Effect of Freezing and Thawing on Resilient Modulus of a Granular Soil Exhibiting Nonlinear Behavior

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Freeze-thaw cycles experienced in areas of seasonal frost can cause wide variations in the supporting capacity of subgrade materials. The U.S. Army Cold Regions Research and Engineering Laboratory is currently engaged in a program to assess these variations in a number of soils used in roadway and airfield construction. The complete testing and analysis procedure for one of these test soils is presented. The procedure uses a layered elastic analysis to link laboratory and field test results. The component materials of experimental road and airfield sections are evaluated in laboratory repeated-load triaxial tests, and the resilient characteristics are determined as a function of imposed stresses, soil moisture tension, dry density, and temperature. Repeated-plate-bearing and falling-weight-deflectometer tests yield surface deflection basins for the test sections at various times throughout the year. Temperature and soil moisture tension are continuously monitored in the field so that the layers of the test section can be characterized at the time the surface deflections are evaluated. The laboratory results have been verified by using statistical models of material behavior to generate various soil-layer properties commensurate with the prevailing conditions of a given field test. An elastic analysis performed by using the layer properties thus obtained and a surface deflection basin measured in the field test have shown, in general, good agreement. A technique is discussed whereby a single soil specimen is tested several times to determine the resilient properties at various levels of soil moisture tension. The computer analysis is discussed, and actual and calculated deflection basins for several points in time are presented.

The supporting capacity of subgrade materials for roads and airfields can vary drastically through the freeze-thaw cycles and subsequent spring-summer recovery experienced in areas of seasonal frost. Currently, work is proceeding at the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) toward evaluating these fluctuations in material properties for a number of test soils and toward development of suitable predictive models. Both laboratory and field tests are being conducted.

Repeated-load triaxial tests were performed to determine the resilient characteristics of the component materials from experimental pavements under conditions designed to simulate those prevailing in the field tests. Empirical relations were then generated by standard statistical techniques to express the resilient modulus M_r as a function of density, soil moisture tension, and the stresses imposed in the triaxial tests.

A repeated-load plate bearing (RPB) apparatus and a falling-weight deflectometer (FWD) were used to determine the response of a paved soil test section in terms of surface deflection basins. The surface response was measured at critical times between late fall and spring to characterize the materials throughout the freeze-thaw-recovery cycle.

The validity of the laboratory results was then demonstrated by the use of a layered elastic analysis of the deflection basins. For this purpose, key environmental and physical parameters representing the in situ materials, as determined from field data, were applied to the empirical models to generate regression coefficients and resilient moduli for use in the deflection-basin analysis. The analysis yielded stresses, strains, and resilient vertical displacements throughout the system; a deflection basin was thus generated and compared with the deflection basin actually measured in the field.

This cycle of laboratory testing and field verification is being carried out for a number of soils at two test sites: the first at the frost-susceptible soil test sections of the Massachusetts Department of Public Works at Winchendon, Massachusetts, and the second at the Albany County Airport at Albany, New York. The results obtained for the soils of the Ikalanian sand test sections at Winchendon are presented here. Results and analysis of repeated-load triaxial tests to characterize the asphalt concrete pavement, the Ikalanian sand test soil, and the natural sandy gravel subgrade are given. The field testing is described, and an analysis of several representative deflection basins is presented. An improved technique for laboratory simulation of the strength-recovery phase of the test soil is also described.

BACKGROUND

This work deals with the development of predictive techniques for the resilient behavior of subgrade soil and granular unbound base courses throughout the seasons. It is part of an extensive research project jointly funded by the U.S. Army Corps of Engineers, the Federal Aviation Administration (FAA), and the Federal Highway Administration (FHWA) (1).

The correlation of laboratory and field repeated-load test results in cold regions has been of interest at CRREL for some time, and the framework for the material presented here has been given earlier (2,3).

Techniques have been developed (4) that allow the monitoring of periodic changes in soil moisture tension induced by changes in moisture content without removing the specimens from the triaxial device. The in situ recovery process of the soil is simulated by the induced increase in moisture tension. The moisture-tension levels were found to correlate reasonably well with the resilient modulus of the soil, which confirmed relations previously observed by other researchers (5,6). Thus, soil moisture tension measured in situ through cycles of thawing and recovery can be used to provide the needed link for laboratory assessment of time-dependent seasonal variation of resilient modulus.

Techniques for the measurement of soil moisture tension in the field have been developed (7). With suitable instrumentation to collect data throughout an annual cycle, moisture-tension profiles can be obtained for a pavement test section as a function of time. In this research, moisture tensions were measured in the test section during the time the field loading tests were performed to indicate the degree to which the soil had recovered from the thaw-weakened condition. These values of moisture tension are then used in the appropriate regression equations, developed from laboratory tests, to generate input moduli for the elastic layered analysis of the field deflection basins.

In the past, several devices have been used to generate surface deflection basins in loading tests on pavements. In work previously cited (3), an RPB apparatus was used that generated a 1-s load pulse applied at 3-s intervals. Surface deflections were measured by means of displacement transducers mounted on a reference beam. Another device that has different operating characteristics but is used for the same purpose is the FWD. In the FWD tests, a mass of 150 kg falls freely and strikes a shock-

absorbing device that imparts a 28-ms pulsed load to a 300-mm-diameter plate resting on the pavement surface. Surface deflections are measured by integrating the output of geophones placed on the pavement surface. The advantages of this device have been discussed elsewhere (8). In the course of this work, both the RPB and the FWD devices were used to generate deflection basins.

