SUMMARY AND CONCLUSIONS

The state of Texas has used a stratified two-stage random sample to obtain a limited amount of highway performance data throughout the state. Highway segments 3.2 km (2 miles) long were used, and approximately 1 percent of total statewide centerline kilometers was sampled. Information on construction, traffic, climate, roughness, visually determined condition, deflection, rut depth, and skid resistance was obtained for each of the sampled highway segments. District and statewide estimates of serviceability index for 1974 and 1976 were indicated.

To examine the method and size of sampling survey currently used in Texas, simulation techniques were used on a complete set (mass inventory) of data available for one highway district, district 21. The precision (as measured by standard error) of the two-stage sampling method was shown to be superior to that of simple random sampling. In addition, a procedure of variance minimization and a utility method both indicated that about a 2 percent sample of total centerline kilometers appears to best minimize sampling error. The analysis further shows that, for Texas conditions, approximately two highway segments for each highway type should be sampled in each sampled county. The above information was determined by using four types of data: serviceability index, pavement rating score, surface curvature index (deflection), and skid number. For two of these data types, the estimates provided by the portion of the original statewide sample in district 21 are generally in reasonable agreement with the population means obtained for that district even though the sample sizes used are about half the optimum size.

The information provided by the sample sizes currently used in Texas is most reliable for statewide data estimates and next most reliable for district estimates. Current instrument, personnel, and sampling errors make small year-to-year variations in district data difficult to detect, but reductions in all three error sources are continuing to be made.

Some highway-oriented government agencies may wish to conduct a sampling survey that conforms to a selected precision. Thus, a determination of optimum sample size may not be necessary for such agencies.

A sampling survey will not answer all of the important questions about the condition and performance of a highway network, but it can provide a significant amount of valuable, relatively inexpensive information. To that end, the information contained in this paper could be used by any state or other government agency in planning a sampling survey.

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**J. P. Mahoney was with Texas A&M University when this research was performed.*

Laboratory Testing of a Full-Scale Pavement: The Danish Road-Testing Machine

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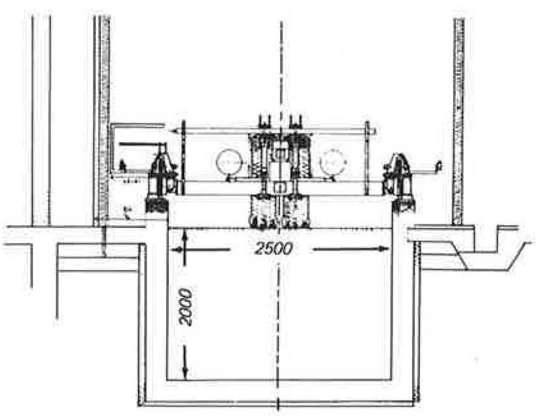
Full-scale pavements can be tested under controlled climatic conditions and with a controlled groundwater level by using the Danish road-testing machine. The response of the pavement in terms of stresses, strains, and deflections can be monitored during performance tests of a maximum ten-thousand 65-kN wheel loads/day. A qualitative evaluation of pavement response during the first two test series (0.5 million loads) has confirmed

that the machine is well suited to simulating heavy traffic loading. The response of the pavement was reasonably well predicted from linear elastic theory, considering the large scatter in the measured values and in the elastic parameters of the materials. No cracking was observed during the two performance tests although both Danish and Nottingham criteria predicted extensive cracking. Subgrade criteria (normal stress and strain) derived partly from the AASHO Road Test agreed fairly well with a decrease of the present serviceability index to about 2.5, but this deterioration was not associated with any appreciable permanent deformation of the subgrade. Finally, prediction of pavement performance in terms of rutting and present serviceability index was attempted by using a simulation program developed at the Technical University of Denmark. Reasonably good agreement was found for both of these performance criteria.

Mechanistic (or theoretical) design procedures are needed in order to design pavement structures if the materials, loads, or climatic conditions are different from those on which the empirical design methods are based. Because an increasing amount of road construction is taking place in the less developed countries and resources of traditional road-building materials are dwindling in many industrialized countries, the need for a mechanistic design procedure is growing. The ideal mechanistic design procedure should be capable of predicting the response as well as the performance of pavement structures from measured fundamental properties of the materials.

To check the validity of different design procedures—or to develop new procedures—a large number of road-testing machines (RTMs) have been developed. Most of these have been of the roundabout type, with limited facilities for controlling climatic conditions or the moisture condition of the subgrade. Recently, some linear tracked RTMs have been developed; of these, the Danish RTM is believed to be one of the most sophisticated (1).

Figure 1. Cross section of road-testing machine.



The Danish RTM can be used for fatigue testing of full-scale pavement structures under controlled climatic conditions and with a controlled groundwater level. During fatigue testing, critical stresses and strains in the structure as well as deflections, temperatures, and pore-water pressure can be monitored.

Cross-sectional and longitudinal views of the Danish RTM are shown in Figures 1 and 2, respectively. The device consists of (a) a test pit with automatic water-level control, (b) a wheel-loading system, (c) a climate chamber, and (d) a system of transducers, amplifiers, and a data logger for measuring and recording stresses and strains.

The test section of the concrete pit is 9 m long, 2.5 m wide, and 2 m deep. Finite-element calculations have shown that there is no perceptible influence from the stiff boundaries on the critical stresses or strains at the centerline of the pit.

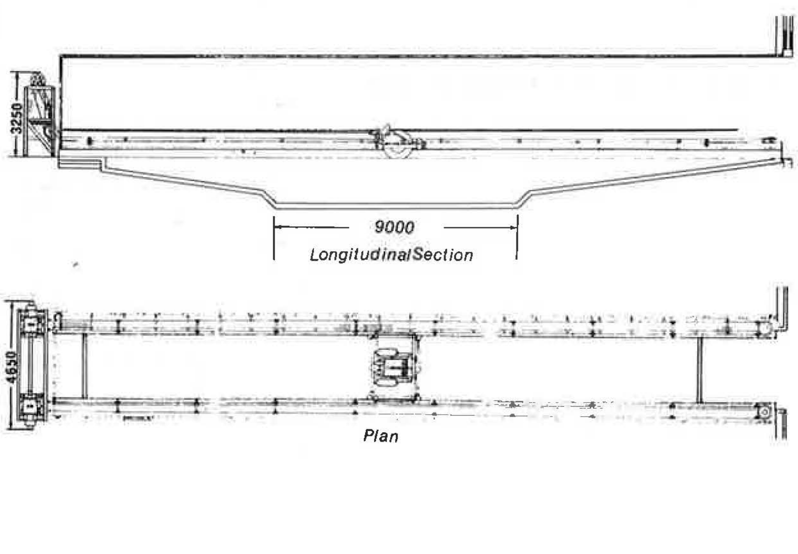
The wheel load is hydraulically applied and may consist of a single or a dual wheel. The maximum dual wheel load is 65 kN and the maximum velocity is approximately 25 km/h, which makes it possible to simulate heavy truck traffic. As many as 10 000 wheel loads can be applied in a 24-h day. This corresponds to approximately 60 000 passages of a standard 80-kN axle load. During fatigue testing, the lateral position of the wheel can be automatically changed to give a desired lateral wheel-load distribution.

