the measurement point in the base. This will be further discussed later.

Temperature

As previously discussed, a variety of measurements was made to relate air and pavement temperatures. Soil cells were used to measure in situ soil temperatures as well as soil resistivity for moisture determination. Frost tubes were used in an attempt to measure locations of frost up to a depth of 4 ft; however, numerous problems arose with these and the resulting data were of little use.

On the basis of air temperature data from the nearest WSDOT maintenance facility, freezing indices were calculated and are given in Table 3. These data show the differences in just two winters (1982-1983 and 1983-1984). The design freezing indices (the average of the three coldest winters out of the last 30 years of record) range between 900 and 1,100 degree-days through the part of Washington State where the test sites are located (the mean freezing index for a 30-year record is about 500 degree-days). Thus the winter of 1982-1983 was slightly less severe than average and the winter of 1983-1984 was above average.

By use of the modified Berggren equation (3), various estimates of the depth of ground freezing were made. Assuming the pavement structure and the upper portion of the subgrade can be characterized as a homogeneous granular material (γ_d = 130 pcf, w = 5 percent) for thermal purposes, depths of freeze were calculated for the appropriate freezing indices at each test site and are given in Table 3. Also given in this table are the measured depths of frost (soil cells). The calculated and measured values are at least within the same range (recall that the soil cells were placed at 1-ft intervals of depth, making such comparisons quite approximate). Overall, the temperature data and associated calculations suggest that typical depths of freeze beneath the test sites are about 3 to 4 ft with maximum frost penetration occurring in late January through mid-February. As will be shown in the next section, relatively few thawing degree days are required to place these pavement structures in a "critical period."

DATA ANALYSIS

The method for determining pavement material properties and their variation with time for the six test sites is described in this section. Further, development of a temperature-based criterion for establishing when to apply load restrictions is presented along with the procedure for determining the magnitude of restrictions.

Material Properties

The BISDEF computer program (4) was used along with FWD data to estimate the moduli (or resilient moduli, M_R) and the associated stress sensitivity relations for the pavement layers. This program is based on layered elastic theory and was developed at the U.S. Army Corps of Engineers Waterways Experiment Station. It uses the concept of minimizing the difference between the program-calculated and the measured deflection basins. The program varies the layer modulus until a match is made between the input basin and the BISDEF-predicted basin within a specific margin of error.

The modulus for each pavement layer was estimated for each site visit and test site. The program was also used to calculate bulk stress at the middle of the base course and bulk or deviator stress at the top of the subgrade. Because a minimum of three stress levels (or FWD load magnitudes) was used during each site visit, it was possible to develop the stress-sensitivity relationships ($M_R - \theta$ or $M_R - \sigma_d$) for the base and subgrade layers. These relationships were necessary for additional modeling, which will be discussed later.

The required inputs for the BISDEF program were

1. Measured deflections (mils) and associated distances from the center of the load (inches).
2. Range of modulus values for each layer (psi),
3. Initial estimate of the modulus value for each layer (psi),
4. Thickness of each layer (inches),
5. Poisson's ratio for each layer,
6. Load stress (psi) and load radius (inches), and
7. Points in the pavement structure where stresses are desired.

TABLE 3 Freezing Indices and Calculated Depth of Frost Penetration for the Six Test Sites

Test Site	Winter	Freezing Index (degree-days Fahrenheit)	Calculated Depth of Frost (ft)	Minimum Measured Depth of Frost (ft)	Probable Date of Maximum Depth of Frost[a]
SR 97, MP 184	1982-1983	475	3.3		Feb. 9, 1983
	1983-1984	685	4.0	5	Jan. 23, 1984
SR 2, MP 117	1982-1983				
	1983-1984	510	3.4	3+	Jan. 23, 1984
SR 2, MP 160	1982-1983	400	3.1		Feb. 11, 1983
	1983-1984	730	4.1	3+	Jan. 23, 1984
SR 172, MP 2	1982-1983	475	3.3		Feb. 15, 1983
	1983-1984	745	4.2		Jan. 23, 1984
SR 172, MP 21	1982-1983	475	3.3		Feb. 15, 1983
	1983-1984	745	4.2		Jan. 23, 1984
SR 174, MP 2	1982-1983	170	2.0		Jan. 5, 1983
	1983-1984	470	3.3	2+	Jan. 24, 1984

[a]Based on recorded temperature data.