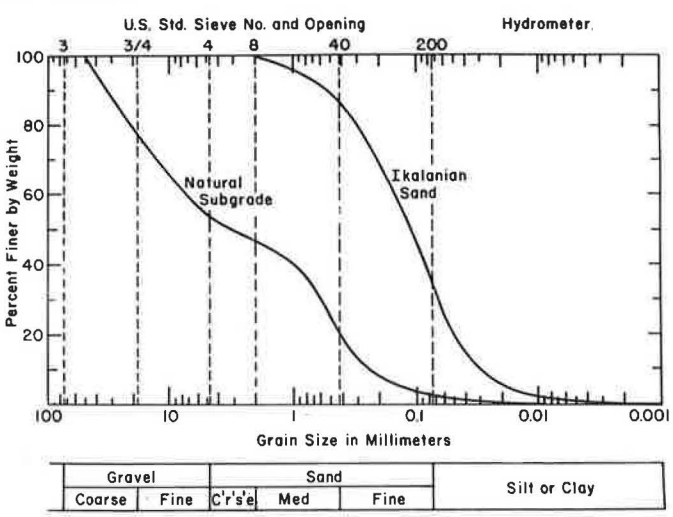
A number of computer routines have been developed to analyze the response of road systems to applied loads. A program based on layered elastic theory has been used extensively for such analyses at CRREL. The CHEVRON program (9) was used in earlier work and has subsequently been modified to incorporate nonlinear materials parameters, which provides additional versatility. The results obtained with this program have recently been compared with results obtained by others (10) by using a finite element technique. The indications were that the nonlinear layered elastic program calculated stresses very similar to those obtained by the finite element analyses throughout most of the system. Thus, the nonlinear elastic layered approach used here appears to yield reasonable results for the purpose at hand.

SAMPLING OF TEST SECTION

The Ikalanian sand test section used in this research consists of about 50-90 mm of asphalt concrete and 1.5 m of a nonplastic silty fine sand, designated the Ikalanian sand (a clean, gravelly sand), overlying the natural subgrade. Grain-size curves are shown in Figure 1. The water table is at a depth of about 1.4 m below the pavement surface. A cross section of the test site is shown in Figure 2.

Core samples of the asphalt concrete and the Ikalanian sand test soil were taken in the fall of 1978, prior to any frost penetration. Bag samples of the sandy gravel natural subgrade material were also obtained. In midwinter of 1979, once frost penetration was sufficient, frozen cores of Ikalanian sand were obtained from the test section.

Figure 1. Grain-size distribution for Ikalanian sand test soil and natural sandy gravel subgrade.



TESTING PROCEDURES

Asphalt Concrete

In the initial laboratory investigations for this phase of the work, the resilient modulus of the asphalt concrete was measured in indirect tension. Cores 101.6 mm in diameter were tested in repeated indirect tension at two load durations and at a range in temperature from about -10°C to +32°C.

To perform cyclic load tests under uniaxial compression, a suitable length-to-diameter ratio was required. This was accomplished by forming a composite cylindrical specimen from three 101.6-mm-diameter, 50- to 90-mm-long cores, which yielded a test specimen 200-250 mm in length. These specimens were then tested in unconfined compression at temperatures of -10°, 5°, 25°, and 39°C, and cyclic axial stresses of 69.0, 103.4, 137.9, 172.4, and 241.3 kPa. Two hundred cycles were applied at each stress level. Three loading waveforms were used: a 1-s pulse applied every 3 s, which simulates the RPB load pulse; a continuous haversine wave form at 1, 4, and 16 Hz (ASTM D3497-76T); and a 28-ms haversine pulse every 2 s, which simulates the FWD load pulse.

Frozen Soil

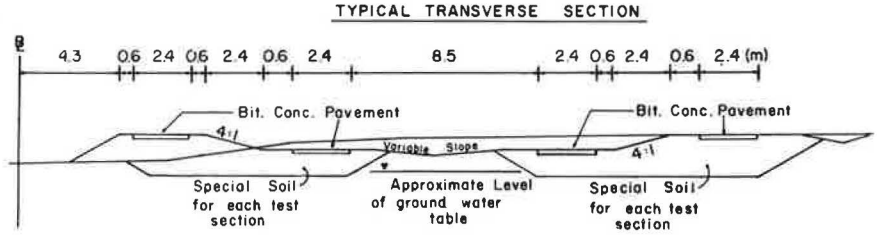
The frozen cores, 50.8 mm in diameter, were trimmed to a length of 127.0 mm and tested in a triaxial cell. This cell is equipped with an internally mounted load cell, a system of four linear variable differential transformers for measuring axial deformation, and three noncontacting displacement transducers for measuring radial deformation. A detailed description of this equipment may be found elsewhere (3,11).

Test temperatures ranged from -0.5° to -10°C. Confining pressure was maintained constant at 69.0 kPa. Deviator stresses ranged from 69 to 827 kPa. A recirculating-air environmental chamber was used to control temperatures to ±0.2°C. The lower platen of the testing machine was refrigerated independently to eliminate any thermal gradient problems.