The RTM is enclosed in a climate chamber that is 27.5 m in length, 4 m wide, and 3.8 m in height. Because of the large size of the chamber, ordinary construction equipment can be used. Heating and cooling aggregates make possible a temperature range of -10° C to +40° C.

The main difference between conditions in situ and in the RTM is the time scale during performance (fatigue) testing. If the loads of, say, a 10-year period are applied in only 1 year, long-term changes in the materials—e.g., hardening of the bitumen—are not allowed to take place. Moisture movements during freezing and thawing pose a special problem in this connection since accelerated testing is not possible.

Other differences between in situ and RTM conditions are hydraulic load application, loads passing in both directions, time between loadings, and variations of materials, but these, like the boundary conditions, are believed to have only limited influence on the response or performance of the pavement structure.

Figure 2. Longitudinal section and plan of road-testing machine showing climate chamber and arrangement of hydraulic motors.



INSTRUMENTATION

For a simple two-layer road structure, such as the bituminous base on a subgrade of silty sand used for the first three test series in the RTM, the stresses or strains usually assumed to be critical to the service life of the structure are at the asphalt-subgrade interface. The horizontal strain at the bottom of the asphalt layer should be limited to avoid cracking, and the vertical stress or strain at the top of the subgrade should be limited to avoid excessive permanent deformation. To predict the performance of the pavement, therefore, these values should be measured.

To find out whether the response of the pavement can be accurately predicted from layered elastic theory, stresses and strains at other depths as well as the deflection of the structure are also of interest. In this connection, the temperature of the asphalt layer and the negative pore pressure of the subgrade soil (suction) must be known in order to determine the elastic parameters of the materials. To measure these values, the test section was instrumented as shown in Figure 3 (deflection and suction gages are not shown).

The pressure cells are of the diaphragm type, and the strains in the diaphragm, when it deflects under stress, are measured by strain gages. The cells are made of titanium. A major problem with pressure cells is that the presence of the cell alters the stress field in the soil. In Figure 4, the changes from a uniform distribution caused by cells with 1- and 2-mm diaphragms are shown. The changes are seen to be highly dependent on the soil modulus E.

Theoretical considerations showed that the error in cell registration, compared with calibration under hydrostatic pressure, would be in the range of 0 to +10 percent for the soil moduli to be expected in the subgrade.

After completion of the pavement structure, an attempt was made at in situ calibration of the pressure cells. Dynamic plate loading tests in which a falling-weight deflectometer was used were carried out at different distances from the cells to obtain the stress distribution at the depth of the cells. From this stress distribution, the total vertical force on the horizontal plane through the cell can be found; this force must be equal to the force applied on the pavement surface.

Figure 3. Instrumentation of test section.

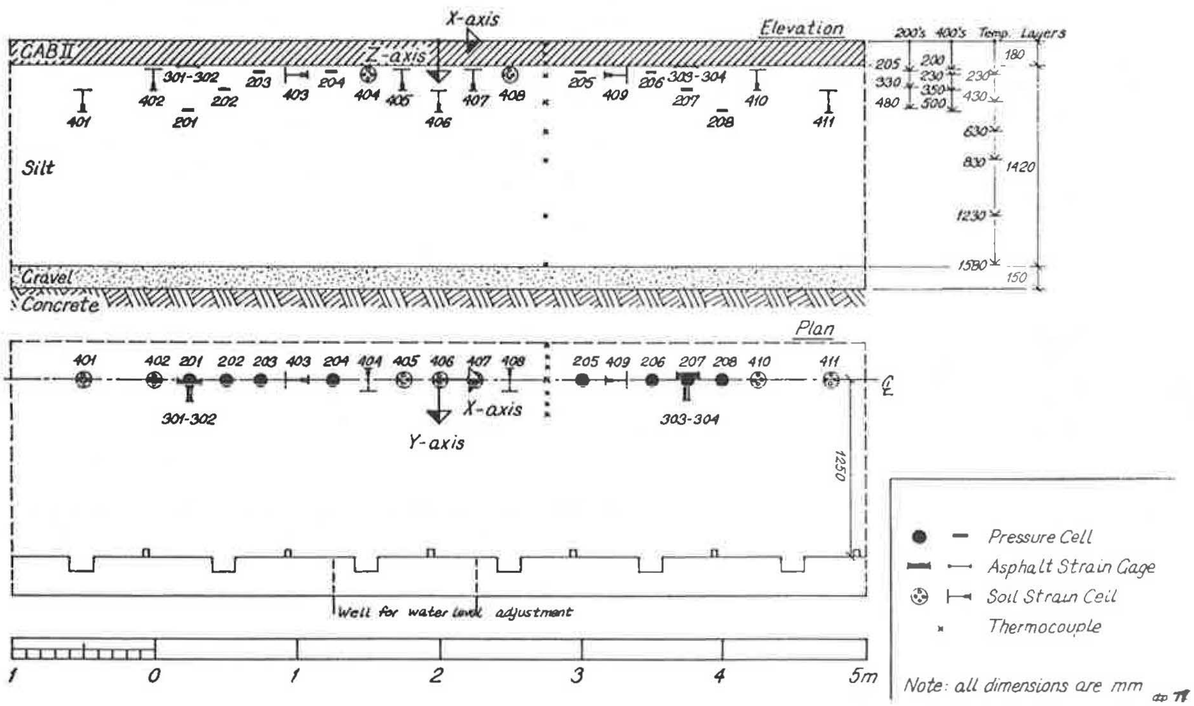
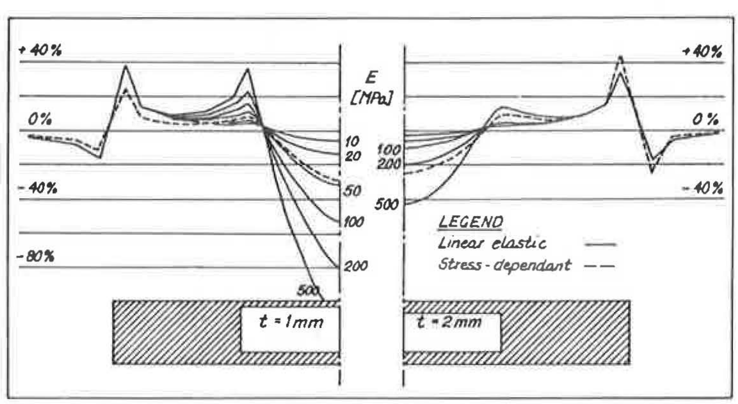


Figure 4. Excess stress distribution in diaphragm plane.