The measured deflections input to the program were the average deflection basins over each test section for each site visit. The input deflections were selected from four of the FWD sensors and were located at spacings of 0, 11.8, 25.6, and 47.2 in. from the center of loading. An initial estimate of each layer modulus was based on judgment and other previously completed work. The load radius was that of the FWD loading plate (5.9 in.) and the load stresses were the actual stresses applied to the pavement structure by the FWD.

The results of the BISDEF analysis are given in Tables 4 and 5 for the two test sites used in this paper to illustrate the study findings (SR 2, MP 160 in Table 4 and SR 172, MP 2 in Table 5). The stress relationships presented are of the form $M_R = k_1\theta^{k_2}$ for layers that behaved as coarse-grained materials and $M_R = k_1\sigma_d^{-k_2}$ for layers that behaved as fine-grained materials (M_R decreasing with increasing stress.) The SR 172, MP 2 test site was run as a two-layer system with the bituminous surface treatment and base course combined into one layer. This was done because the computer program would not close when the site was run as a three-layer system.

In general, for all test sites as well as the two sites shown, the base and subgrade moduli were higher for the August 1983 site visit than at other times of the year (as might be expected). Further, the pavement layer moduli were substantially higher when frozen. An interesting trend for SR 2, MP 160 was that the base modulus decreased 41 percent from August 1983 to March 1984, but the subgrade modulus on both dates was about the same. For SR 172, MP 2, the base modulus decreased about 27 percent and the subgrade modulus 44 percent for the same time interval (this test site had the largest decrease in subgrade modulus of the six). The maximum observed decrease in modulus for the base was 78 percent (August 1983 and March 1984 testing dates). For all test sites (except SR 2, MP 117, which exhibited extensive fatigue cracking and was actually weaker during the summer months), the base course modulus was reduced by an average of about 52 percent and the subgrade modulus by about 23 percent.

Magnitude of Load Restrictions

The PSAD2A computer program (5) was used to calculate deflections and strains under a given wheel load for the summer (strongest condition) and the spring (weakest condition) for each of the test sites. This was done to determine the change in strains and deflections between the two cases so that a spring load could be found that induced the same strains and deflections, and hence potential pavement damage, as occurred in the summer under maximum loading.

Several input values were required for the PSAD2A program and included for each layer

1. Poisson's ratio,
2. Dry density,
3. Thickness,
4. Stress-modulus relationship (from BISDEF analysis), and
5. Initial estimate of modulus for each layer.

Because the vast majority of trucks uses tubeless tires and the maximum wheel load is in part a func-

TABLE 4 SR 2, MP 160—Results of BISDEF Analysis for Determination of Resilient Moduli, Stresses, and Stress Relationships for each Site Visit