Thawed Soil

After specimens were tested in the frozen state, they were thawed in the triaxial device and retested in order to characterize the thawed condition. For the first thawed test, stresses were kept at relatively low levels to avoid excessive permanent deformation of the specimen. After this test, the top cap was removed and the specimen was air dried to attain a somewhat higher level of moisture tension ψ. Specimens were allowed to equilibrate

Figure 2. Cross section of Winchendon test site.



overnight before being retested. Deviator stress ranged from 6.9 to 69.0 kPa, and confining pressure ranged from 3.4 to 103.4 kPa. This procedure was repeated until the thawed specimen was tested at three separate levels of ψ. Thus, the resilient modulus M_r was determined for several conditions that were representative of the recovery process experienced in the field. This technique resulted in a much greater amount of information per sample than was previously possible.

The moisture tension was monitored by means of a porous tip mounted at the center of the porous drainage element.

Recovered Soil

The unfrozen cores, 57.2 mm in diameter, taken in the fall to represent the fully recovered state, were trimmed to a length of 127 mm and tested under the same stress levels as the thawed specimens. However, the recovered specimens were tested only at the in situ level of moisture tension that prevailed when the samples were taken.

Subgrade Material

Specimens measuring 152.4 mm in diameter and 381.0 mm in length were compacted at 4 percent water content and 50 percent relative density (approximately 2 Mg/m^3). The relative density of the specimens was selected to represent the estimated in situ condition of the soil, which borings showed to be of loose to medium density. The axial and radial measurement devices were the same as those used in the testing of the 50.8-mm-diameter specimens.

Each specimen was subjected to 200 cycles at stress levels selected to simulate the overburden pressure experienced within the Ikalanian sand test section, plus the stresses estimated to be generated at this depth by the applied cyclic surface load. Deviator stresses were 3.4, 6.9, and 13.8 kPa. Confining stresses were 5.5, 6.9, and 13.9 kPa. Specimens were allowed to equilibrate under the static stresses before the cyclic stress was applied.

Although the cyclic loading was performed under undrained conditions, the specimens were allowed to drain after application of each increment of static stress. The reservoir level was maintained at the top of the specimen for this purpose. The deviator stresses were applied under two different waveforms that simulated both the RPB and the FWD load pulses.

RESULTS OF LABORATORY TESTS AND STATISTICAL ANALYSIS

A stepwise multiple linear regression analysis was performed on the results in all cases. The forms of the equations given in Table 1 were selected to represent the relations suggested by the test data. The following notation is used in Equations 1-7b:

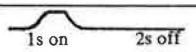
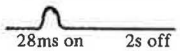
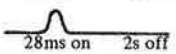
M_r = resilient modulus,
T = temperature (°C),
f = load waveform frequency (Hz),
w = moisture content (%),
τ_{oct} = octahedral shear stress (kPa) $= (1/3)[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]^{1/2}$,
ψ = soil moisture tension (kPa),
γ_d = dry density (Mg/m^3),
J_2 = second stress invariant $(\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1)$, and
J_1 = first stress invariant $(\sigma_1 + \sigma_2 + \sigma_3)$.

These models were formulated based on the use of stress-invariant parameters. The significance level for acceptance of a variable in a model was held at α = 5 percent for all analyses.

Asphalt Concrete

The asphalt concrete was found to be insensitive to the level of deviator stresses applied for each waveform. For the slower RPB waveform (1 s on, 2 s off), the resilient modulus M_r was found to be a function of temperature only, as seen in Equation 1 in Table 1. For the haversine loading according to ASTM standards, M_r was found to be a function of

Table 1. Results of regression analyses.

Material	Load-Pulse Characteristics	Regression Equation	n	R^2	SE	Equation No.
Asphalt	 1s on 2s off	$M_r = \exp(9.204 - 5.552 \times 10^{-2}\,T - 9.744 \times 10^{-4}\,T^2)$	85	0.97	0.287	1
	 Haversine f = 1, 4 & 16 Hz	$M_r = \exp[9.183 - 7.47 \times 10^{-2}(T)]\,f^{0.1777}$	158	0.81	0.469	2
	 28ms on 2s off	$M_r = \exp(9.429 - 7.47 \times 10^{-2}T)$	-	-	-	3[a]
Ikalanian sand Frozen	 1s on 2s off	$M_r = \exp[13.74 - (0.820)T - (0.0538)T^2 - (83.78)w + (141.6)w^2]\,\tau_{oct}^{-0.382}$	62	0.90	0.308	4
Thawed	-[b]	$M_r = 3.021 \times 10^4 (101.36 - \psi)^{-3.226}\,\gamma_d^{11.634}\,(J_2/\tau_{oct})^{0.480}$	119	0.89	0.276	5a
		$M_r = 8.129 \times 10^4 (101.36 - \psi)^{-3.324}\,\gamma_d^{11.578}\,J_1^{0.490}$	119	0.84	0.323	5b
Recovered	-[b]	$M_r = 5.69 \times 10^6 (101.36 - \psi)^{-3.118}\,J_1^{0.537}$	38	0.88	0.205	6a
		$M_r = 2.405 \times 10^6 (101.36 - \psi)^{-2.918}\,(J_2/\tau_{oct})^{0.442}$	38	0.84	0.238	6b
Natural subgrade[c]	 1s on 2s off	$M_r = 8.829\,J_1^{0.708}$	65	0.67	0.235	7a
	 28ms on 2s off	$M_r = 20.74\,(J_2/\tau_{oct})^{0.352}$	65	0.76	0.201	7b