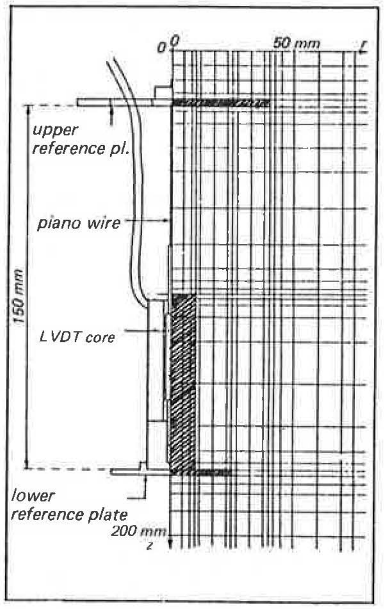


Figure 5. Soil strain cell.

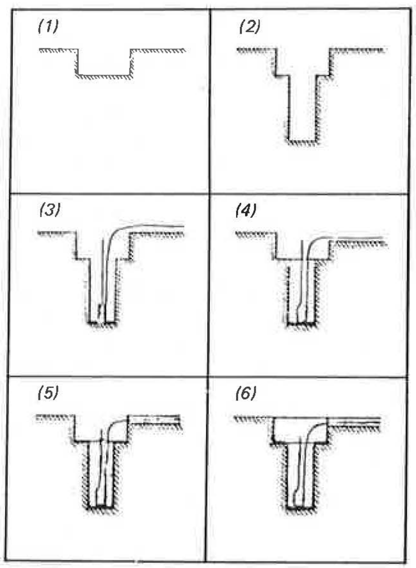


Figure 6. Installation procedure for soil strain cell.

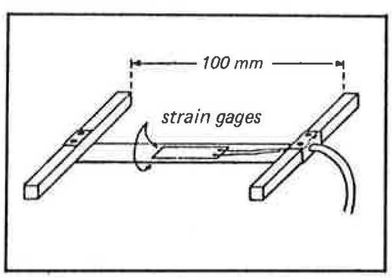


Figure 7. H-gages.

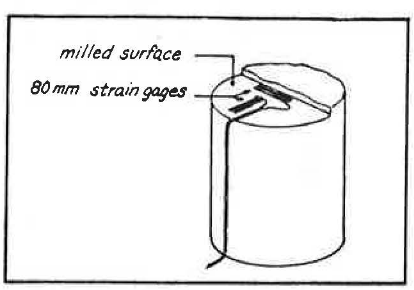


Figure 8. Asphalt gages glued to cores.

To determine the total force from the stress distribution, an integration was performed in which it was assumed that the Boussinesq stress distribution could be used and that the ratio between the observed and the actual stress was a constant (a) independent of the distance from the load because the cross sensitivity of the cells is negligible (<4 percent). By varying the depth in the Boussinesq expression, a "best" a-value (error factor) and equivalent depth were determined on the criterion that Σ (observed stress - a $\times$ calculated stress)^2 should be minimized. This resulted in equivalent depths that compared well with equivalent depths determined from the elastic moduli of the materials, but it also resulted in widely scattered error factors: from 0.9 to 3.5. For high asphalt temperatures, the mean error factor (six gages) was 1.45 and the standard deviation 0.39. The error factors were also found to be highly dependent on the temperature of the asphalt layer, which indicated that the procedure used for installing the gages—in dug-out holes after completion of the subgrade—was rather unsuccessful even though great care had been taken to fill back exactly the amount of soil dug out minus the volume of the cell. It should be noted that this calibration is correct only if the stress follows a Boussinesq distribution.

Strains in the subgrade were measured by linear variable differential transformers (LVDTs) as the relative change in length over a reference length of 100-150 mm. Finite-element calculations were made by using the net shown in Figure 5 to determine the influence from the cell on the strains in the soil. It was found that the results were much influenced by the assumption of either full friction or no friction between cell and soil. With full friction the cell would underestimate the strains by approximately 30 percent, whereas with no friction it would overestimate by 40 percent.

The effect of the installation procedure, shown in Figure 6, could not be evaluated, but it is hoped that the unsuccessful backfilling of soil found with the pressure cells will be of less importance to the strain cells because holes drilled through the upper reference plate ensure that most of this plate rests on undisturbed soil. A pair of electromagnetically coupled coils (BISON) were tried because they are much simpler to install, but they were found not to be suited for measuring dynamic strains because the signal induced by the metal masses of the loading vehicle was larger than the strain signal.

The strains at the bottom of the asphalt layer were measured by strain gages mounted on an aluminum strip anchored to the asphalt by two steel bars, or H-gages (see Figure 7). These gages functioned immediately after completion of the pavement, but they quickly deteriorated. After completion of the first test series, two gages were recovered by drilling and were found to be extensively corroded. A new attempt at measuring the strains was made by cementing strain gages to the cores drilled from the pavement (see Figure 8), and refitted in the pavement with araldite. These gages worked satisfactorily at the beginning of the second performance test, but the araldite eventually failed.

Deflections were measured by using a geophone or an accelerometer and one or two analog integrations of the signal, respectively. The geophone is the sturdier and cheaper of the two instruments. Because the signal is proportional to the velocity of the instrument and thus needs only one integration, the drift on the signal is much less than that of the accelerometer. Because the geophone does not respond to low frequencies, however, it is not very well suited for measuring deflections under a slow-moving wheel, a condition in which the accelerometer can be used, even if with some difficulty. The geophone on the other hand is an excellent deflection gage in connection with dynamic plate loading tests.

Both psychrometers and agricultural tensiometers were installed in the subgrade at different depths. The tensiometers showed that the suction (measured in millimeters) water column was equal to the height of the tensiometer above groundwater level. The suction values were too low (maximum 1.5 m of water) to be measured with the psychrometers.

