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NONDESTRUCTIVE TESTING OF IN-SERVICE HIGHWAY PAVEMENTS IN MAINE--ETC(U)  
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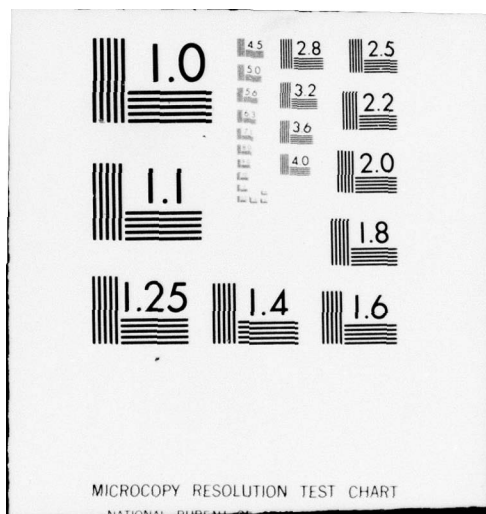
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## REPORT 79-6

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*Nondestructive testing of in-service  
highway pavements in Maine*

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*Cover: Nondestructive pavement test site in  
Maine. (Photograph by R.A. Eaton.)*

CRREL Report 79-6



*Nondestructive testing of in-service  
highway pavements in Maine*

N. Smith, R.A. Eaton and J.M. Stubstad

April 1979

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→ various materials were also compared. The residual surface deflections during testing for several pavement systems indicated a linear logarithmic relationship with number of load applications. A relationship between the modulus of the asphalt cement concrete pavement and pavement temperature was developed for the limited temperature range during the testing. ←

## PREFACE

This report was prepared by N. Smith and R.A. Eaton, Research Civil Engineers, of the Geotechnical Research Branch, and J.M. Stubstad, Mechanical Engineer, of the Applied Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory.

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**CONVERSION FACTORS: U.S. CUSTOMARY TO METRIC (SI)  
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<i>Multiply</i>	<i>By</i>	<i>To obtain</i>
inch	25.4*	millimeter
foot	0.3048*	meter
mile	1.6093	kilometer
kip (1000 lbf)	4448.222	newton
kip/inch	175126.8	newton/meter
pound-force/inch <sup>2</sup>	6894.757	pascal
kip/inch <sup>2</sup>	6894757	pascal
degrees Fahrenheit	$t_{°C} = (t_{°F} - 32)/1.8$	degrees Celsius

\*Exact

## NONDESTRUCTIVE TESTING OF IN-SERVICE HIGHWAY PAVEMENTS IN MAINE

N. Smith, R.A. Eaton and J.M. Stubstad

### INTRODUCTION

Evaluation of the performance of highway pavements in seasonal frost areas is a process of continuing importance to pavement design engineers. The depletion of high quality materials, environmental concerns and high costs require that these engineers constantly seek to improve the standard pavement cross sections presently used, to minimize the thickness requirements of the sections, and to incorporate marginal quality materials in them. The consequences of adopting revised pavement designs must always be evaluated. This requires that laboratory and field tests be conducted to determine the performance characteristics of the various new materials and pavement systems available for construction.

As part of its pavements research program, the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL), Hanover, New Hampshire, constructed a mobile repetitive plate bearing (RPB) test apparatus for conducting research investigations and developing better methods of nondestructive evaluation of pavement strengths. Then, in cooperation with the Department of Transportation in Maine, CRREL conducted 12 nondestructive field strength tests with test apparatus at locations shown in Figure 1. These tests were accomplished over a three-day period starting 13 April 1976.<sup>1</sup>

Each RPB test consisted of 500 repeated applications of an approximately 9-kip load (actual loading is determined by a load cell) on a 12-in.-diameter, 1-in.-thick steel plate at the rate of approximately 20 repetitions/minute. The resilient pavement surface deflections measured on top of the load plate at two points near its edge separated by approximately 90° and on the pavement surface at four radial distances from the center of the plate were monitored with linear variable differential transformers (LVDT's) during loading. The LVDT's output signals were recorded on a strip-chart recorder inside the tractor-trailer test vehicle.

These resilient pavement surface deflections were then analyzed with the Chevron computer program for a layered<sup>2,3</sup> elastic system to determine the resilient modulus values of the various pavement layers. The composite resilient stiffness of the total pavement cross section was calculated by dividing the load on the plate by the resilient pavement surface deflection measured on the load plate. In addition, the field data were used to generate a comparison of the stiffness properties of each pavement cross section.

This report presents descriptions of the repetitive plate bearing tests of road test sections at various locations in Maine; the test equipment and the procedures used; an analysis of test data obtained; and the conclusions reached at the end of the program.

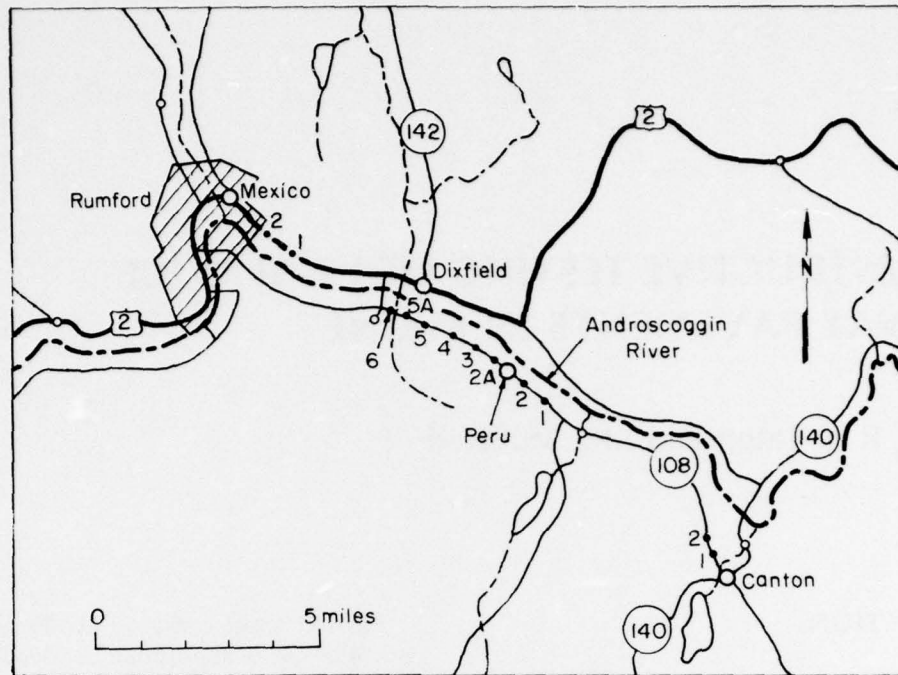


Figure 1. Test sites, Maine.