Note: M_r is determined in megapascals in all equations except Equation 4, where M_r is in gigapascals.
[a] M_r estimated.
[b] Same pulse as frozen condition.
[c] Test results for both waveforms shown were merged into one data set for this analysis.

both temperature and frequency, as seen in Equation 2 in Table 1. Here, a given load was applied at three frequencies: 1, 4, and 16 Hz. The modulus is seen to increase with increasing frequency of loading.

Representative results are shown in Figure 3. The lines are generated by the regression equations.

This set of laboratory tests was intended for correlation with the field RPB test results obtained in 1979. Since all of the field testing in 1980 was carried out by using the FWD, it was necessary to characterize the asphalt concrete also under the FWD load waveform in the laboratory.

A second set of tests in repeated uniaxial compression was performed on the same composite specimens mentioned above. These tests included the same RPB and haversine load waveforms as well as the FWD waveforms. The M_r results were internally very consistent for these data but were noticeably low in relation to the original M_r values. It is believed that the previous repeated-load testing of these specimens resulted in fatigue damage that lowered the resilient modulus.

It was noted in this second data set, for the damaged samples, that the values of M_r resulting from the FWD waveform differed by an almost constant factor of 1.5 from the 4-Hz haversine waveform results over the full range in temperatures. Based on this observation, the same factor of 1.5 was applied to the 4-Hz haversine results from the original tests on undamaged samples to obtain an estimate of the FWD results on the specimens in their original condition (Figure 3).

Ikalanian Sand

Frozen State

Results for tests on the frozen soil specimen I-4-3 are shown in Figure 4. Confining pressure was not varied for these tests, since earlier research (3,12) showed it has little effect on M_r over the range of stresses that can be safely applied with the triaxial cell used. All cyclic loading was applied in the waveform of the RPB pulse.

The resilient modulus was found to be a function of temperature, moisture content, and deviator stress. The stress function is expressed in terms of octahedral shear stress τ_{oct}. The regression equation has been used to generate the line shown. A 95 percent confidence interval is plotted as well.

Moisture content and density for the specimens tested in the frozen state are given below:

Specimen No.	Dry Density (Mg/m³)	Moisture Content (%)
I-4-3	1.470	24.7
I-4-4	1.475	29.1
I-5-2	1.529	22.7
I-5-3	1.347	32.5

Thawed State

The technique of alternately testing and drying the thawed specimens to cover a greater range in specimen moisture tension and density proved highly satisfactory.

Five specimens were tested at as many as three different levels of moisture tension each. The sample properties at each stage of testing are given in Table 2. The dry densities and moisture contents for the intermediate tests are estimates generated by back calculating volumes from deformation records and by using the moisture-retention curve (see Figure 5) to determine gravimetric moisture content given a calculated density and a measured moisture tension.

As seen in Equations 5a and 5b in Table 1, the significant variables here are a moisture-tension parameter $[f(\psi)]$, which is atmospheric pressure minus the gauge value of soil moisture tension $(U_a - \psi)$; dry density (γ_d); and a stress parameter $[f(\sigma)]$. Two alternative stress functions were used in the analysis of laboratory results: the first stress invariant $J_1 = \sigma_1 + \sigma_2 + \sigma_3$ (often given as θ) and the ratio of the second stress invariant to the octahedral shear stress J_2/τ_{oct}. As of this writing,

Figure 3. Resilient modulus of asphalt concrete versus temperature for various loading conditions.

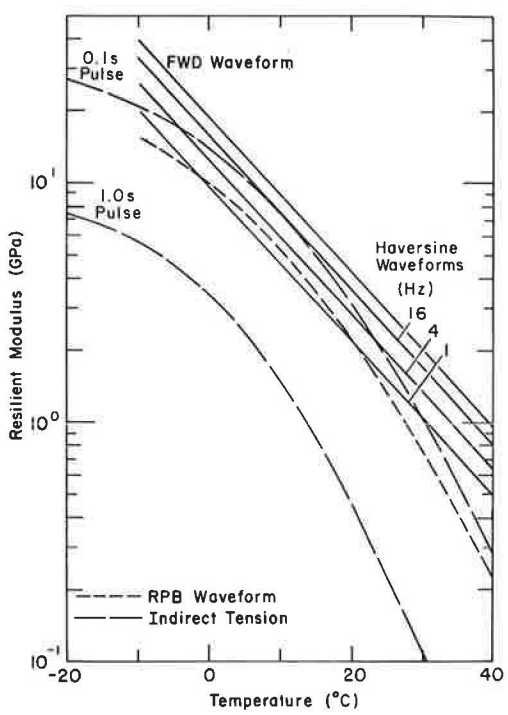


Figure 4. Resilient modulus of frozen Ikalanian sand versus temperature (regression line for Equation 3, Table 1).