Date	Temperature (°F)	Applied Stress (psi)	AC M_R (psi)	Base M_R (psi)	Subgrade M_R (psi)	Base θ (psi)	Subgrade σ_d (psi)	Base Stress Relationship	Subgrade Stress Relationship
02/24/83	50	57.86	1,200,000	14,800	13,900	21.86	11.67	$1{,}266\theta^{0.814}$	$29{,}862\ \sigma_d^{-0.307}$
		88.17	1,500,000	20,800	13,100	28.42	15.85		
		123.01	1,200,000	29,400	11,700	49.18	20.29	$r^2 = 0.962$	$r^2 = 0.944$
03/03/83	45	56.72	1,200,000	21,600	13,100	18.94	10.51	$6{,}706\theta^{0.396}$	$17{,}462\ \sigma_d^{-0.118}$
		83.06	1,200,000	24,400	13,000	26.53	14.96		
		121.37	1,268,000	27,500	12,100	34.75	20.38	$r^2 = 0.997$	$r^2 = 0.791$
03/09/83	47	55.80	1,000,000	25,300	13,500	18.98	10.51	$9{,}508\theta^{0.327}$	$25{,}782\ \sigma_d^{-0.268}$
		82.44	1,222,000	26,400	13,000	25.71	14.62		
		116.70	1,300,000	30,000	11,500	31.02	18.74	$r^2 = 0.836$	$r^2 = 0.864$
03/13/83	60	42.70	1,100,000	15,200	10,000	13.90	7.27		
		83.96	1,029,000	26,800	12,700	26.90	15.10	$2{,}787\theta^{0.658}$	$21{,}904\ \sigma_d^{-0.199}$
		121.83	1,267,000	28,000	12,200	35.08	20.53	$r^2 = 0.953$	$r^2 = 0.926$
		152.87	1,318,000	30,500	11,500	40.01	24.33		
03/24/83	40	58.52	2,155,000	16,800	13,000	17.84	9.90		
		88.36	2,460,000	18,800	12,800	25.05	14.20	$4{,}245\theta^{0.474}$	$21{,}088\ \sigma_d^{-0.201}$
		125.46	1,600,000	26,300	12,200	34.44	20.41	$r^2 = 0.669$	$r^2 = 0.632$
		150.67	2,400,000	20,800	10,600	36.41	22.05		
08/17/83	72	80.08	931,000	28,800	11,100	23.08	13.50	$282\theta^{1.473}$	$6{,}152\ \sigma_d^{0.227}$
		125.98	1,000,000	43,300	12,000	30.44	19.04		
01/10/84	34	76.63							
		108.83							
		144.39							
02/21/84	42	71.0	1,096,000	20,800	12,900	24.54	13.50	$6{,}277\theta^{0.376}$	$21{,}492\ \sigma_d^{-0.197}$
		95.7	1,140,000	23,000	12,200	30.73	17.30	$r^2 = 0.988$	$r^2 = 0.994$
		129.2	1,258,000	24,600	11,700	38.35	22.16		
03/01/84	48	71.3	1,184,000	16,500	12,500	25.30	13.75	$4{,}628\theta^{0.398}$	$26{,}576\ \sigma_d^{-0.287}$
		95.9	1,327,000	18,800	11,800	31.05	17.31		
		125.8	1,462,000	19,300	11,000	37.60	21.46	$r^2 = 0.886$	$r^2 = 0.994$
03/09/84	60	66.6	972,000	24,700	12,800	22.26	12.39	$4{,}461\theta^{0.554}$	$26{,}189\ \sigma_d^{-0.282}$
		91.0	552,000	30,400	12,000	30.80	16.83		
		121.8	579,000	32,700	11,000	37.29	21.03	$r^2 = 0.984$	$r^2 = 0.968$
03/21/84	49	66.0	658,000	30,300	12,900	22.53	12.36	$18{,}504\theta^{0.151}$	$17{,}637\ \sigma_d^{-0.122}$
		93.3	819,000	29,600	12,600	30.00	16.86		
		127.1	700,000	33,000	12,000	39.20	22.19	$r^2 = 0.536$	$r^2 = 0.947$

TABLE 5 SR 172, MP 2—Results of BISDEF Analysis for Determination of Resilient Moduli, Stresses, and Stress Relationships for each Site Visit

