AC on 122 mm (4.8 in) of DGA. Temperature and moduli distributions and the associated mean pavement temperature and modulus were determined. The mean pavement temperature and modulus were used to determine the deflection factor needed to adjust the field deflections to reference conditions. Plots of temperature and AC modulus distributions are shown in Figure 16. The relationships between measured and projected deflections and subgrade moduli for both theory and field behavior are shown in Figure 17; the after-overlay test data shown in Figure 17b indicate a behavior equivalent to the effective structure plus the overlay thickness.

SUMMARY

A system for the rational design of an AC overlay has been presented in a step-by-step format. Evaluation of one of many test sites has been presented to illustrate the before-and-after conditions and the agreement between the test data and the theory.

ACKNOWLEDGMENT

The concepts, data, and analyses reported in this paper are a result in part of a research study on the development of a rational overlay design method for pavements that was conducted as a part of a program funded by the Federal Highway Administration and the Kentucky Department of Transportation. The contents of the report reflect our views; we are responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Kentucky Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

REFERENCES

1. W. B. Drake and J. H. Havens. Kentucky Flexible Pavement Design Studies. HRB, Bull. 233, 1959, pp. 33-66.
2. H. F. Southgate, R. C. Deen, J. H. Havens, and W. B. Drake. Kentucky Research: A Flexible Pavement Design and Management System. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977, pp. 269-297.
3. Interim Guide for Design of Pavement Structures. AASHO, 1972.
4. W. Heukelom and C. R. Foster. Dynamic Testing of Pavements. Proc., ASCE, Journal of the Soil Mechanics Division, Vol. 86, No. SM1, Feb. 1960.
5. H. F. Southgate, D. C. Newberry, Jr., R. C. Deen, and J. H. Havens. Resurfacing, Restoration, and Rehabilitation of Interstate Highways: Criteria and Logic Used to Determine January 3, 1977, Needs and Estimates of Costs. Division of Research, Kentucky Department of Transportation, Lexington, July 1977.
6. H. F. Southgate, G. W. Sharpe, and R. C. Deen. Case Histories of Pavement Evaluations Using Dynamic Deflections. Division of Research, Kentucky Department of Highways, Lexington, in preparation.
7. J. A. Deacon and R. C. Deen. Equivalent Axle Loads for Pavement Design. HRB, Highway Research Record 291, 1969, pp. 133-143.
8. R. L. Rizenbergs, J. L. Burchett, and L. E. Davis. Pavement Roughness: Measurement and Evaluation. Division of Research, Kentucky Department of Highways, Lexington, Dec. 1971.

Publication of this paper sponsored by Committee on Pavement Rehabilitation Design.

Overlay Design Based on Falling Weight Deflectometer Measurements

R. C. Koole, Koninklijke/Shell-Laboratorium, Amsterdam

The technique used for measuring deflections in an asphalt pavement by means of a falling weight deflectometer is described in some detail. Two models of the deflectometer that have different force ranges have been developed at Koninklijke/Shell-Laboratorium, Amsterdam. The deflectometer is used for the routine evaluation of pavements. The data it produces are of sufficient quantity and quality to serve as input for an analytical method of overlay design. The validity of the data and the interpretation method has been verified by wave-propagation measurements. The basic principles of the new Shell design method are outlined, with specific reference to the determination of overlay thicknesses. It is shown that the required thickness of an overlay depends on one of two criteria, subgrade strain and asphalt-fatigue strain, and that all designs must be checked to determine which of the two criteria is the limiting one. To illustrate this, several examples are given. Some possible refinements to the basic overlay design procedure are discussed, such as the incorporation of various mix characteristics and the procedure for use if the type of mix to be used for the overlay differs significantly from that of the existing pavement.

The economic growth of the 1950s and early 1960s was accompanied by rapid expansion of the existing road network in almost all of the countries of North America and western Europe. Many of the roads constructed at that time, however, are now nearing the end of their structural design lives and in need of major repair.

The structural strength of a pavement refers to its ability to limit strains to such an extent that, during its design life, virtually no cracking occurs in any part of the structure and there is no excessive permanent deformation in the subgrade.

Structural strength is not the only factor that determines the serviceability of a road. Skid resistance and rut depth, for example, are also important in determining the acceptability of a pavement as a riding surface. The recently published Shell Pavement Design Manual (1) specifically recognizes that rut depth due to permanent deformation of the asphalt (and the prediction thereof) is a separate criterion; this has been discussed in several other publications, for example, Van de Loo (2). This paper is concerned solely with structural aspects.