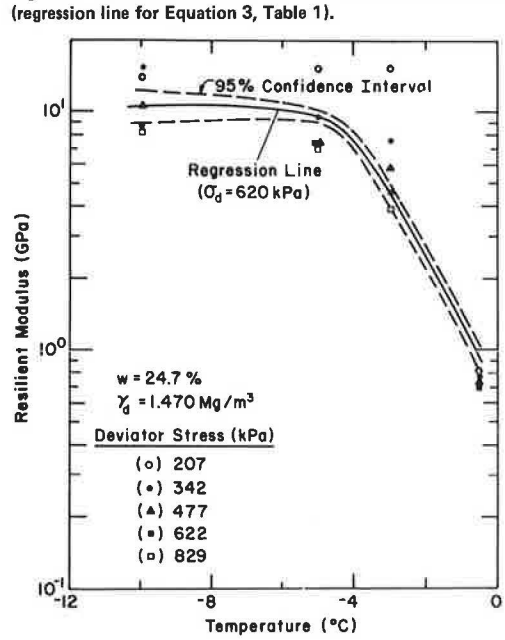


Table 2. Specimen properties for thawed Ikalanian sand.

Specimen No.	Dry Density (Mg/m^3)		Moisture Tension (kPa)			Moisture Content (%)		
	Initial	Final	Test 1	Test 2	Test 3	Test 1	Test 2	Test 3
I-2-3	1.504	1.609	0	4.0	11.5	25.8	25.0	14.0
I-3-2	1.532	1.640	0	7.0	15.0	25.8	21.5	10.0
I-4-3	-	1.638	0	4.0	7.8	24.7	23.5	20.5
I-4-4	1.576	1.690	0	5.0	9.0	21.4	21.0	18.2
I-5-1	1.541	1.656	0	4.5	21.0	25.8	24.5	7.2

Figure 5. Moisture-retention curve for Ikalanian sand.

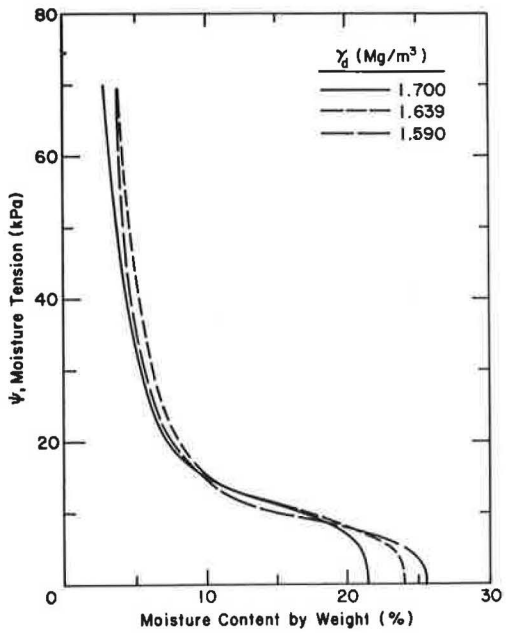
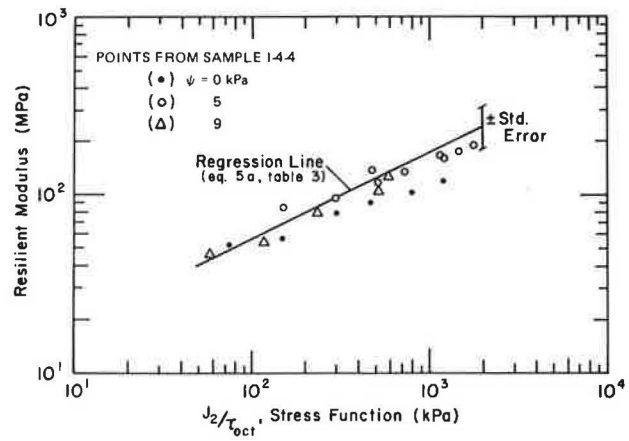


Figure 6. Resilient modulus versus J_2/τ_{oct}.



only the stress parameter has been used in the analysis of deflection basins.

The stress parameter J_2/τ_{oct} was developed by inspection to reflect two trends observed in the test data. The first trend was the often-observed tendency of M_r to increase with increasing bulk stress J_1. The second trend was somewhat more subtle in its influence but nonetheless discernible in some of the test data. This was a tendency for M_r to decrease with increasing principal stress ratio, an effect that was especially evident when σ_3 was held constant and σ_d increased. The stress parameter J_2/τ_{oct} reflects this behavior, as shown in Figure 6.

The stress parameter can be expressed in terms of the applied stresses:

$$J_2/\tau_{oct} = (9\,\sigma_3^2 + 6\,\sigma_3\,\sigma_d)/\sqrt{2}\sigma_d \quad (8)$$

The octahedral shear stress τ_{oct} is itself an invariant, since it can be expressed as a function of stress invariants (13):

$$\tau_{oct}^2 = (2/9)(J_1^2 - 3J_2) \quad (9)$$

Thus, the ratio J_2/τ_{oct} is also an invariant.

The suitability of this parameter is evident in a reasonably high correlation coefficient (r^2 = 0.89) for these test data (Equation 5a in Table 1).