Date	Temperature (°F)	Applied Stress (psi)	1st Layer M_R (psi)	Subgrade M_R (psi)	1st Layer θ (psi)	Subgrade σ_d (psi)	1st Layer Stress Relationship	Subgrade Stress Relationship
02/24/83	50	47.46	13,400	5,000	47.58	15.91	$613\theta^{0.781}$	$2{,}593\ \sigma_d^{0.239}$
		72.69	14,800	5,700	73.09	24.62		
		103.32	24,300	5,700	97.88	28.70	$r^2 = 0.788$	$r^2 = 0.937$
03/03/83	38	49.54	25,900	5,800	46.72	13.55	$12{,}272\theta^{0.195}$	$5{,}457\ \sigma_d^{0.024}$
		71.65	28,000	5,900	67.02	19.03		
		105.24	29,900	5,900	97.54	27.07	$r^2 = 0.996$	$r^2 = 0.741$
03/09/83	47	47.95	28,800	6,800	45.45	13.34	$12{,}978\theta^{0.209}$	$7{,}634\ \sigma_d^{-0.045}$
		68.40	31,000	6,700	64.20	18.40		
		101.05	33,500	6,600	93.67	26.01	$r^2 = 0.999$	
03/17/83	39	49.22	21,200	6,600	48.25	15.34	$14{,}995\theta^{0.096}$	$6{,}205\ \sigma_d^{0.021}$
		70.68	23,700	6,600	68.36	21.06		
		102.86	22,800	6,700	100.23	31.43	$r^2 = 0.386$	$r^2 = 0.806$
08/17/83	75	74.70	26,000	8,600	73.76	23.85	$6{,}393\theta^{0.326}$	$8{,}600\ \sigma_d^{0.0}$
		105.82	29,000	8,600	103.09	32.32		
01/10/84	34	72.84	371,100	59,900	65.82	17.08		
		104.21	348,300	59,300	94.82	25.06		
		143.49	326,800	50,500	128.97	32.96		
03/01/84	46	56.8	23,700	4,500	52.39	14.36	$952\theta^{0.817}$	$2{,}268\ \sigma_d^{0.258}$
		78.5	32,000	4,800	70.38	17.86		
		110.0	39,200	5,100	96.80	23.27	$r^2 = 0.982$	$r^2 = 0.994$
03/07/84	60	55.2	19,900	5,300	53.14	16.19	$2{,}355\theta^{0.520}$	$3{,}352\ \sigma_d^{0.163}$
		74.9	19,000	5,500	72.79	22.69		
		103.9	27,400	5,800	97.13	27.56	$r^2 = 0.619$	$r^2 = 0.935$
03/21/84	50	57.1	28,400	6,000	53.42	15.20	$13{,}761\theta^{0.173}$	$7{,}719\ \sigma_d^{-0.093}$
		77.6	27,000	5,800	72.80	20.82		
		107.0	31,600	5,700	98.14	26.49	$r^2 = 0.432$	$r^2 = 0.988$

tion of tire width, it was decided that the following tire sizes would be used in the subsequent analysis: 8-22.5, 9-22.5, 10-22.5, 11-22.5, 12-24.5, 14-17.5, and 16-22.5. Only single tires on single axles were evaluated because these were considered to be the most critical.

For the summer load cases, the maximum allowable load per time would be input. This maximum was determined by use of the Revised Code of Washington (RCW) 46.44.042, which allows 550 lb per inch width of tire up to a tire width of 12 in. and 660 lb per inch width for tires 12 in. wide or wider. For example, an 11-in.-wide tire can legally carry 6,050 lb and a 12-in.-wide tire 7,920 lb. Corresponding tire pressures were based on tire inflation pressures recommended by the Tire and Rim Association, Inc.

TABLE 6 Tire Loads and Tire Pressures for the Spring Condition

Percentage of Maximum Load	Tire Size	Tire Pressure (psi)	Load/Tire (lb)
100	8-22.5	105	4,400
	9-22.5	115	4,950
	10-22.5	105	5,500
	11-22.5	100	6,050
	12-24.5	115	7,920
	14-17.5	100	9,240
	16-22.5	90	10,000
75	8-22.5	80	3,300
	9-22.5	75	3,712
	10-22.5	70	4,125
	11-22.5	65	4,538
	12-24.5	80	5,940
	14-17.5	100	6,930
	16-22.5	75	7,500
50	8-22.5	55	2,200
	9-22.5	55	2,475
	10-22.5	55	2,750
	11-22.5	65	3,025
	12-24.5	65	3,960
	14-17.5	65	4,620
	16-22.5	55	5,000

These pressures were found to be reasonable for modeling purposes on the basis of a previous study performed for WSDOT (6). The tire loads and pressures for the summer condition (maximum condition) are given in Table 6 (100 percent of maximum load).