## DESCRIPTIONS OF ROAD TEST SECTIONS

The Department of Transportation of the State of Maine constructed various test sections in state highways in the area of the town of Peru, Maine (Fig. 1) to evaluate the performances of the sections under similar environmental and subgrade conditions.<sup>4,5</sup>

In 1962, the Department of Transportation constructed three test sections on Maine Route 108 in Peru. Cross-sectional views of the test sections are shown in Figure 2. The first section was 1.69 miles long and consisted of 12 in. of gravelly sand subbase, 6 in. of cement stabilized soil base, and 3 in. of asphalt concrete over a sandy subgrade (Fig. 2a). The second section was 1.72 miles long and consisted of 12 in. of gravelly sand subbase, 4 in. of asphalt stabilized aggregate base, and 3 in. of asphalt concrete over a sandy subgrade (Fig. 2b). The last section was 1.00 mile long and consisted of 18 in. of bank-run gravel subbase, 6 in. of crushed gravel base, and 3 in. of asphalt concrete over a sandy subgrade (Fig. 2c).

During the 1962-63 winter, the cement stabilized soil section developed large transverse cracks approximately every 30 ft; therefore, in the summer of 1963, a 1.75-in.-thick asphalt concrete overlay containing a 3-in. x 6-in. 10/10 gage wire mesh measuring 16 ft x 11 1/2 ft was placed over it (Fig. 2a).

In 1969, on Route 108, near Canton, approximately 3 miles east of the sections described above, two sections were constructed to evaluate the effect of polystyrene insulation within the pavement structure. An uninsulated control section (approximately 1 mile long) containing 16 in. of sandy gravel subbase, 4 in. of crushed gravel and 2 in. of asphalt stabilized aggregate base, and 3 in. of asphalt concrete over a free-draining fine sand subgrade (Fig. 2d) was constructed. Adjacent to this section, in a rock ledge cut area, a 1,000-ft-long insulated section was constructed with the following cross section: 6 in. of sandy gravel fill, 2 in. of Styrofoam board insulation, 20 in. of sandy gravel subbase, 4 in. of crushed gravel and 2 in. of asphalt stabilized aggregate base, and 3 in. of asphalt concrete (Fig. 2e).