Finally, 10 thermocouples were installed to record temperature at different depths.

All signals from stress and strain transducers were amplified on direct-current amplifiers with peak detectors. The peak signals were monitored by a scanner, digitalized, and fed into a programmable desk-top calculator, which would divide each signal by the appropriate sensitivity and amplification and print out the result in the desired unit (megapascals or microstrain). A maximum of 30 signals/wheel passage could be monitored. If the shape of the signals was needed, an analog record of 12 signals could be made by a UV recorder.

EQUIPMENT

The elastic moduli of the materials in the pavement structure were evaluated in situ by using the falling-weight deflectometer (FWD) and the lightweight deflectometer (LWD). These determinations were supplemented by tests on samples of the materials that used an LWD with test tank, a triaxial apparatus, and a bending machine. Rutting and longitudinal roughness were measured by a profilometer. This equipment and the interpretation of the test results are briefly described below.

The FWD is a dynamic plate loading apparatus where the load pulse is produced by a weight falling on a system of springs (2).

The duration of the dynamic load pulse is 26 ms with a peak force magnitude of 50 kN. For asphaltic materials, where the duration of the load pulse influences the modulus, the FWD impact corresponds to vehicle velocities of 40-60 km/h.

On a linear elastic semi-infinite half space the modulus of the material will be equal to the surface modulus E_0, calculated from Boussinesq's equation (3):

$$E_0 = 2(1 - \nu^2)\sigma_0 a/d_0 \qquad (1)$$

where

ν = Poisson's ratio,
σ_0 = uniformly distributed stress at the surface (contact pressure),
a = radius of the loaded area, and
d_0 = deflection at the center of the load.

If the material is nonlinear elastic or if the subgrade is layered, the surface modulus cannot be used as a materials modulus. If the nonlinearity follows the simple relation $E = C \times (\sigma_1/\sigma')^n$, the constants C and n can be determined from the surface modulus at different stress levels by using the following relation (4):

$$E_0 = (1 - 2n) \times C \times (\sigma_0/\sigma')^n \qquad (2)$$

If the modulus is constant with depth, the surface modulus $E_0(r)$ at different distances r from the center of the load will be constant. For $r > 2a$, the surface modulus can be calculated from

$$E_0(r) = (1 - \nu^2)\,\sigma_0 a^2/[r \times d_0(r)] \qquad (3)$$

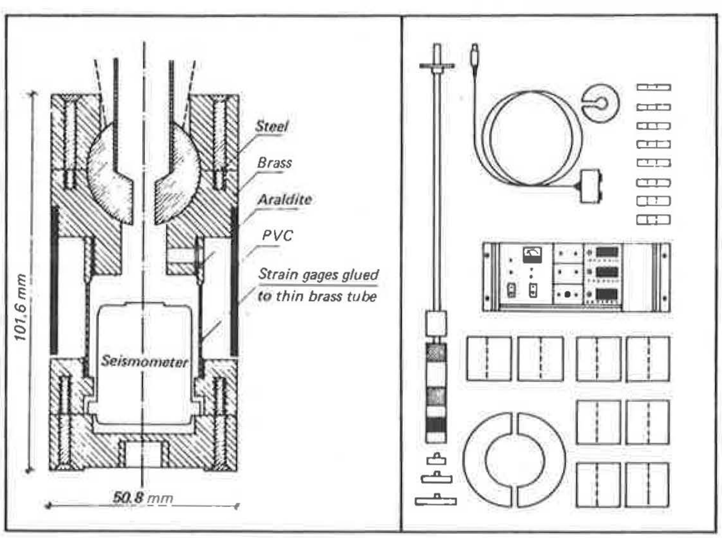
where $d_0(r)$ is the surface deflection at distance r.

Because the compression (or extension) of the material between the surface and a depth equal to the distance from the load center is negligible, the surface modulus $E_0(r)$ will reflect the surface modulus at a depth $\cong r$.

For two-layer systems, the moduli may be determined from graphs based on surface deflections if the subgrade is either linear elastic or nonlinear with n = -0.25 or -0.5 (5). For more complex structures, a nondestructive determination of the moduli is still possible through a trial-and-error approach in which the moduli are estimated, the surface moduli calculated and compared with the measured values, and changes made to the original estimates until a reasonably good fit is obtained. Tests made directly on granular materials are avoided by using these nondestructive methods. This is important for two reasons: (a) The bearing capacity at the surface of a granular material is low—zero if the cohesion is zero—and (b) the moduli of granular materials are stress dependent and will change when the surcharge is removed.

Because the critical strain at the bottom of an asphalt layer is greatly influenced by the modulus of the material immediately below the asphalt layer, an LWD was developed (see Figure 9). The modulus of a granular material can be measured directly by the LWD because a surcharge can be applied around the loaded area.

Figure 9. Lightweight deflectometer.



The LWD works on the same principle as the FWD, but the peak force and the radius of the loaded area are much smaller. The LWD is thus particularly well suited for determining moduli within a shallow depth.

Apart from in situ tests, the LWD can be used on material built into a test tank at varying moisture contents, degrees of compaction, etc. To find the elastic parameters or the constants in nonlinear elastic relations, correlations have been developed between these values and the surface deflections at varying contact pressures and surcharges (6). These correlations have been established through numerous finite-element calculations and can be used for linear as well as nonlinear elastic materials; the moduli can be dependent on either the major principal stress (in cohesive materials) or on the minor principal stress (in granular materials).

Because of the problems encountered in reproducing in situ conditions of materials and loading in the laboratory, emphasis was placed for the most part on in situ tests. Some laboratory tests have been carried out, however, partly to check on the in situ determined moduli and partly to get an estimate of Poisson's ratio.

The subgrade has been tested in a dynamic triaxial apparatus by using a simple pneumatic loading system (7). In these tests, an attempt was also made to determine the plastic characteristics of the subgrade material. The same loading system has been used for monoaxial short-term creep testing on cylindrical asphalt specimens extracted from the pavement section. Finally, three- and four-point bending tests have been carried out on prismatic asphalt specimens at temperatures between -20° C and +40° C in a frequency range of 0.01-100 Hz (8).

The performance of the pavement structure with respect to rutting and longitudinal roughness was determined by using a simple profilometer: The changes in surface level in relation to a 2-m-long aluminum rail were recorded directly on waxed paper. The slope variance was calculated from the longitudinal profile by using the slope over 225 mm at 300-mm intervals. The present serviceability index was calculated from the slope variance and rutting values by using the AASHO Road Test equation for flexible pavements (9).