For the spring condition, the following cases were developed:

1. The maximum load and tire pressure as used for the summer condition,
2. Seventy-five percent of the maximum load and corresponding tire pressure as recommended by the Tire and Rim Association, and
3. Fifty percent of the maximum load and the recommended tire pressure.

The resulting tire loads and pressures are given in Table 6.

The output parameters from PSAD2A, which were evaluated for both the summer and spring analyses, were

1. Surface deflection (δ),
2. Horizontal strain at the bottom of the bituminous bound layer (ε_t),
3. Vertical strain at the top of the base course (ε_{VB}), and
4. Vertical strain at the top of the subgrade (ε_{VS})

When these deflections and strains had been calculated, the spring load that caused the same damage as the maximum allowable load during the summer was computed. This was done by use of plots developed from the previously listed program outputs for each test site and tire size and is shown for SR 2, MP 160 and tire size 11-22.5 in Figure 7. This figure was constructed as follows:

1. Surface deflection versus load was plotted for the three loads used in the spring analysis and a curve was fitted through the points and

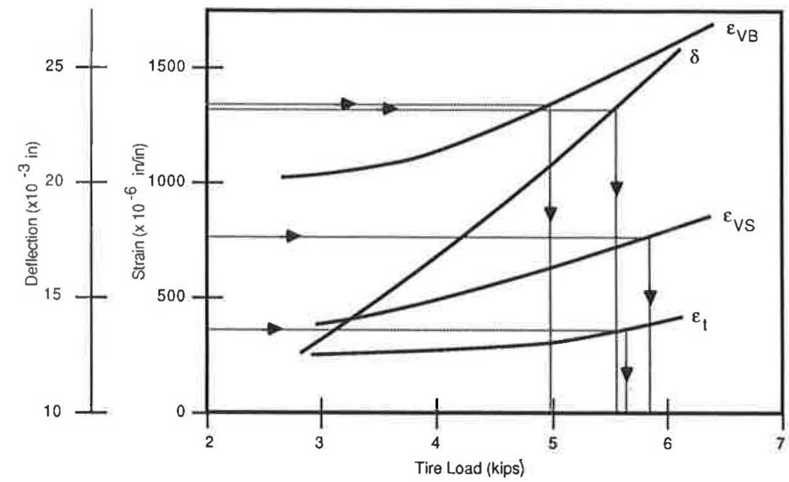

FIGURE 7 SR 2, MP 160–tire size 11-22.5.

2. ε_t, ε_{VB}, and ε_{VS} versus load were plotted for the same three loads and the corresponding curves were drawn.

The next step was to determine the spring load that would result in the same deflections and strains as did the summer case. This was done by entering the plot (such as Figure 7) on the vertical axis with δ, ε_t, ε_{VB}, or ε_{VS}. A horizontal line was then drawn to intersect the appropriate curve and then drawn vertically down to the tire load axis.

The allowable spring wheel loads so determined are given in Tables 7 and 8 for the two featured test sites. For SR 2 and tires up to 12 in. wide, the reduction in allowable load is no more than about 20 percent (from summer to spring conditions). This reduction increases for 14- and 16-in. tires. For SR 172, the reduction in allowable load for the critical criterion (surface deflection) is about 60 percent. A comparison of the percentage reduction between just these two pavement structures illustrates a basic difference between SR 2, which was originally designed to mitigate the effects of frost action, and SR 172, which was not.

Finally, Table 9 gives the allowable spring load and critical criterion for each tire size and test site. The low-volume routes such as SR 172 and SR 174 clearly have the largest reduction in allowable loads, as would be expected. On the basis of this type of analysis, actual load restrictions could be varied for each site; however, from a practical standpoint, this is not enforceable. If load restrictions are needed for a specific pavement structure, then only one or two levels of restrictions should be considered. From the analysis a spring allowable load of about 40 percent of the summer allowable appears reasonable (a 60 percent reduction). Interestingly, the corresponding allowable spring

TABLE 7 SR 2, MP 160–Spring Allowable Loads and Corresponding Percentage of the Maximum Legal Load