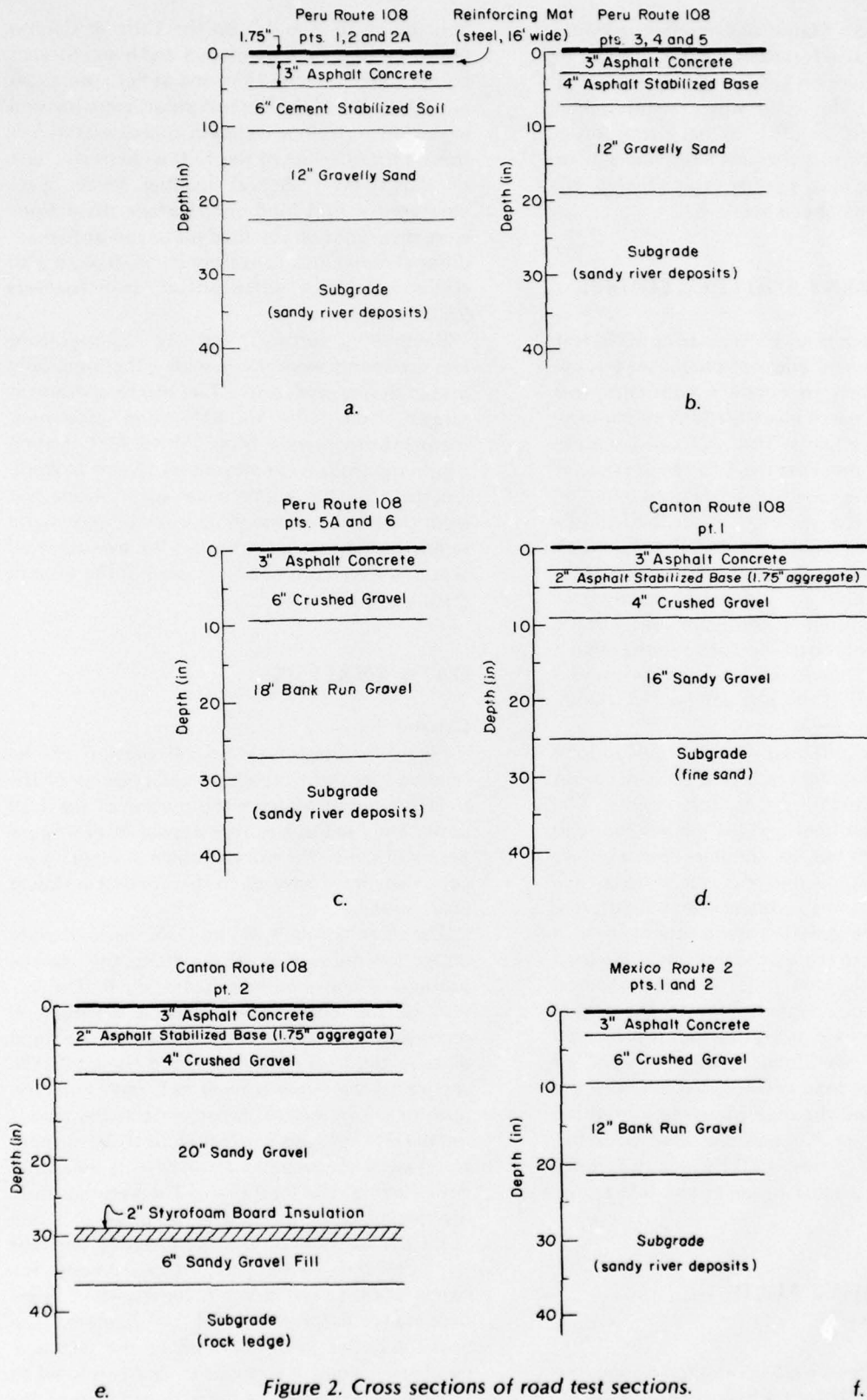


Figure 2. Cross sections of road test sections.

The "standard" Maine pavement cross section, which was also tested, is located across the Androscoggin River on U.S. Route 2 just east of Mexico (Fig. 1). This road was constructed in 1957 and consists of 12 in. of bank run gravel subbase, 6 in. of crushed gravel base, and 3 in. of asphalt concrete over a sandy subgrade (Fig. 2f). This section had not been overlaid.

## TEST EQUIPMENT AND PROCEDURES

The CRREL Repetitive Plate Bearing (RPB) test vehicle (Fig. 3) is a mobile, self-contained test apparatus developed to conduct nondestructive RPB tests for research investigations on highway and airfield pavements. This unit can generate successive load pulses in the 1 to 14-kip range at application rates of up to 20 repetitions/min. The profile of a typical series of 9-kip load pulses generated by this unit is presented in Figure 4. The test vehicle contains all the instrumentation required to provide a continuous recording of the force transmitted to the pavement surface and the motion of the pavement surface within a 4-ft radius about the load plate. Figures 5 and 6 show the interior of the test vehicle and the installed instrumentation.

For the tests conducted in Maine, a 9-kip load pulse was applied 500 times to a 12-in.-diameter load plate at each of the test points. The magnitude of the load and the size of the load plate were selected to simulate the contact pressure and surface area of a pair of truck tires inflated to 80 lb/in.<sup>2</sup> Two linear variable differential transformers (LVDT's) were mounted on a reference beam to record the motion of the load plate. Four additional LVDT's were mounted along the reference beam to monitor the deflection of the pavement along one radius projected outward from the load plate. A standard strain-gage-type load cell located between the load actuator and the load plate monitored the load pulse. Figure 7 shows the load actuator, load cell, load plates, LVDT's and reference beam during a pretest setup of the test equipment.

## FIELD REPETITIVE PLATE BEARING TESTS

The RPB field tests were conducted during the period 13-15 April 1976. Tests were conducted at

Peru points 1, 2 and 3 on the 13th; at Canton points 1 and 2, Peru points 4, 5 and 6 and Mexico points 1 and 2 on the 14th; and at Peru points 2A and 5A on the 15th. The test points were located within the outside wheel path approximately 9 ft from the centerline of the 12-ft-wide traffic lane of the roads. Actual loading levels were measured with a load cell. Surface deflections were measured on the load plate and at five additional radial distances from the load plate with linear variable differential transformers (LVDT's).

Pavement surface and air temperature measurements were made with a thermocouple probe during the tests. The air temperatures ranged from 52° to 63°F and pavement temperatures ranged from 56° to 78°F. Strong southerly winds were present on 13 and 14 April. On the 15th the winds were again strong but from the north. Although no borings were made in any of the test sections, it is believed that all sections were completely thawed at the time of testing.

## DATA ANALYSIS

### General

Figure 8 presents a typical portion of the recorded output of the load cell and one of the LVDT's used to measure the motion of the load plate. The load pulses that appear in this figure are identical to the pulses shown in Figure 4 except that these have been recorded at a slower chart speed.

The three points A, B and C on the load plate deflection curve (Fig. 8) delineate the various portions of one complete loading cycle. The section of the curve from point A to point B represents the downward motion of the load plate as the load pulse is applied. Similarly, the section of the curve from B to C represents the upward return motion of the plate as the load is removed. Thus, the vertical chart displacement from point A to point B represents the total deflection of the load plate. The vertical chart displacement from point B to point C represents the amount of this total deflection that is recovered, while the displacement between point C and point A represents a time-dependent deformation of the system. The resilient deflection is defined as the displacement from point B to point C, and is related to the elastic properties of the various layers of the

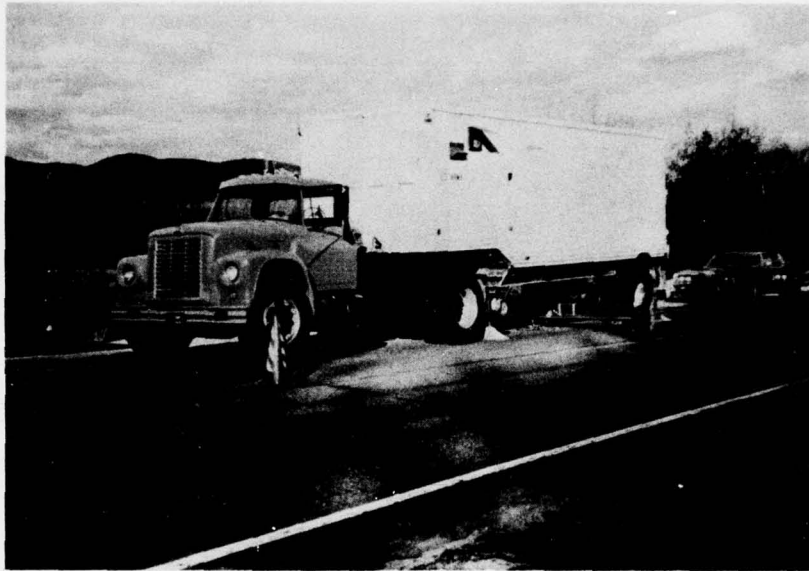


Figure 3. CRREL Mobile Repetitive Plate Bearing (RPB) test vehicle.