MATERIALS TESTS

A number of conventional tests were made on asphalt and subgrade materials. The subgrade consisted of a non-plastic sandy silt with approximately 50 percent sand and 50 percent silt. The subgrade was constructed in 150-mm-thick lifts and compacted to 100 percent Proctor density (1700 kg/m^3) at approximately optimum moisture content (14 percent). At this density and moisture content, the California bearing ratio was found to be 18 percent and the capillary suction greater than 1 m.

The asphalt layer was a base-course material made from a well-graded, naturally occurring gravel with a maximum grain size of 20 mm. The thickness of the compacted layer varied between 170 and 180 mm. Tests made on recovered bitumen after construction of the asphalt layer showed that penetration at 25°C had dropped from the original 60 to 43. Three years later, after completion of the first test series, the penetration had dropped further to 27. The void content was rather large—11.5 percent—and compaction was 98.5 percent of the Marshall density of 2210 kg/m^3. The bitumen content was 3.65 percent by weight (standard deviation of 0.10 from five tests).

The modulus of the subgrade material was determined from dynamic triaxial tests, in situ FWD and LWD tests, and LWD tests in a test tank. The triaxial tests showed that the modulus depended on the cell pressure (σ_3), whereas the FWD tests showed a dependence on the major principal stress. In both cases, however, the nonlinearities were not very pronounced, the absolute value of the powers being in the range of 0.2 to 0.3. Thus, for the range of vertical and horizontal stresses to be expected in the subgrade, the modulus was considered to be constant in the calculation of the pavement response.

The minimum moduli were those that corresponded to a cell pressure of zero in the triaxial tests and to a contact pressure of 0.6 MPa in the FWD tests. The in situ LWD test made at a moisture content of 20 percent was carried out 800 mm below formation level. That the modulus of the subgrade decreased with depth was confirmed by FWD testing with different plate radii.

After conclusion of the first test series, the groundwater level was raised to 0.3 m below formation level. The resulting subgrade modulus was determined from FWD tests on top of the asphalt and LWD tests made in boreholes. A modulus range of 62-82 MPa was found with the FWD and 62-66 MPa with the LWD. These results agree well with the above-reported moduli at high moisture content.

Poisson's ratio, determined from dynamic triaxial tests, was found to be in the range of 0.15-0.20 for the vertical and horizontal stresses to be expected at the top of the subgrade. For large bulk stresses, Poisson's ratio decreased to about 0.1.

The modulus of the asphalt was determined through bending tests on prismatic specimens cut from the pavement. Results of tests carried out when the penetration of the bitumen was 27 are shown in Figures 10 and 11 (8). The modulus is seen to be a function of temperature, frequency, and maximum strain. Tests on specimens cut horizontally and vertically showed the asphalt to be isotropic although an analysis of the particle orientation showed a markedly horizontal orientation of the principal axes.

Figure 10. Results of bending test on bars cut vertically and horizontally.

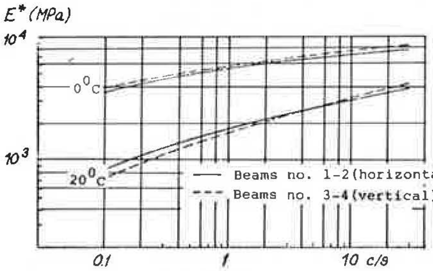
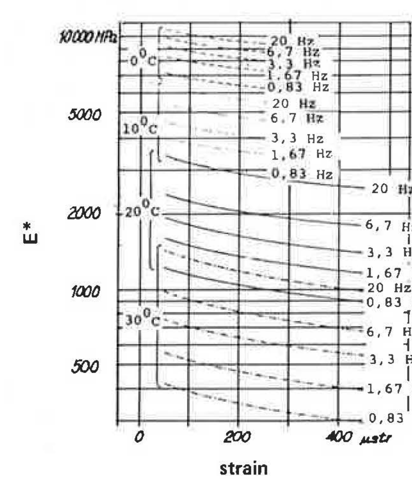


Figure 11. E* versus strain amplitude in outer fibers at different temperatures and frequencies.



FWD tests carried out during the second test series gave asphalt moduli about 30-40 percent above the values obtained from bending tests. For comparison of calculated and measured response during the second test series, the FWD-determined values were used.

The Poisson's ratio of the asphalt, determined from short-term creep tests on cylindrical specimens mounted with strain gages, was found to be in the region of 0.3-0.35 and to increase with increasing temperature.

CALCULATION OF STRESSES AND STRAINS

Three different methods were used to determine moduli and calculate pavement response and performance. To calculate the response of the pavement, a program developed by Chevron for an n-layer, linear elastic system was used (10). To determine the influence of rigid boundaries, nonlinear stress-strain relations from plate loading tests, and errors of pressure and strain cells, an axisymmetric finite-element program (11) was used (this program is a modified version of a program developed at the University of California). Finally, the method of equivalent thicknesses was used in an attempt to predict the performance as well as the response of the pavement. The method of equivalent thicknesses is much simpler than the other two methods and is therefore very useful when a large number of calculations must be done (12-14).

The basic principle of the method of equivalent thicknesses is to transform a system composed of layers with different moduli into a semi-infinite space for which Boussinesq's equations are valid. The two kinds of transformations used (see Figure 12) are as follows:

1. For calculations of the stresses and strains above the interface or the compression of the upper layer, the system is treated as a semi-infinite space with modulus E_1.
2. For calculation of the stresses, strains, and deflections at or below the interface (including the vertical stress and the horizontal strain at the bottom of the upper layer), the upper layer is transformed to an equivalent layer with modulus E_2 but with the same stiffness as the original layer.

For the stiffness to remain the same, the equivalent thickness of the transformed layer h_e must be

$$h_e = h_1 \times \sqrt[3]{(E_1/E_2) \times (1 - \nu_2^2)/(1 - \nu_1^2)} \qquad (4)$$

Because the assumptions are not quite correct, a correction factor is often introduced to obtain the same results as one would obtain with exact elastic theory.

Besides being very simple to use, the method of equivalent thicknesses can be used with nonlinear elastic materials. Finite-element calculations (4) have shown that a nonlinear relation of the type $E = C \times (\sigma_1/\sigma')$ has a negligible influence on the stress distribution. When the stress distribution is known, the modulus can be calculated at any point, and from this the strains and deflections can be found.

Because the plastic characteristics of road-building materials can often be expressed by the above nonlinear relation, even the permanent (or plastic) deformations can be calculated. To do this, the separative method (15) is used—i.e., stress state and equivalent thicknesses are determined from the elastic parameters and from these the permanent deformations are computed by using the plastic stress-strain relations.