Tire Size	Maximum Legal Tire Load (lb)	Spring Allowable Load for δ (lb)	Percentage of Maximum Legal Load	Spring Allowable Load for ε_t (lb)	Percentage of Maximum Legal Load	Spring Allowable Load for ε_{VB} (lb)	Percentage of Maximum Legal Load	Spring Allowable Load for ε_{VS} (lb)	Percentage of Maximum Legal Load
8-22.5	4,400	4,020	91	4,080	93	3,670	83	4,400	100
9-22.5	4,950	4,600	93	4,600	93	4,190	85	4,920	99
10-22.5	5,500	5,050	92	5,020	91	4,600	84	5,390	98
11-22.5	6,050	5,570	92	5,830	96	4,990	82	5,900	98
12-24.5	7,920	7,170	90	7,120	90	6,180	78	7,600	96
14-17.5	9,240	8,115	88	6,640	72	6,020	65	8,790	95
16-22.5	10,000	8,900	89	7,820	78	6,760	68	9,560	96

Note: δ = surface deflection, ε_t = horizontal strain at the bottom of the asphalt concrete, ε_{VB} = vertical strain at the top of the base, and ε_{VS} = vertical strain at the top of the subgrade.

TABLE 8 SR 172, MP 2–Spring Allowable Loads and Corresponding Percentage of the Maximum Legal Load

Tire Size	Maximum Legal Tire Load (lb)	Spring Allowable load for δ (lb)	Percentage of Maximum Legal Load	Spring Allowable Load for ε_{VS} (lb)	Percentage of Maximum Legal Load
8-22.5	4,400	1,820	41	2,330	53
9-22.5	4,950	2,180	44	2,720	55
10-22.5	5,500	2,400	44	2,980	54
11-22.5	6,050	2,450	40	3,200	53
12-24.5	7,920	3,800	48	4,400	56
14-17.5	9,240	4,400	48	4,920	53
16-22.5	10,000	4,680	47	5,300	53

Note: δ = surface deflection and ε_{VS} = vertical strain at the top of the subgrade.

TABLE 9 Summary of the Critical Criteria and Corresponding Spring Allowable Load for Each Tire Size Modeled

Tire Size	Site	Critical Criterion for Each Site	Spring Allowable Load (lb)	Percentage of Maximum Legal Load
8-22.5	SR 97, MP 183.48	δ	3,775	86
	SR 2, MP 117.38	ϵ_t	5,200	118
	SR 2, MP 159.6	ϵ_{VB}	3,670	83
	SR 172, MP 2.0	δ	1,820	41(critical)
	SR 172, MP 21.4	ϵ_{VB}	2,400	54
	SR 174, MP 2.0	ϵ_{VB}	3,130	71
9-22.5	SR 97, MP 183.48	δ	4,325	87
	SR 2, MP 117.38	δ	5,460	110
	SR 2, MP 159.6	ϵ_{VB}	4,190	85
	SR 172, MP 2.0	δ	2,180	44 (critical)
	SR 172, MP 21.4	ϵ_{VB}	2,730	55
	SR 174, MP 2.0	ϵ_{VB}	3,490	70
10-22.5	SR 97, MP 183.48	δ	4,900	80
	SR 2, MP 117.38	δ	6,230	113
	SR 2, MP 159.6	ϵ_{VB}	4,600	84
	SR 172, MP 2.0	δ	2,400	44 (critical)
	SR 172, MP 21.4	ϵ_{VB}	2,750	50
	SR 174, MP 2.0	ϵ_{VB}	3,700	67
11-22.5	SR 97, MP 183.48	δ	4,875	80
	SR 2, MP 117.38	δ	6,770	112
	SR 2, MP 159.6	ϵ_{VB}	4,990	82
	SR 172, MP 2.0	δ	2,450	40
	SR 172, MP 21.4	ϵ_{VB}	2,290	38 (critical)
	SR 174, MP 2.0	ϵ_{VB}	3,850	64
12-24.5	SR 97, MP 183.48	δ	6,300	80
	SR 2, MP 117.38	δ	8,550	108
	SR 2, MP 159.6	ϵ_{VB}	6,180	78
	SR 172, MP 2.0	δ	3,800	48
	SR 172, MP 21.4	ϵ_{VB}	3,600	45 (critical)
	SR 174, MP 2.0	ϵ_{VB}	4,780	60
14-17.5	SR 97, MP 183.48	ϵ_t	6,020	65
	SR 2, MP 117.38	δ	9,380	102
	SR 2, MP 159.6	ϵ_{VB}	6,020	65
	SR 172, MP 2.0	δ	4,400	48
	SR 172, MP 21.4	ϵ_{VB}	3,460	37 (critical)
	SR 174, MP 2.0	ϵ_{VB}	4,670	50
16-22.5	SR 97, MP 183.48	ϵ_t	5,990	60
	SR 2, MP 117.38	δ	11,100	111
	SR 2, MP 159.6	ϵ_{VB}	6,760	68
	SR 172, MP 2.0	δ	4,680	47
	SR 172, MP 21.4	ϵ_{VB}	3,320	33 (critical)
	SR 174, MP 2.0	ϵ_{VB}	4,780	48