Regression equations are presented in terms of the two alternative stress parameters: J_2/τ_{oct} and the more conventional J_1 (or θ) term. In most cases, the J_2/τ_{oct} parameter results in a somewhat higher correlation coefficient, but the bulk stress parameter J_1 has been used, for reasons noted below. Figure 7 shows the results of the regression analysis in which J_1 was used along with data from one thawed specimen.

The exponent of this stress function--i.e., the slope of the ln M_r versus ln J_1 curve--was found to be independent of the moisture-tension level ψ.

The slope was also assumed to be independent of density. There is an unavoidable bias in the test data that clouds the effect of γ_d somewhat; this stems from the fact that the first thawed tests, at the lowest values of ψ, also had the lowest dry densities. Through the course of testing, the sample density would increase as a result of the stress cycling. The moisture tension would also be increased as part of the testing procedure. The net result is a rather high induced covariance between the two terms. In future work, it would seem advisable to avoid this problem by testing a resaturated specimen that has been densified during previous testing. This will result in data sets with low ψ and high γ_d values, offsetting the present difficulty and better isolating the effects of the two parameters.

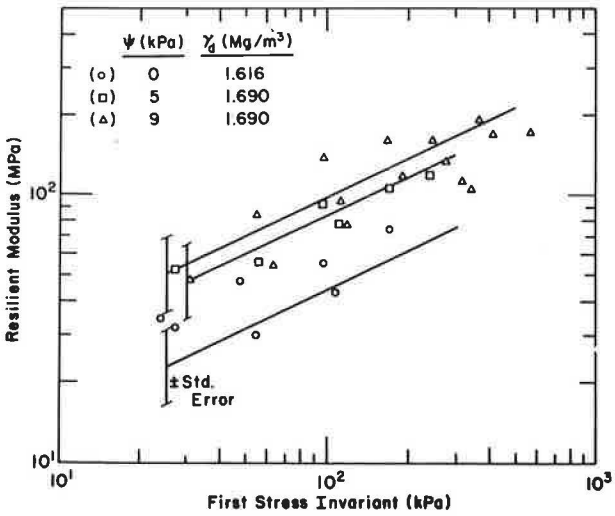
Recovered State

The results of tests on recovered specimens of Ikalanian sand reflected the same general trends as the thawed material. Specimen properties are given below:

Specimen No.	Dry Density (Mg/m^3)	Moisture Tension (kPa)	Moisture Content (%)
I-8-1	1.608	17.5	8.3
I-8-3	1.491	9.0	12.5
I-8-4	1.508	11.0	12.8

Typical results and the regression line are shown in Figure 8. The stress parameter J_1 and moisture tension were accepted by the analysis, whereas density was not.

Figure 7. Resilient modulus versus J_1 for thawed sample I-4-4 at three levels of ψ (regression lines for Equation 4b, Table 1).



Because of difficulties in obtaining and handling samples, only three specimens were tested. It is interesting to note in the table above that specimens with higher density also showed higher moisture tension. This results in a high covariance between these two variables and makes it difficult to isolate their influence on M_r. Caution must be exercised in using this model, therefore, since the effect of density is not included and hence the effect of ψ is probably exaggerated.

Subgrade

The subgrade material was tested under two loading waveforms, the RPB simulated pulse and the FWD waveform. Since no statistically significant difference was found between the results for each waveform, the two data sets were merged and reanalyzed in order to form a more extensive data base. The resulting equation, analyzed in terms of J_1 and J_2, is given in Table 1. The specimen properties of the subgrade are given below:

Specimen No.	Dry Density (Mg/m³)	Moisture Content (%)
WS-1	2.026	9.7
WS-2	2.053	10.6
WS-3	2.060	9.2
WS-4	2.085	10.3

The test data and regression line are shown in Figure 9. A 95 percent confidence interval is shown. A considerable amount of scatter is evident in these results. This is due not only to modulus variations from sample to sample but also to the apparent lack of dependence of M_r on the applied stress at low values of J_1, especially in the range of J_1 from 20 to 30 kPa.

ANALYSIS OF DEFLECTION BASINS

The primary purpose of measuring the field deflection basins was to judge the validity of the laboratory assessment of subgrade behavior. Toward this end, extensive field plate bearing testing was performed as outlined above, and it is intended to analyze the deflection basins in some detail. Several techniques will eventually be used to analyze the nonlinear moduli of the granular soils (13,14). Deflection basins will be calculated by an iterative version of the CHEVRON program, which will use stress-dependent coefficients of the moduli determined by laboratory testing; the calculated basins will be compared with the measured basins following a procedure suggested by Witczak (13). A second approach suggested by Witczak will also be applied; this approach seeks equivalent moduli that yield calculated deflections at small radial offsets that agree with measured deflections. Following the suggestions of the board of consultants convened for the project (14), other approaches, including a finite element model, will be applied to selected deflection basins. Currently, the analysis has not progressed to an extent that permits comparison of these approaches. One of these techniques, a modified elastic layer approach that uses a computer program termed Nonlinear Elastic Layer Analysis for Pavements (NELAPAV), is presented. The results of the analysis of deflection basins on three dates are given.

Figure 8. Resilient modulus versus J_1 for recovered Ikalanian sand sample I-8-3 (regression line for Equation 5a, Table 1).