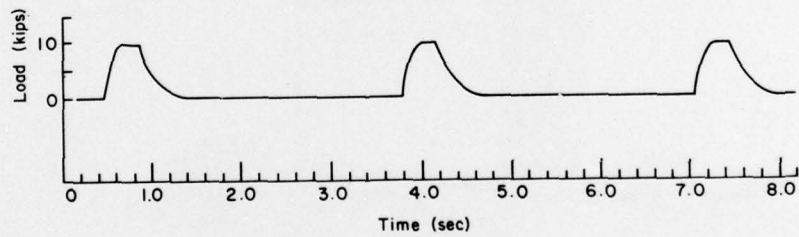
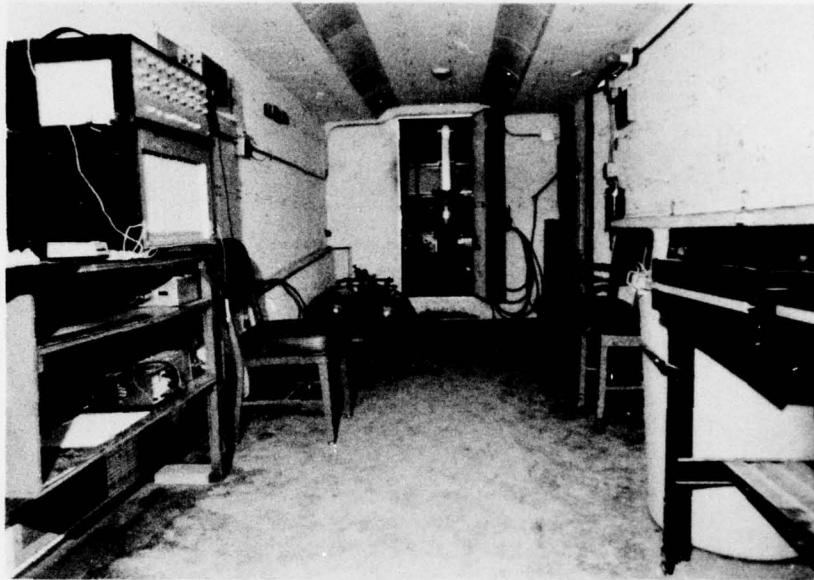
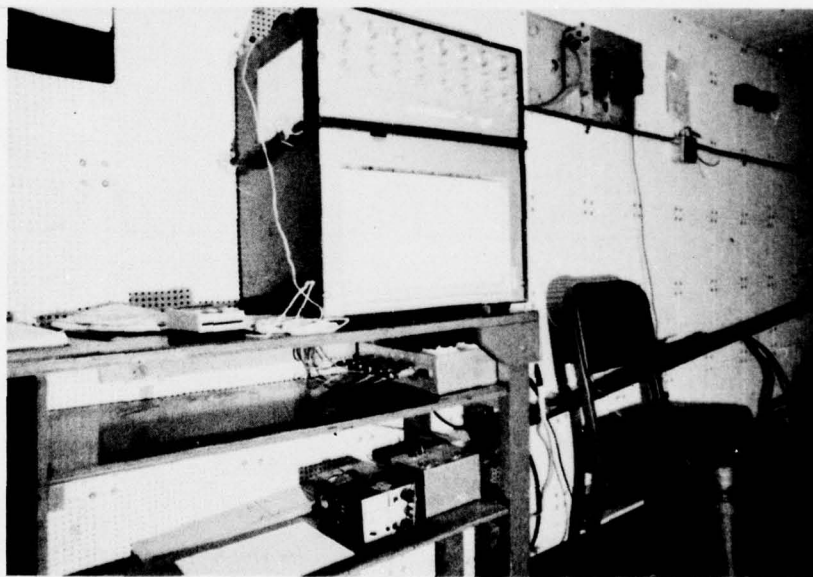


Figure 4. Typical load pulses generated by the RPB apparatus.



*Figure 5. Interior view of the RPB test vehicle.*



*Figure 6. Instrumentation installed in test vehicle.*



Figure 7. Adjusting a linear variable differential transformer before start of test.



Figure 8. Schematic of recorded output of the load cell and an LVDT mounted on the load plate.

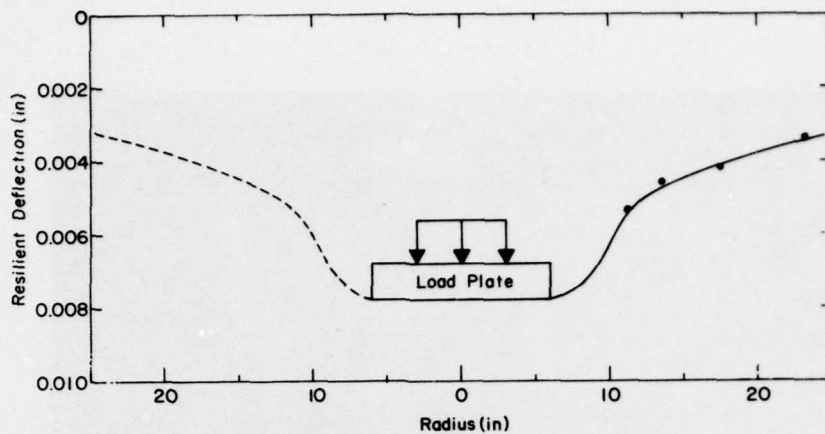


Figure 9. Measured resilient deflection profile for test point 2 at Peru.