loads from this analysis fall within the range of the current WSDOT load restrictions (Table 1).

Criterion for When to Apply Load Restrictions

A basic issue addressed in the study was when to establish load restrictions on a specific highway (assuming that load restrictions are necessary). A criterion based on deflection measurements provides certainty as to the need for load restrictions. At least for the near future, it is impossible for WSDOT equipment and personnel to be at all the necessary locations during the potentially critical months of January, February, and March. An alternative approach is to use temperature data to estimate the depth of thaw in a pavement and thus whether it is near or in the critical period.

Figure 8 was prepared by calculating the depth of thaw for various thaw indices using the modified Berggren equation (3):

$$x = \lambda\ [(48\ k_{avg}\ n\ TI/L)^{1/2}]$$

where

x = depth of thaw (ft),
λ = dimensionless coefficient that corrects formula for neglected effects of volumetric heat,
k_{avg} = average thermal conductivity of the soil (Btu/hr•ft•°F),
n = factor for converting air thawing index to surface thawing index,
TI = air thawing index (degree-days, Fahrenheit), and
L = soil latent heat (Btu/ft^3).

The pavement structure was assumed to be homogeneous and composed of either a coarse-grained or a fine-grained soil (with corresponding dry densities of 130 and 100 pcf, respectively). An $n = 1.5$ was assumed (dark bituminous surface) along with $\lambda = 0.7$ for the fine-grained soil and $\lambda = 0.6$ for the coarse-grained soil. The pavement surface thickness was assumed to have a negligible effect on the depth of thaw (other than color). As shown in Figure 8 the depth of thaw for equal thawing indices is clearly greater for coarse-grained soils than for fine-grained soils. Further, it is reasonable to expect that the upper portions of all WSDOT pavement structures will behave as coarse-grained soil. Thus at an air TI $\approx$ 30 the depth of thaw will be about 12 in. and at an air TI $\approx$ 50 about 15 in. For most pavement structures this would result in the surface and base courses but not necessarily all of the subgrade being thawed.

The temperature data from the test sites and the BISDEF analysis of FWD data reinforce the modified Berggren calculations that the test sites reached their critical condition after receiving about 50 degree-days of thawing temperature. Thus it was

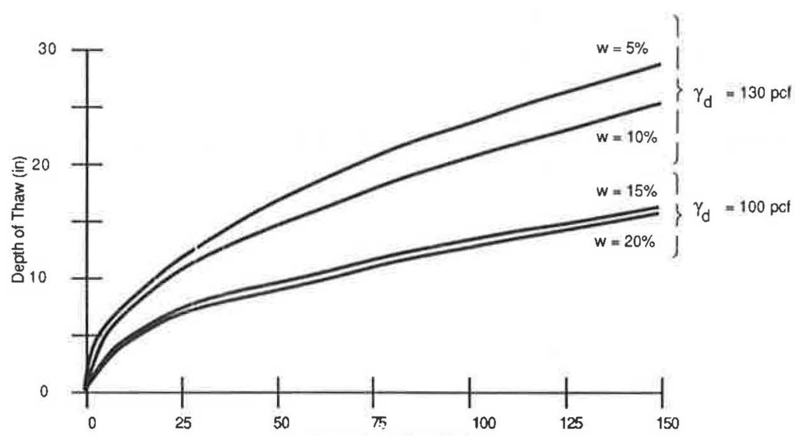

FIGURE 8 Air thawing index versus depth of thaw for thin asphalt surfaced pavements.