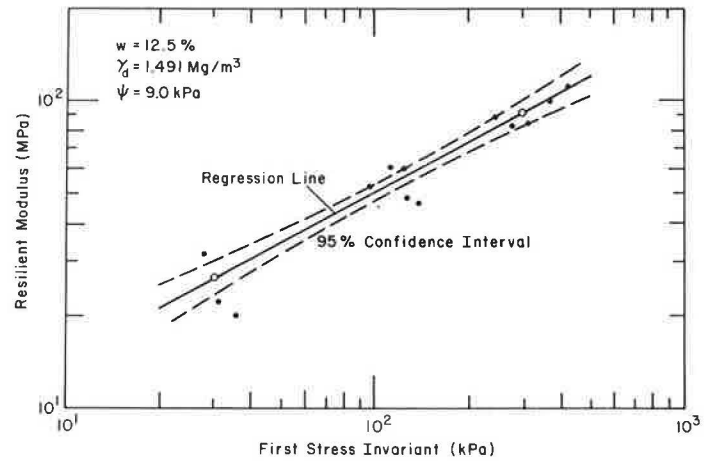
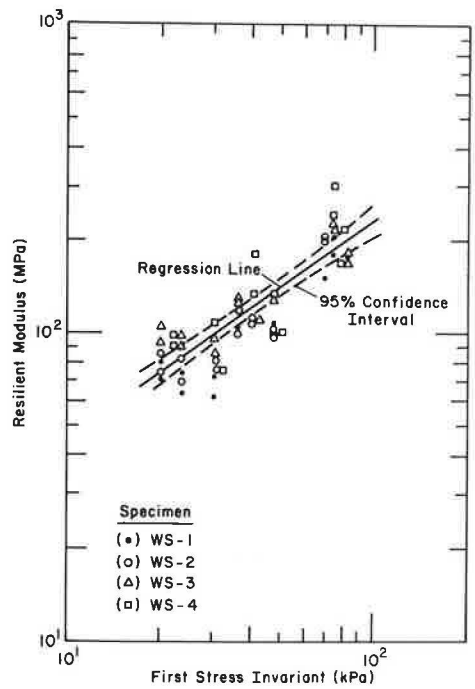


Figure 9. Resilient modulus versus J_1 for the natural subgrade (regression line for Equation 6a, Table 1).



Modified Elastic Layer Approach

The modified elastic layer approach, developed for analysis of a large number of deflection basins, offered substantial cost savings in comparison with the finite element approach, since stress calculations would be made at only a limited number of points of interest. Vertical variation in elastic modulus could easily be accommodated by introducing artificial boundaries, or sublayers, within layers of a given material. The major disadvantage of the layered elastic approach is the inability of the computational method to consider radial variation in elastic modulus. To reduce this problem, an approach has been adopted whereby the moduli are recomputed to obtain stress compatibility for each radius and depth combination that defines a point of interest.

The resulting computer program, NELAPAV, incorporates the CHEVRON elastic layer computation code. Both the CHEVRON code (including the recently developed COFE subroutine) and the BISAR code were compared, and the CHEVRON code was found to have satisfactory accuracy even for very large modular ratios, while its running cost was about one-fourth that of BISAR. To check the accuracy of NELAPAV, stresses were computed for a number of points corresponding to reported results (10) by using the finite element program FEPAV, and the results of the comparison were very satisfactory.

NELAPAV has been structured to incorporate several models of stress dependency, any one of which may be selected by the program user for any pavement layer. The NELAPAV program can easily be modified to include additional stress-dependent models as they are developed.

The major difference between NELAPAV and CHEVRON is the fact that the NELAPAV program has a "front end" that, for a nonlinear layer, begins with an assumed modulus, calculates the stress in the layer, uses the selected model to calculate a modulus, and thereafter continues to loop around until sequential modulus calculations for all nonlinear layers agree within a specified tolerance (typically 2 percent). Once the stress-compatible moduli have been determined, NELAPAV proceeds to calculate the stress, strains, and deflections in the same manner as CHEVRON. Convergence is achieved in relatively few iterations by using a relaxation technique that has been described elsewhere (15).

An innovative feature of NELAPAV is its ability to account for the horizontal variation of elastic modulus within a pavement layer. This is accomplished by recomputing the set of stress-compatible moduli for each different radial distance at which stresses, strains, and deflections are to be computed. The modulus iteration procedure is carried out at middepth for each stress-dependent layer. However, in the layer that contains the point where the stresses, strains, and deflections are to be computed, the modulus iteration is carried out at the depth of that point. Once the program has determined the stress-compatible set of moduli, the mathematics of layered elastic systems are used to calculate the stresses, strains, and deflections at that radius. This implies, of course, that there are no horizontal variations in elastic modulus. Although the modulus cannot vary radially for the calculations of responses at any individual point, different stress-compatible moduli are used for calculations at other radii. In this way, NELAPAV accounts to some extent for the horizontal variations of moduli.

Calculated Deflection Basins

For the NELAPAV analysis, deflection basins measured on three dates in 1980 were selected. These represented the Ikalanian sand in a frozen condition (February 25), a condition immediately after thawing was completed (March 12), and a condition of partial recovery (April 10). On each of these dates, deflection basins were generated by using the FWD at two drop heights. The deflections analyzed here were obtained at the greater of two drop heights.