First, a method is discussed that enables the road engineer to determine, in situ, those factors from which the mechanical properties of an existing pavement can be determined. This method is based on deflections measured with an instrument known as a falling weight deflectometer (FWD).

Second, the way is discussed in which these mechanical properties can be used as a basis for a quantitative determination of the residual life of an existing pavement and of the strengthening measures (in terms of overlay thickness) that may be required for the desired future service life.

Most nondestructive techniques for testing flexible pavements are based on measurements of deflections of the pavement under a known load. Empirical techniques of interpretation derive overlay thicknesses more or less directly from the deflection amplitude. More analytical techniques use this amplitude to determine certain parameters significant for the design life of the pavement (e.g., moduli of elasticity of the component layers of the pavement) and then use these parameters in a design model to calculate the thickness of overlay required.

Falling weight deflectometers, which have been used at the Koninklijke/Shell-Laboratorium in Amsterdam over the past three years, have proved to be particularly suitable for the routine evaluation of pavements. At the same time, the information they yield about the mechanical properties of a pavement provides a sound basis for the calculation of the overlay thickness required, for example, by using the Shell design method.

The technique used for interpreting the FWD measurements has been validated on a number of pavements by wave-propagation measurements with the heavy road-vibration machine, the development of which began at Amsterdam some 35 years ago, and with the Goodmans vibrator.

Preferably, pavement properties determined by a pavement evaluation technique should be used in an analytical pavement model from which the required overlay thicknesses for a given future design life can be quantified.

The pavement model that provides the basis for the method described in the Shell Pavement Design Manual is a three-layer structure: an upper asphalt layer, a middle layer of either unbound or bound material, and a lower subgrade layer. Previous publications (3, 4) have discussed the details of the method and its presentation extensively. In this paper, therefore, only a brief outline is given of the pavement-design principles; the discussion is limited to the part concerned with overlay design. It is stressed that, in the three-layer design model, there are two criteria that may govern the design—subgrade strain and asphalt strain—and an overlay-thickness design must be checked for each criterion separately. To illustrate this point, several examples are given.

Asphalt-mix properties can differ widely; moreover, the properties of the mix to be used for an overlay are not necessarily those of the existing pavement. Thus, the design method includes a procedure by which allowance for differing mix characteristics can be incorporated in an overlay design.

MEASURING DEFLECTIONS WITH THE FALLING WEIGHT DEFLECTOMETER

The basic principle of the FWD, as described by Claessen (5), is that of a mass falling on a footplate that is connected to a baseplate by a set of springs (see Figure 1). The peak force (F) thus exerted on the pavement is

$$F = \sqrt{2Mghk} \tag{1}$$

where

M = mass of the falling weight (kg),
h = drop height (m), and
k = spring constant (N/m).

There are several methods of varying the magnitude of the maximum force.

1. Changing the mass of the falling weight: This is impractical for routine investigations where many measurements must be made as quickly as possible to obtain a meaningful impression of the pavement under investigation rapidly.
2. Changing the drop height: This is feasible for routine investigations if the design of the mechanical method of setting the drop height permits it.
3. Changing the spring constant: When mechanical springs are used, the only way to do this is to substitute a set of springs that have different characteristics (which is not normally feasible in the course of routine investigations).

Both changing the mass and changing the springs also affect the pulse width of the force. This means that, if a constant pulse width at different force levels is required and the method by which the force is changed is by substitution of a different mass, there must also be a change of springs.

It is obvious, therefore, that the most practical way to change the force level is to change the drop height.

It should be noted that Equation 1 assumes a linear spring constant, which is not correct for rubber springs. However, a linear spring constant can be assumed if only a small range of the spring characteristic is used (see Figure 2).

The Koninklijke/Shell-Laboratorium currently uses two automated FWDs. The first, shown in Figure 3, has been in use since 1975 and drops a mass of 150 kg from a height that can be varied from 0.04 to 0.40 m. The peak force exerted on the pavement can thus be varied from 15 to 48 kN, a force level representative of the actual wheel loads of most commercial vehicles.

The characteristic of the set of springs has been chosen in such a way that a pulse width of 0.028 s is produced. Numerous measurements on actual pavements have shown that this corresponds to the pulse width produced by commercial traffic traveling at approximately 60 to 70 km/h.