**Table I. Typical resilient surface deflections (test point 2, Peru, Maine, cement stabilized soil base).**

Radius from center of load plate (in.)	Resilient surface deflection (in.)
5.5 (on load plate)	0.0078
11.25	0.0053
13.5	0.0046
17.5	0.0042
23.25	0.0034

pavement system. The time-dependent deformation (C minus A) is called the residual deflection and is indicative of the susceptibility of the pavement to fatigue and rutting.<sup>6 7</sup>

A typical set of resilient surface deflections for test point 2 at Peru is presented in Table I. The deflection profile for these data is illustrated in Figure 9. A complete listing of the resilient surface deflections for each test point is presented in the Appendix.

#### Layered-elastic analysis of the pavement systems

The Chevron computer program for a layered elastic system was employed to compute elastic moduli for the pavement layers using the measured surface deflections in the following manner. Initially, a set of modulus values was assumed and surface deflections were calculated. The calculated deflections were then compared with the field deflection measurements and the modulus values were revised as

required. This procedure was repeated until the calculated deflections were within  $\pm 5\%$  of the measured values. Normally, 20 to 30 revisions were required to correlate the calculated and measured deflections.

When the deflections corresponded within the specified difference ( $\pm 5\%$  of the measured deflections), it was assumed that the trial elastic moduli used in the computer program were equal to the moduli of the various layers of the pavement, and the analysis was complete.

Table II presents a complete list of the results of this analysis for each test site. Pavement cross sections for each of the test points are provided in Figure 2. In this analysis, the number of pavement layers was limited to five, except for the insulated section where it was not feasible to do so. It was assumed that Poisson's ratio for all the layers was equal to 0.40 except for the wire-reinforced pavement overlays at Peru and the insulation at Canton for which 0.30 and 0.25 were used, respectively.

Figure 10 illustrates the variations in the asphalt concrete pavement resilient modulus with temperature determined through the use of the Chevron computer program. A least squares regression analysis of the data (excluding the two widely scattered points shown in Fig. 10) indicates that, over the limited range of temperature that occurred during this test series, the resilient modulus of the asphalt concrete tended to decrease linearly with respect to increasing temperature. The empirical relationship from the regression analysis is

$$M_r = (1219 - 11.37) 10^3 \text{ kips/in.}^2$$

**Table II. Resilient moduli from Chevron Program.**

Test site	Layer number and description	Layer thickness (in.)	Resilient modulus (kips/in.)
Peru Point 1 Sta. 638 + 00	1. AC 85-100 asphalt concrete with wire mesh at 75°F	1.75	750
	2. AC 85-100 asphalt concrete	3	480
	3. Cement stabilized soil base	6	65
	4. Gravelly sand subbase	12	48
	5. Sandy river deposits/subgrade	—	37
Peru Point 2 Sta. 684 + 50	1. AC 85-100 asphalt concrete with wire mesh at 75°F	1.75	480
	2. AC 85-100 asphalt concrete	3	300
	3. Cement stabilized soil base	6	130
	4. Gravelly sand subbase	12	62
	5. Sandy river deposits/subgrade	—	27
Peru Point 2A Sta. 702 + 00	1. AC 85-100 asphalt concrete with wire mesh at 55°F	1.75	80
	2. AC 85-100 asphalt concrete	3	100
	3. Cement stabilized soil base	6	180
	4. Gravelly sand subbase	12	95
	5. Sandy river deposits/subgrade	—	40
Peru Point 3 Sta. 725 + 50	1. AC 85-100 asphalt concrete at 75°F	3	350
	2. Asphalt stabilized base	4	95
	3. Gravelly sand subbase	6	43
	4. Gravelly sand subbase	6	46
	5. Sandy river deposits/subgrade	—	32
Peru Point 4 Sta. 744 + 50	1. AC 85-100 asphalt concrete at 77°F	3	200
	2. Asphalt stabilized base	4	85
	3. Gravelly sand subbase	6	55
	4. Gravelly sand subbase	6	58
	5. Sandy river deposits/subgrade	—	25
Peru Point 5 Sta. 771 + 00	1. AC 85-100 asphalt concrete at 78°F	3	400
	2. Asphalt stabilized base	4	70
	3. Gravelly sand subbase	6	28
	4. Gravelly sand subbase	6	48
	5. Sandy river deposits/subgrade	—	27
Peru Point 5A Sta. 808 + 50	1. AC 85-100 asphalt concrete at 56°F	3	650
	2. Crushed gravel base	6	22
	3. Bank run gravel subbase	9	30
	4. Bank run gravel subbase	9	32
	5. Sandy river deposits/subgrade	—	33
Peru Point 6 Sta. 850 + 00	1. AC 85-100 asphalt concrete at 70°F	3	500
	2. Crushed gravel base	6	36
	3. Bank run gravel subbase	9	30
	4. Bank run gravel subbase	9	33
	5. Sandy river deposits/subgrade	—	35
Canton Point 1 Sta. 367 + 50	1. AC 85-100 asphalt concrete at 63°F	3	600
	2. "Grading A" asphalt stabilized base (1.75 in. max. size aggregate)	2	450
	3. Crushed gravel	4	40
	4. Sandy gravel (F2)	16	48
	5. Fine sand subgrade	—	33
Canton Point 2 Sta. 377 + 00	1. AC 85-100 asphalt concrete at 67°F	3	600
	2. "Grading A" asphalt stabilized base (1.75 in. max. size aggregate)	2	400
	3. Crushed gravel	4	35
	4. Sandy gravel subbase	20	19
	5. Styrofoam board insulation	2	1
	6. Sandy gravel fill subbase	6	20
	7. Rock ledge/subgrade	—	80
Mexico Point 1 Sta. 211 + 15	1. AC 85-100 asphalt concrete at 66°C	3	400
	2. Crushed gravel base	6	20
	3. Bank run gravel subbase	6	30
	4. Bank run gravel subbase	6	35
	5. Sandy river deposits/subgrade	—	38
Mexico Point 2 Sta. 135 + 65	1. AC 85-100 asphalt concrete at 64°F	3	200
	2. Crushed gravel base	6	33
	3. Bank run gravel subbase	6	34
	4. Bank run gravel subbase	6	35
	5. Sandy river deposits/subgrade	—	25