recommended that WSDOT tentatively adopt a TI ≈ 30 degree-days to indicate pavement structures approaching a critical condition and a TI ≈ 50 degree-days to indicate pavement structures in a critical condition. Clearly, pavement structure, subgrade soil, and winter temperature history will influence such criteria; however, WSDOT district maintenance personnel in the numerous maintenance offices record high and low daily temperatures for other purposes each winter. Now this same, available information can be used as a rule-of-thumb to assess the need for load restrictions.

CONCLUSIONS

The following conclusions are warranted:

1. The falling weight deflectometer is an excellent device for collecting the information required to evaluate the structural capacity of pavements. Further, Benkelman beam and FWD maximum deflections correlated well; however, the deflection basins obtainable with the FWD provide a significantly improved ability to analyze pavement structure.
2. For the field test sites that normally require seasonal load restrictions, the base course moduli vary more than the subgrade moduli. The subgrade moduli are relatively stable throughout the year (except when frozen). The base course weakness is due to excessive moisture available during the thawing period. The excessive moisture in the base course is exacerbated by either a still frozen subgrade or a low permeability subgrade soil (i.e., a water drainage path is temporarily reduced or eliminated), or both.
3. A multilayered elastic analysis computer program (BISDEF) was used along with FWD data to characterize the materials in the pavement layers for each test site with time. Criteria were developed that essentially reduce the allowable loads for a summer condition to equivalent loads during the critical period (spring thaw). On the basis of this analysis for the more critical test sites, a reduction in legal loads of about 60 percent is required. Further, the analysis tends to reinforce the current WSDOT load restriction tables.
4. A criterion was developed that can be used to determine when load restrictions should be initiated on a pavement structure requiring such limitations (the criterion does not identify which pavements require load restrictions). The criterion is based on thawing degree-days and can be readily used by the WSDOT maintenance offices that record daily high and low temperatures. Both field data and an analytical procedure suggest that pavements susceptible to weakening during the critical period will approach this condition after a thawing index of 30 degree-days has occurred and will be in the critical period after accumulating 50 degree-days (one thawing degree-day is equal to an average daily temperature of 1°F above freezing). Clearly, site-specific deflection data are the single best criterion to use in assessing the need for load restrictions, but deflection data can be expensive to obtain and difficult to get at the needed time. A temperature-based criterion is the next best alternative (and the least expensive).

REFERENCES

1. T.C. Johnson, R.L. Berg, K.L. Carey, and C.W. Kaplar. Roadway Design in Seasonal Frost Areas. NCHRP Synthesis of Highway Practice 26. TRB, National Research Council, Washington, D.C., 1974.
2. 1982 Annual Traffic Report. Washington State Department of Transportation, Olympia, 1982.
3. Calculation Methods for Determination of Depths of Freeze and Thaw in Soils--Emergency Construction. Technical Manual 5-892-6. Department of the Army, Sept. 1966.
4. A.J. Bush III. Nondestructive Testing for Light Aircraft Pavements. Report FAA-RD-80-9-II. U.S. Army Corps of Engineers, Nov. 1980.
5. R.G. Hicks, J.D. Swait, Jr., and E.O. Chastain. Use of Layered Theory in the Design and Evaluation of Pavement Systems. 3rd ed. Department of Civil Engineering, Oregon State University, Corvallis, Jan. 1978.
6. J. Sharma, J. Hallin, and J.P. Mahoney. Evaluation of Present Legislation and Regulations on Tire Sizes, Configurations, and Load Limits. Report WA-RD-59.1. Washington State Department of Transportation, Olympia, July 1983.

Publication of this paper sponsored by Committee on Flexible Pavements.