For calculations of the deflection basins, the nonlinear modulus was expressed in terms of the first stress invariant J_1, according to usual practice. It was intended to use the stress-dependency model in terms of J_2/τ_{oct}, since this model correlates better in some cases with the laboratory data. Limited trial use of this model with NELAPAV indicated either conceptual problems related to use of this stress function in the layered system or still undetected problems with that part of the computer program. Pending further investigation of this question, it was decided to analyze the deflection basins using nonlinear moduli expressed in terms of J_1.

The measured and calculated deflection basins are plotted in Figures 10-12. The reasonably good agreement of the basins suggests that the laboratory testing yielded an acceptable characterization of the various materials. The nonlinear moduli calculated by NELAPAV in compatibility with the calculated values of J_1 also appeared reasonable. Our earlier experience with nonlinear materials in matching measured deflection basins to those calculated by an elastic layered system analysis has not always produced such favorable results. Consequently, the results presented here give reason to hope that the NELAPAV formulation of the problem will prove to be useful.

CONCLUSIONS

Based on the work presented in this paper, the following conclusions can be made.

Laboratory repeated-load triaxial tests can be used to determine the nonlinear resilient behavior of subgrade materials tested in frozen, thawed, and recovered states. Pertinent variables are moisture tension, dry density, temperature, and applied stress.

The technique of testing each thawed soil specimen at several levels of moisture tension gives a reasonable characterization of resilient modulus during the recovery phase and greatly increases the amount of information obtained per specimen.

Multiple linear regression is a suitable means of analyzing laboratory test results. The laboratory-determined moduli of the thawed materials studied here can be expressed in the following form:

$$M_r = K_1 J_1^{K_2} \tag{10}$$

where K_1 can either be a constant or a coefficient and its value is dependent on moisture tension and density. The use of J_1 (bulk stress) as a stress parameter gives good results for the materials studied here. However, somewhat better correlation is found in some cases with the use of J_2/τ_{oct} as a stress parameter.

The computer program NELAPAV, by using laboratory-determined material characteristics, generates surface deflections that are in reasonably good agreement with those observed in field tests by using the FWD.

Deflection basins can be adequately predicted when subgrade materials are in the frozen, thawed, or recovered stage of the annual freeze-thaw-recovery cycle, provided complete laboratory characterization is determined for each condition.

Figure 10. Calculated and actual deflection basins for February 25, 1980: Drop height = 219 mm and plate pressure = 582.2 kPa.

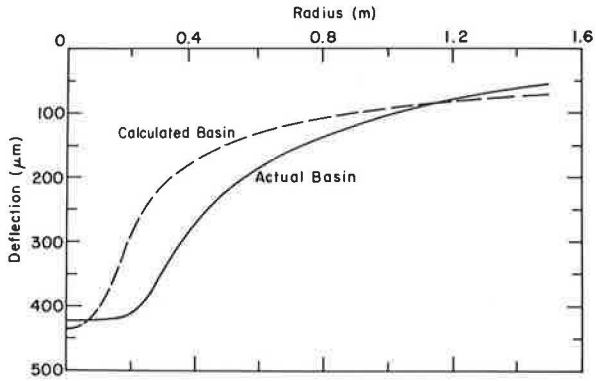


Figure 11. Calculated and actual deflection basins for March 12, 1980: Drop height = 200 mm and plate pressure = 543.8 kPa.

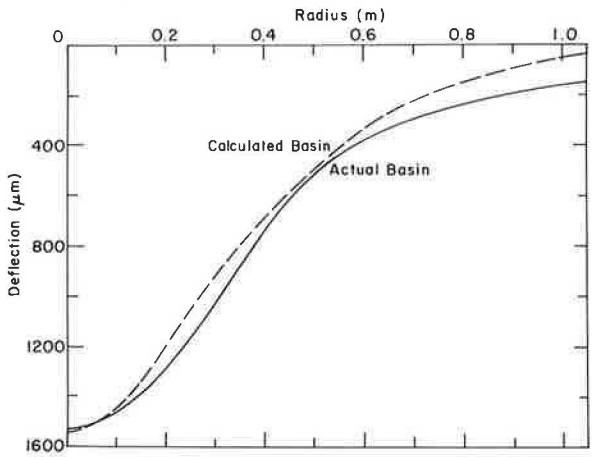
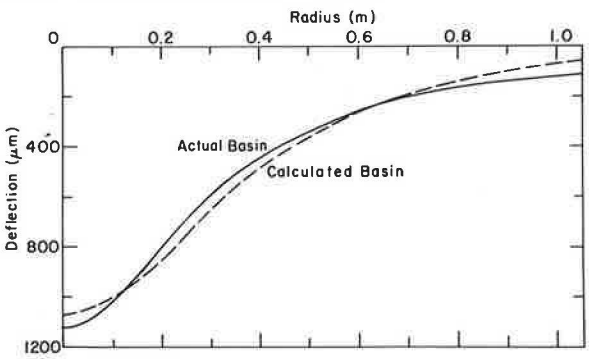


Figure 12. Calculated and actual deflection basins for April 10, 1980: Drop height = 100 mm and plate pressure = 331.6 kPa.



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