**Table III. Variation in resilient modulus of pavement base, subbase and subgrade materials.**

Material	Resilient modulus (kips/in. <sup>2</sup> )			Standard deviation
	Max	Min	Mean	
Crushed gravel base	40	20	31.0	7.4
Gravelly sand subbase	95	28	53.7	17.3
Bank run gravel subbase	48	19	32.6	6.8
Sand gravel fill and sandy river deposits/subgrade	40	20	31.0	5.9

**Table IV. Resilient stiffnesses for various pavement sections.**

Section description* (section thickness, in.)	Test point	Stiffness (kips/in.)	Mean stiffness and standard deviation
Asphalt concrete pavement, cement stabilized soil base, sandy gravel subbase (22 1/4)	Peru 1	1154	1150 ± 7
	Peru 2	1154	
	Peru 2A	1142	
Asphalt concrete pavement, asphalt stabilized base, sandy gravel subbase (19)	Peru 3	881	814 ± 58
	Peru 4	787	
	Peru 5	773	
Asphalt concrete pavement, crushed gravel base, bank run gravel subbase† (Peru, 27; Mexico, 21)	Peru 5A	686	710 ± 20
	Peru 6	735	
	Mexico 1	616	
	Mexico 2	573	
Asphalt concrete pavement, asphalt stabilized base, bank run gravel subbase (25)	Canton 1	1113	—
Asphalt concrete pavement, asphalt stabilized base, sandy gravel subbase with polystyrene insulation (37)	Canton 2	686	—

\*See Table II and Figure 9 for details.

†Peru sections have 6 in. more bank run gravel than Mexico sections.

where  $M_r$  = resilient modulus  
 $T$  = temperature, °F.

#### Statistical analysis

In a similar manner, the variations in resilient modulus can be analyzed for the various base, subbase, and subgrade materials through the use of statistics. For those materials for which there are at least five determinations of modulus, the mean modulus value and the deviation about the mean were calculated. Table III presents these results for four of the materials encountered in the test roads.

These data indicate that the maximum resilient modulus of the gravelly sand is 1.9 and 2.3 times the maximum resilient moduli of the bank run and crushed gravel, respectively, while its mean value is 1.6 and 1.7 times the mean moduli

of the bank run and crushed gravel, respectively. The modulus values of the crushed gravel, bank run gravel, and the sandy gravel fill and sandy river deposits/subgrade are nearly equal for maximum, minimum, mean and standard deviation values. The values for the crushed gravel base and the subgrade are identical except for the standard deviations. The high value for the sandy subgrade soil reflects its high quality and the higher confining pressures at greater depths in the section.

The resilient stiffness of the roadway, which is equal to the plate load divided by the resilient surface deflection measured on the load plate (Table IV), is an indicator of the load-bearing capacity of the pavement. The resilient stiffness data illustrate the variation in resilient stiffness

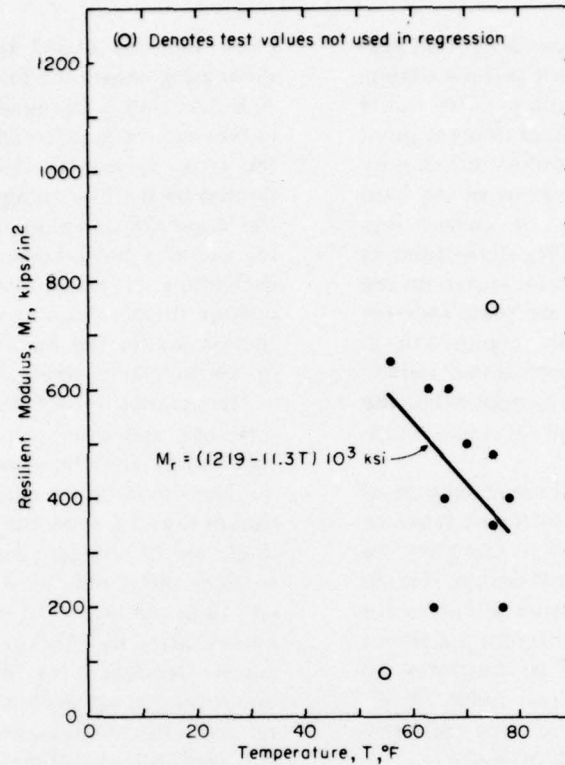


Figure 10. Resilient modulus of asphalt pavement as a function of temperature.

for different pavement cross sections.\* The mean stiffness of the sections with the cement stabilized soil base is about 1.4 times that of the sections with the asphalt stabilized aggregate base and 1.6 to 1.9 times that of the sections with the crushed gravel base. However, the sections with the cement stabilized soil base were 3 ¼ in. thicker than the sections with the asphalt stabilized aggregate base and contained a wire mesh reinforced asphalt concrete overlay (Table II).

A good comparison of stiffness with pavement thickness can be made with the results for Peru test points 5A and 6 and Mexico test points 1 and 2, where the 6-in. greater thickness of bank-run gravel at Peru resulted in about a 20% increase

in the mean stiffness over the stiffness at Mexico. A larger increase of about 37% in stiffness resulted with the 6-in. additional total thickness at Canton test point 1 over the stiffnesses at Peru test points 3, 4 and 5. An exception to this trend is seen at Canton test point 2 where it appears that the Styrofoam board insulation with the low modulus value results in a much weaker and less stiff section even though the section is 18 in. thicker than the sections at Peru test points 3, 4 and 5.

#### Flexural analysis

To analyze the flexural characteristics of each pavement section, the shapes of the deflection profiles were examined. A standard format for eliminating minor differences produced by the small variations in the applied load was developed by defining the deflection profiles in dimensionless form.