DETERMINATION OF SERVICE LIFE

If a structural design is to make sense, the limit(s) between acceptable and unacceptable pavement conditions must be defined. The limit should preferably be based on the functional characteristics of pavements, such as safety, riding comfort, and effects on the environment. One of the most widely used criteria is the present serviceability index (PSI), a riding comfort criterion mainly influenced by the longitudinal roughness (slope variance) of the pavement surface. Roughness, expressed in centimeters per kilometer and measured with a "bump integrator", is a similar criterion, and for a regular sinusoidal pavement surface (without rutting or cracking) the approximate PSI value can be found by using

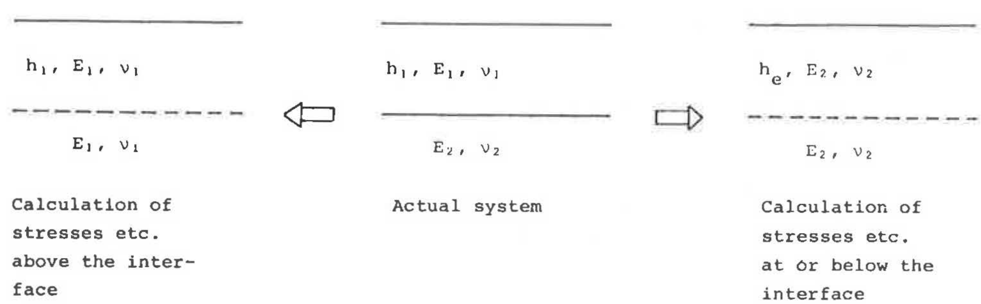
$$\mathrm{PSI} = 3.262 \log [(2 \times 10^5)/\mathrm{roughness}] - 7.25 \qquad (5)$$

Rutting does not have much influence on riding comfort, but because of its importance in relation to safety it ought to be included as a separate criterion. When ruts measure 15-25 mm (depending on camber), ponding of water is a risk.

Structural failure is often, but not always, related to functional failure. An indication of structural failure is increasing surface deflection under a given load—an easily measured factor that is often used as a design criterion. However, since overall surface deflection does not tell in which layer the failure is occurring, it is sometimes supplemented by a determination of the radius of curvature of the deflection bowl. An even better localization of failure can be obtained if several points on the deflection bowl are determined and the previously described method of fitting computed to measured surface moduli is used.

Prediction of pavement performance was attempted by using several different methods. Two methods were used to predict cracking of the asphalt layer: the Danish methods developed by Kirk (16, 17, 18) and the Nottingham method (19). Both methods relate fatigue life, in terms of number of load repetitions N, to critical tensile strain in the asphalt layer. Failure caused by excessive deformation in the subgrade was predicted by using five methods, some of which were based partly on the AASHO Road Test and all of which relate service life to either

Figure 12. Transformations used with the method of equivalent thicknesses.



critical vertical stress or vertical strain at the top of the subgrade. Rutting was calculated by using the Shell method (20) for the asphalt layer and the previously described nonlinear elastic theory with the plastic stress-strain relation for the subgrade. In the plastic stress-strain relation, two zones were used: A decreasing strain rate was assumed for strains below a critical level and a constant strain rate above that level (see Figure 13). Finally, the PSI value was predicted by using a simulation program developed at the Technical University of Denmark (21).

This program uses a modified random walk to generate important input parameters at points with given spacing (0.3 m). Both materials and structural parameters are generated to given mean values and standard deviations. The increase of permanent deformation with number of load applications is calculated at each point and, from these values, mean rut depth and slope variance are evaluated. A fatigue model is used to estimate crack propagation and, finally, the PSI value is calculated. The program has facilities for using static (constant) or dynamic loading and for varying the climatic conditions with respect to temperature changes during the year and changes in the moduli of unbound layers caused by the spring thaw.

RTM TESTING

Three test series were run on the first pavement in the RTM. The first two series consisted of response and performance tests, and the third was a freeze-thaw test.

In the response test during the first series, stresses, strains, and deflections were measured at temperatures of 0.5°C, 10°C, 20°C, and 30°C; at wheel velocities of 2.5, 5, 10, and 20 km/h; and at wheel loads of 10, 20, 30, and 40 kN. For some of these combinations, the effects of single versus dual wheel load, lateral wheel position, and tire pressure were also examined. The measured quantities were then compared with values calculated by using layered elastic theory.

During the first performance test, loading was done in two tracks 1 m apart by using a 20-kN and a 30-kN load, respectively. The tests were made at a temperature of 30°C and a wheel velocity of 20 km/h. In each track, a lateral wheel-load distribution that approached a normal distribution was used and, for each 30-kN load, five loads of 20 kN were applied to get equal damage in the two tracks if the AASHO load-equivalency factors were applicable. The groundwater level was kept at the bottom of the pit during this first test series but was then raised to 0.3 m below formation level for the second and third series.

In the second series, response tests were repeated at 10, 20, and 30 kN at temperatures of 10°C and 30°C, and measured and calculated values were again compared. This time, the performance test was carried out at a temperature of 25°C at the centerline of the pit, which made it possible to follow changes in critical stresses and strains. Longitudinal roughness was measured in addition to rutting, and the change in PSI values was calculated from the difference between these values and the original surface profiles.

In the third test series, the pavement was frozen to a depth of approximately 0.5 m. This resulted in a frost heave of 40 mm and longitudinal cracking of the asphalt layer. During thawing, a brief performance test of 5000 loads was made. It did not produce any additional cracking of the asphalt or any appreciable increase in permanent deformation.

ANALYSIS OF RESULTS

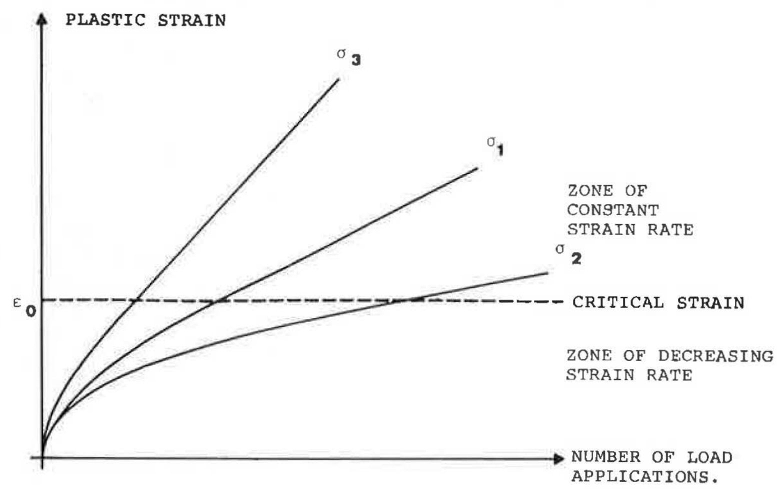
The objectives of the first tests in the RTM can be summarized as (a) the running in of the machinery, (b) qualitative and quantitative evaluation of pavement response, and (c) prediction of pavement performance.