The second, shown in Figure 4, has been developed and constructed recently for use on heavy pavement structures, e.g., airfield pavements. It drops a mass of 407 kg from a height set between 0.04 and 0.40 m and exerts a force that varies between 40 and 125 kN at the same pulse width of 0.028 s. To improve its versatility, this FWD also has a 240-kg mass that is kept in reserve and, together with a set of springs with modified characteristics, covers a peak-force range of 23-90 kN, again at the 28-s pulse width.

The same measuring technique (and interpretation of results) is used for both FWDs. The effect of the force exerted is to deflect the pavement under and around the area of loading. The deflection in the center of the area is a function of the properties and dimensions of the pavement structure but, as is illustrated in Figure 5, this is not sufficient for an exact interpretation because pavements that have entirely different deflection bowls and thus entirely different pavement properties can very well show the same deflection in the center of the area of loading. Therefore, in addition to the deflection at the center of the area of loading (δ_0) the deflection at at least one other point must be measured. This point can be chosen arbitrarily but in routine investigations is usually fixed at 0.3, 0.6, 1.0, or 2.0 m, depending on the type of pavement structure.

The interpretation of the measurements requires two deflection values: the deflection at the center of the area of loading and a deflection value approximately half this. The distance (r) from the center of the loading area at which this latter value is found must also be known. A sensitivity analysis has shown that the interpretation technique yields the most accurate results on the basis of these two deflections.

The deflections are measured by velocity transducers (geophones). The transducers use the inertia of a mass; because their original (predeflection) position serves as reference, they do not require any rigid support from a base outside the deflection bowl.

The deflection signals are projected on a screen in the instrumentation van, where they are evaluated by an operator for acceptability before being printed on a continuous paper sheet or stored on magnetic tape for later

Figure 1. Schematic representation of falling weight deflectometer.

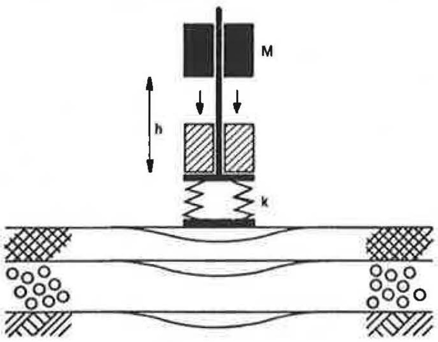


Figure 2. Spring characteristic: rubber springs of second falling weight deflectometer.

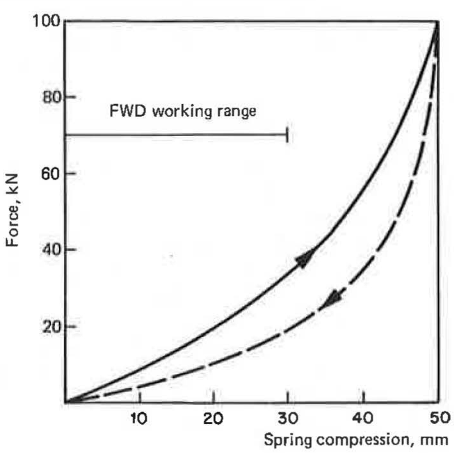


Figure 3. First falling weight deflectometer.

Figure 4. Second falling weight deflectometer.

Figure 5. Schematic representation of pavement deflection.

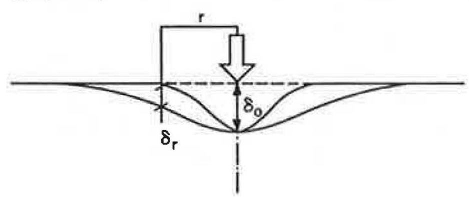
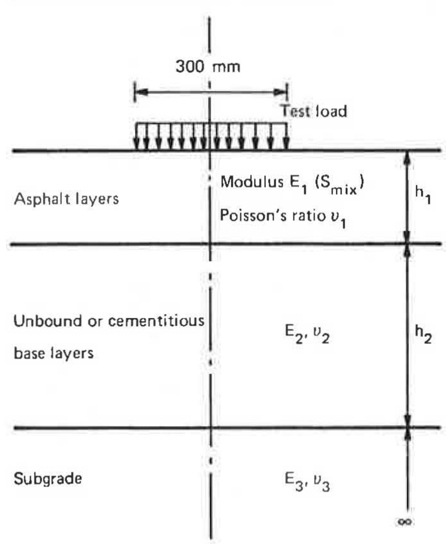


Figure 6. Schematic representation of a pavement structure under a test load.



automated processing (or both).

Every deflection measurement is accompanied by an indication of the location of the measuring point. The force level, pulse width, and asphalt temperature are checked at intervals and recorded.