To describe the deflection profile in dimensionless form, the resilient surface deflection of

\*The effect of temperature variation was not taken into account; however, it is felt that the total section stiffness does not vary as markedly over the temperature range of 55-78°F as the resilient modulus of the asphalt pavement does.

each measurement point on the deflection profile was divided by the resilient surface deflection measured on the load plate. The radial distance to each deflection measurement point was converted into a dimensionless quantity by dividing the radius  $R$  by the radius of the load plate  $R_{plate}$ . Thus, a family of curves was generated that relates the surface deflections as a percentage of the deflection measured on the load plate to the number of load plate radii for each pavement cross section. Figure 11a illustrates the family of nondimensional deflection profile curves for the tests conducted on the asphalt stabilized aggregate base test sections at Peru test points 3, 4, and 5.

Curves representing the arithmetic average of the family of curves for the different types of cross sections were developed to compare the responses of different pavement designs. Figure 11b illustrates the average response curves for the family of curves of four different pavement cross sections tested. Curve B illustrates an anomalous response at Peru test point 2A for which design drawings show the same cross section as Peru test points 1 and 2 (Curve A).

The proposed use of the average response curves is explained in the following manner. First, let us consider a very high strength pavement system consisting of materials highly resistant to compressive and flexural deformation supported by an extremely weak subgrade. When subjected to a plate load, the entire upper surface of this system would move downward uniformly because all the vertical deflection in the system would be due to yielding in the subgrade. Thus, a dimensionless deflection profile for this system would be a horizontal line at 100% of the plate deflection.

Second, let us consider a very low strength pavement system consisting of highly deformable materials supported by a high strength subgrade. The deflection of the surface for such a system would occur only under the load plate and immediately adjacent to it and a dimensionless deflection basin curve would be a nearly vertical line at an  $R/R_{plate}$  value of one.

The first system might perform satisfactorily but would be very costly, while the second system would be prone to high maintenance costs because of pavement cracking and rutting. Therefore, a dimensionless response curve lying somewhere between the horizontal and vertical positions would represent the ideal system. The response curves in Figure 11b indicate that the

cross sections could be ranked in order of decreasing resistance to cracking and rutting as A, D, C, E and B, because of the steepness of the curves near  $R/R_{plate}$  equal to one. The ability of the cross section to distribute the load is reflected by the percentage of plate deflection at the larger  $R/R_{plate}$  values; i.e., at equal loads, stiffer sections have higher percentages of plate deflection at greater values of  $R/R_{plate}$ . The greater stiffness can be the result of a thicker section and/or the material strength properties of the pavement layers including the subgrade.

The relative effectiveness of the layers with different materials can be evaluated if their thicknesses and the applied load are the same and the subgrade strengths are essentially equal. Curves D and E show the effect of an increase in thickness of the pavement cross section. The sections differ only by 6 in. of bank run gravel, i.e., 18 in. for D and 12 in. for E. The mean stiffnesses differ by 116 kips/in. (Table IV) and the greater tendency for load puncture (cracking and rutting) is apparent in Curve E (Fig. 11b). The relative effectivenesses of the cement stabilized soil and asphalt stabilized aggregate bases cannot be determined because of the difference in thicknesses. However, the benefit of asphalt stabilization is fairly apparent when making a comparison of curves C and E. The asphalt stabilized section has a 220-kips/in. greater stiffness than the nonstabilized section even with 2 in. less thickness. Also, a comparison of the stiffnesses for sections C and D shows a greater stiffness for C (by 104 kips/in.) than for D, which is 8 in. thicker than C.

The final discussion of this test series deals with the residual deflection of the pavement surface during testing. Figure 12 presents the permanent surface deflection of the load plate during testing as a function of load applications for Peru test points 3, 5, and 5A. The results indicate that the permanent surface deflections increase linearly with the logarithm of the number of load repetitions. Thus, the settlement that occurs between the first 10 load applications equals the settlement that occurs between 100 and 1000 load applications. It appears that the weaker subgrade at test point 5 might be the cause of the greater permanent surface deflection (see Table II and the Appendix).

Unfortunately, more data are not available for these tests because of several problems encountered during testing. Uneven settlement of the load plate and movement of the reference

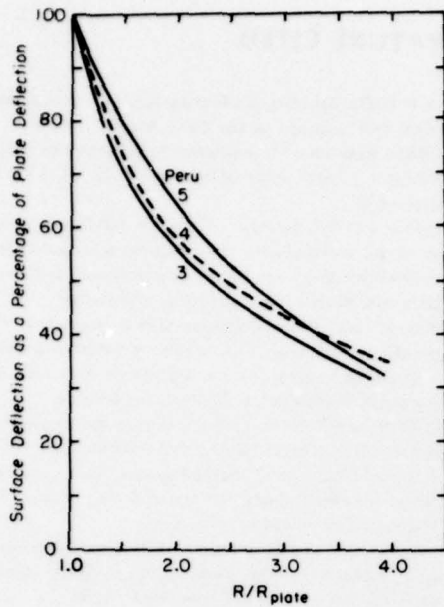


Figure 11a. Dimensionless deflection profiles for asphalt stabilized base test section at Peru.

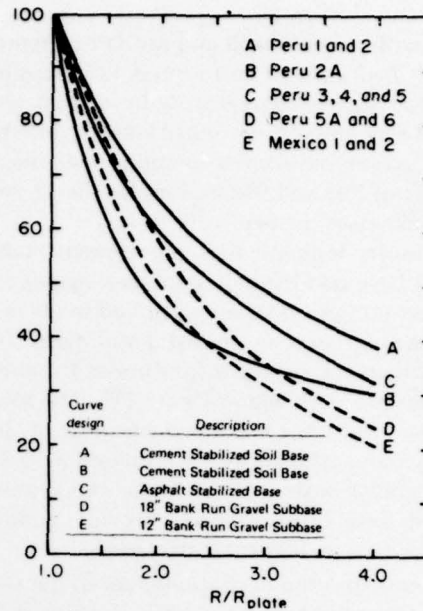


Figure 11b. Average dimensionless deflection profiles for various cross sections.