The main parts of the machinery were brought into line during the first two years, but improvements are still being made, especially in the system for measuring and recording stresses, strains, and deflections.

The qualitative evaluation of pavement response confirmed that the RTM is well suited to simulating heavy traffic loadings. The variations of stresses and strains with variations in, for instance, wheel load, velocity, and tire pressure were the same as those measured in situ under heavy traffic. The performance in the RTM should also be compared with the performance of in situ pavements, but this has not yet been possible.

The quantitative evaluation of pavement response consists of (a) evaluation of the accuracy of the measured values and (b) comparison of measured and calculated values. To measure the critical vertical stress and strain in the top of the subgrade, four pressure cells and four strain cells were installed. The typical coefficient of variation for both measured quantities was about 0.25. Even larger deviations were encountered for lower levels; the two lowest pressure cells typically showed a difference of a factor of four. Most of the variations are likely to be caused by an inappropriate installation procedure, as indicated by the in situ cali-

Figure 13. Permanent strain model.



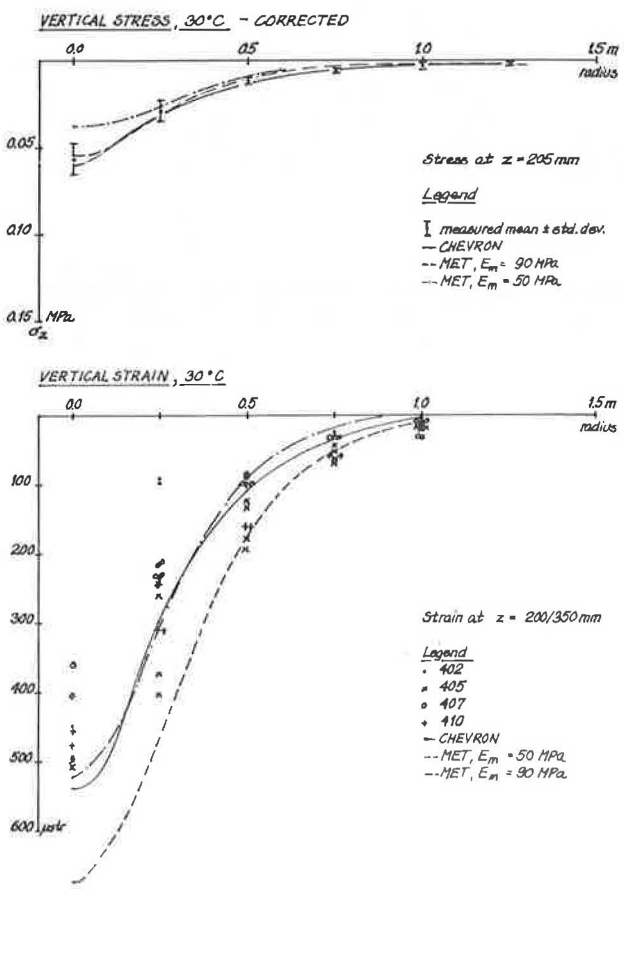
bration of the pressure cells, but part of the variation is definitely the result of varying materials and structural characteristics and thus cannot be avoided. Severe problems were encountered in attempting to measure the horizontal strain at the bottom of the asphalt layer, and this was only achieved in a few of the tests. For this quantity, a coefficient of variation of up to 0.8 was found.

These large variations in the measured values must be kept in mind in comparing measured and calculated quantities. Figure 14 shows the variation with distance from the center of the load of the measured vertical stress and strain at the top of the subgrade. The stress and strain were measured in FWD tests at a peak force of 30 kN and an asphalt temperature of 30°C. Superimposed on the measured values are calculated values found by using the Chevron program and the method of equivalent thicknesses. The agreement between the measured and calculated values is seen to be reasonably good. The variation of the measured vertical strain, however, indicates a horizontal variation of the subgrade modulus, which was not included in any of the theoretical methods.

Four kinds of performance prediction were attempted: (a) cracking of the asphalt layer, (b) excessive deformation of the subgrade, (c) amount of permanent deformation, and (d) PSI.

The two previously mentioned methods of crack prediction were used. For series 1, the prediction was based on calculated critical strains; for series 2, it was based on measured values plus and minus one standard deviation. There was some indication from the measured stresses and strains for test series 2 that crack initiation

Figure 14. Measured and calculated stress and strain values.



might have taken place in the asphalt, but no visible cracks were detected nor did FWD tests in any of the test series show any decrease in the asphalt modulus. It can be concluded that either both fatigue criteria are somewhat conservative or the assumption of a standard (weighted) asphalt temperature is too simple a model of the real fatigue performance.

Five different methods were used to test for excessive deformation of the subgrade. The only criterion that did not predict a fair amount of permanent subgrade deformation was that of Witczak (23). The actual permanent deformation of the subgrade was found to be 4 mm of a total pavement deformation of 11 mm.

The AASHO-based criteria were somewhat conservative, perhaps because most of the damage during the AASHO Road Test was caused by longitudinal roughness (85 percent) and the variation in permanent deformation was therefore more important than the average deformation. The first few tests, therefore, supported the criteria suggested by Witczak with respect to average permanent deformation of the subgrade. If, however, a riding comfort criterion is accepted, the variations in permanent deformation may be more relevant than the average deformation. In Figure 15, four of the predictions used are compared with the experimental PSI values. The agreement between observed and predicted life is seen to be better when a limiting value of PSI = 2.5 is used, but the scatter in the predicted values is considerable.

The amount of rutting during the first test series was calculated as previously described. For the asphalt, the following relation between mixture stiffness E and bitumen stiffness S_{bit} (both in megapascals) was used:

$$E = 63 \times S_{bit}^{0.31} \qquad (6)$$

This relation was obtained from three-point bending tests that were carried out at high temperature and low frequencies before the first test series. Two calculations were done, one using a softening point (ring and ball) of 57°C and a penetration index (PI) of 0 found in August 1973 before the first test series and another with a softening point of 65.5°C and a PI of 0.5 found in May 1976 after completion of the first test series. In Figure 16, the calculated rut depths are compared with the measured values. The agreement appears to be reasonably good. Part of the discrepancy in predicting the rutting caused by the 20- and 30-kN wheel loads results from the fact that the real load distributions are not circular as assumed in the calculations.