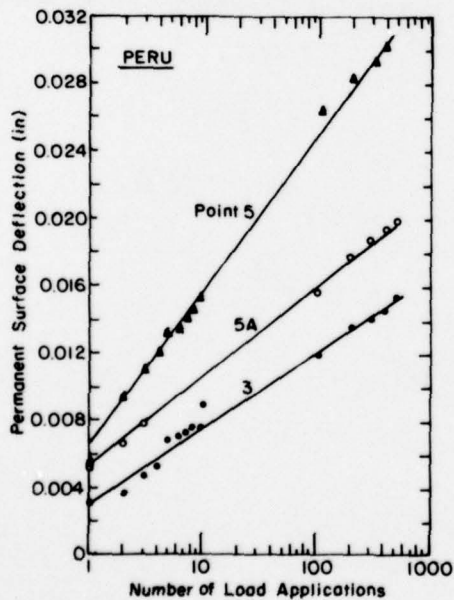


Figure 12. Permanent surface deflections during testing as a function of the number of load applications for Peru test points 3, 5 and 5A.

beam caused by passing traffic made it impossible to determine accurately the residual deflections during most of the testing. Fortunately these two factors do not affect the capability of determining resilient deflections. Methods of alleviating these problems are currently being investigated.

## SUMMARY AND CONCLUSIONS

Twelve Repetitive Plate Bearing tests were conducted on two in-service roads in Maine from 13 through 15 April 1976.

The pavement cross sections tested included a cement stabilized soil base, an asphalt stabilized aggregate base, and a crushed gravel base.

Test results indicate that the crushed gravel base had a mean resilient modulus of 31 kips/in.<sup>2</sup>; the gravelly sand subbase had a mean of about 54 kips/in.<sup>2</sup>; the bank run gravel subbase had a mean of about 33 kips/in.<sup>2</sup>; and the subgrade of sandy gravel fill and sandy river deposits had an average of about 31 kips/in.<sup>2</sup>

The cement stabilized soil base cross sections

overlaid with a steel mesh and asphalt concrete pavement had a mean stiffness of 1150 kips/in. The asphalt stabilized aggregate base cross sections had a mean stiffness of 814 kips/in., and the crushed gravel base cross sections had mean stiffnesses of 710 and 594 kips/in. for the 27 and 21-in. thicknesses, respectively.

Test results indicate that the cement stabilized soil base and asphalt stabilized aggregate base cross sections distribute applied loads in a similar manner except at test point Peru 2A, which indicated a surface rutting under a slightly higher loading as shown in Figure 11b. The total deflections were not the same because of different pavement thicknesses. Compared with the crushed gravel base cross sections, the asphalt stabilized base cross sections provide greater strength with 8-in. less total thickness.

It appears that the insulation layer in the section at Canton test point 2 at a burial depth of 29 in. contributed to a somewhat less stiff section because of the low modulus value. However, the bank run gravel and sandy gravel fill layers in the section also had relatively low modulus values; this might indicate a difficulty in obtaining compaction during construction because of the presence of the insulation.

Residual deflection of the surface during testing appears to increase linearly with the logarithm of the number of load repetitions. Thus, the settlement that occurs between the first 10 load applications would equal the settlement that occurs between 100 and 1000 load applications.

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**Appendix A. Resilient surface deflections for the test points.**

<i>Location and load (lb)</i>	<i>Radius (in.)</i>	<i>Resilient surface deflection (in.)</i>	<i>Location and load (lb)</i>	<i>Radius (in.)</i>	<i>Resilient surface deflection (in.)</i>
Canton	5.50*	0.0082	Peru	5.50	0.0116
Test point 1	10.75	0.0062	Test point 4	11.50	0.0069
9125 lb	14.25	0.0051	9125 lb	14.50	0.0058
	17.13	0.0043		17.50	0.0053
	23.00	0.0031		23.50	0.0041
Canton	5.50	0.0133	Peru	5.50	0.0118
Test point 2	11.50	0.0101	Test point 5	11.00	0.0081
9125 lb	15.00	0.0077	9125 lb	14.50	0.0064
	18.25	0.0066		17.25	0.0055
	24.00	0.0045		23.50	0.0038
Peru	5.25	0.0078	Peru	5.50	0.0133
Test point 1	11.75	0.0051	Test point 5A	11.25	0.0083
9000 lb	15.00	0.0043	9125 lb	14.75	0.0059
	18.25	0.0037		17.75	0.0048
	23.75	0.0027		23.75	0.0031
Peru	5.50	0.0078	Peru	5.25	0.0119
Test point 2	11.25	0.0053	Test point 6	11.50	0.0068
9000 lb	13.50	0.0046	8750 lb	14.75	0.0051
	17.50	0.0042		17.38	0.0044
	23.25	0.0034		23.13	0.0030
Peru	5.50	0.0081	Mexico	5.50	0.0146
Test point 2A	10.88	0.0041	Test point 1	11.75	0.0075
9250 lb	14.25	0.0033	9000 lb	14.88	0.0053
	17.00	0.0029		18.00	0.0040
	23.25	0.0025		23.75	0.0027
Peru	5.25	0.0105	Mexico	5.50	0.0157
Test point 3	10.50	0.0064	Test point 2	11.00	0.0085
9250 lb	13.75	0.0054	9000 lb	14.25	0.0066
	17.00	0.0046		17.38	0.0057
	22.75	0.0033		23.25	0.0039

\*First value for each point is for LVDT mounted on 12-in.-diameter load plate.

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Smith, N.

Nondestructive testing of in-service highway pavements in Maine / by N. Smith, R.A. Eaton and J.M. Stubstad. Hanover, N.H.: U.S. Cold Regions Research and Engineering Laboratory; Springfield, Va.: available from National Technical Information Service, 1979. v, 20 p., illus.; 27 cm. ( CRREL Report 79-6. )

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