Finally, an attempt was made to predict the PSI value through a computer simulation of pavement performance. Parts of the input, such as the standard deviations of the materials and the structural characteristics, were not known and had to be estimated.

Ten computations were done. The range of calculated values is shown in Figure 17 superimposed on the experimental results. The predicted life is seen to be rather longer than the observed life. Cracking of the asphalt layer was not predicted in any of the simulations. A few additional simulations were carried out for yearly temperature variations between +20°C and -2°C and a spring thaw reduction of the subgrade modulus of 70 and 90 percent. In neither case was any cracking produced, nor did the PSI value decline to 2.5 within the first million axle passages (30-kN wheel load) applied during the first four years. This, at least, does not contradict the results of the freeze-thaw test.

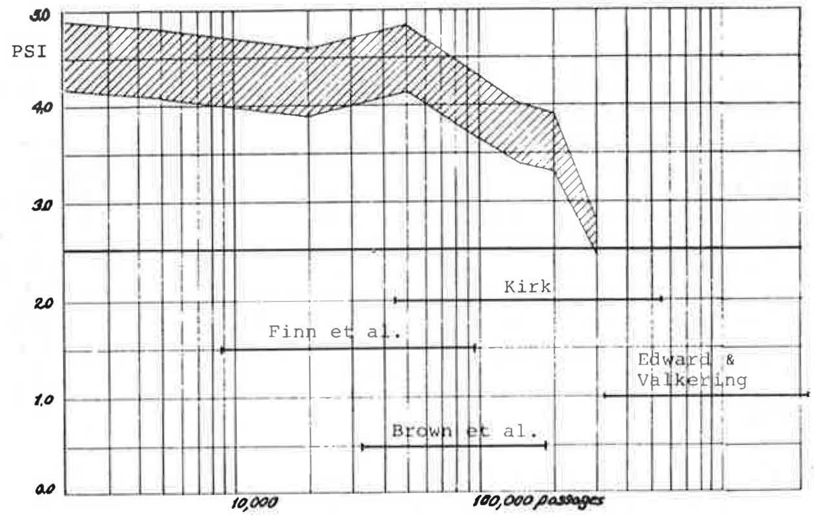


Figure 15. Subgrade criteria versus PSI values.

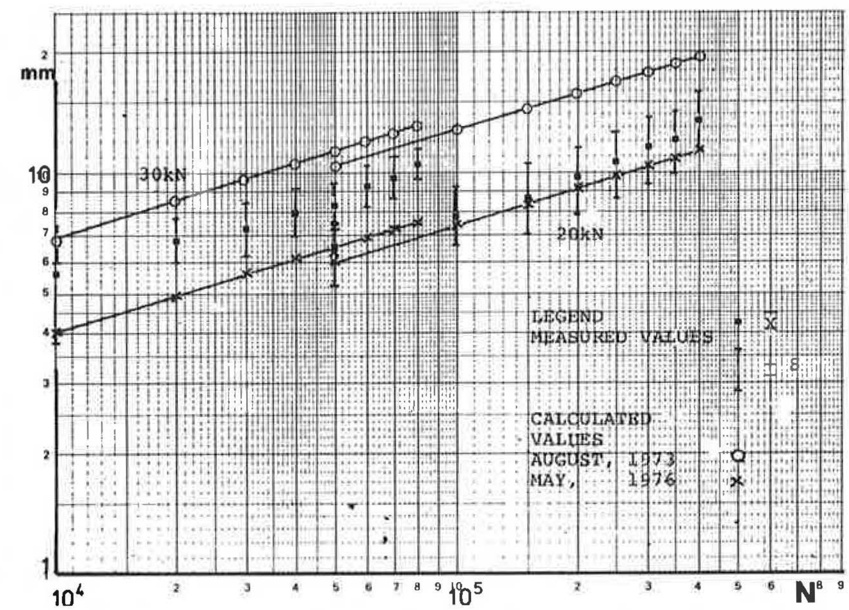


Figure 16. Computed rutting versus measured rutting.

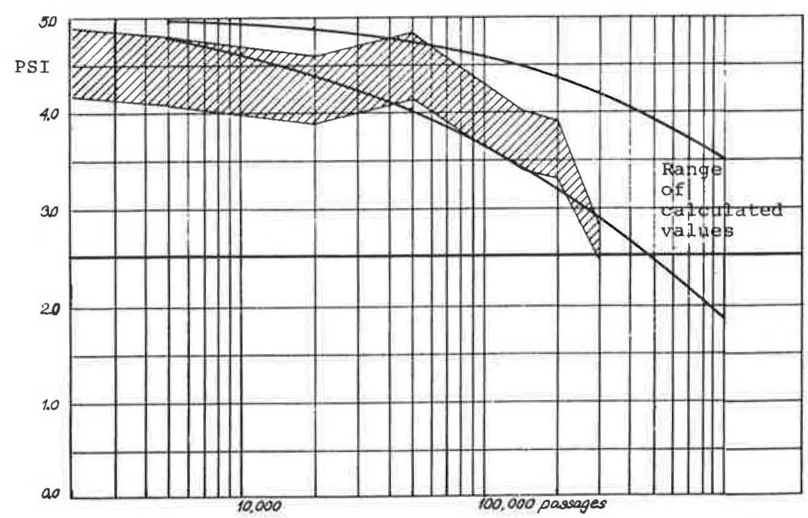


Figure 17. Computed PSI values versus measured PSI values.

CONCLUSIONS

The main conclusion of the first few test series is that the RTM appears to be well suited to simulating the effects of heavy traffic loadings and climatic variations on full-scale pavement structures. As a result, a five-year research program jointly sponsored by the National Danish Road Laboratory and the Technical University of Denmark has been initiated. In this research program, nonconventional pavement structures and materials will be tested under varying climatic conditions simultaneously with more traditional structures. It is hoped that these tests will also contribute to the development of a predictive design procedure that will be superior to the deterministic procedures currently being used.

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Utility Decision Model for Pavement Recycling

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A decision model developed by using utility theory to evaluate various techniques for recycling of pavement materials is described. The model is quantified by using subjective opinions of experienced engineers who are familiar with pavement rehabilitation. Limited objective field data