The force level and pulse width are measured by an accelerometer that can be attached to the falling mass. The accelerometer registers the deceleration of the mass after it has hit the footplate.

The temperature of the asphalt pavement under investigation is measured at regular intervals in the course of the day. This is done by taking a temperature reading of a spike that is shot into the asphalt to a specific depth by using a special gun developed for building practice. Experiments have shown that the heat generated by insertion of the spike dissipates in less than one minute, which makes this method of measuring asphalt temperature practicable for routine use. The unsatisfactory method of estimating the asphalt-layer temperature from a measured surface temperature is thus avoided.

The temperature measurements are used for estimating the modulus of elasticity (E) of the asphalt pavement from known mix characteristics; normally, the values obtained by this route are accurate enough. If, however, a higher degree of accuracy is required, the E-modulus of the asphalt pavement can be determined by high-frequency (80-3000 Hz) wave-propagation measurements. For this purpose, both FWD carriers also contain Goodmans vibrator equipment. However, this procedure is rather time consuming and it is therefore not used unless the higher accuracy is specifically required.

INTERPRETATION OF FALLING WEIGHT DEFLECTOMETER DATA

The pavement structure is schematized as a three-layer model, as shown in Figure 6. The top layer represents the asphalt layer, the middle layer represents the base materials, be they granular (unbound) or cementitious (bound), and the third layer, taken as being of infinite dimensions, represents the subgrade or original soil.

The materials of which the separate layers consist are assumed to behave in a linear elastic way; this has proved an acceptable assumption for the (short) loading times in question. The layers are further characterized by the following properties:

1. For the asphalt layer, an E-value (E_1 or S_{mix}), a Poisson's ratio (ν_1), and a layer thickness (h_1);
2. For the base layer, an E-value (E_2), a Poisson's ratio (ν_2), and a layer thickness (h_2); and
3. For the subgrade layer, an E-value (E_3), and a Poisson's ratio (ν_3) (the layer thickness is taken as infinite).

If these values are known, the stresses and strains, and thus the shape of the deflection bowl of a pavement under a given load (6), can be calculated, for example by using the BISAR computer program (7, 8).

In the interpretation of the FWD measurements, some of these values are assumed or estimated as closely as possible from coring or from existing records (e.g., construction reports). The Poisson's ratios and the layer thicknesses of the base layers are assumed because small variations in these values have little effect. The E-moduli of cementitious base layers can be derived from past experience or measurements. Actually, the unbound base layers will show an increasing modulus from the subgrade up. This range of moduli can be replaced by an effective modulus of the total unbound base layer that is a function of the subgrade modulus E_3:

$$E_2 = 0.2 \times h_2^{0.45} \times E_3 \quad (2)$$

where h_2 is in millimeters and is subject to the limits $(2 \leq 0.2 \times h_2^{0.45} \leq 4)$. This effective modulus can only be used for calculation of stresses and strains in layers other than the unbound base layer itself.

In the normal interpretation procedure, the E-modulus of the asphalt layer is determined from the stiffness (E_1) modulus of the asphalt mix and the temperature of the asphalt during the FWD measurements, provided that the type of mix used is known or can be estimated very closely [for example, by using a nomograph given in the Shell design manual (1)]. Determining the E-modulus of the asphalt layer by wave-propagation measurements is recommended only in cases where the greater accuracy is specifically required.

The remaining two pavement properties—the thickness of the asphalt layer and the subgrade modulus—can be calculated from the measured values of δ_0/F and δ_r/F (where F is in Newtons and δ_0 and δ_r are in 10^{-10} meters per Newton). The value of the asphalt-layer thickness thus calculated is called the effective asphalt-layer thickness ($h_{1\,eff}$) because it incorporates and compensates for errors in the estimation of the stiffness modulus of the asphalt layer and/or the deterioration of the asphalt layer.

Another feasible procedure would be to use the actual asphalt-layer thickness (from cores or old records) and calculate the effective E-modulus of the asphalt layer. The two procedures do not differ significantly.

It is not possible to determine the residual life of a pavement solely from deflection measurements. The reason for this is shown in Figure 7. The change in E-modulus as the number of load repetitions increases has been observed in laboratory fatigue tests and is corroborated by deflection measurements in practice. Deflection values are almost constant over a long period; however, when the pavement approaches the end of its design life, the deflections increase quite sharply. It is, however, possible to determine the original design life, in terms of the number of repeated applications of a standard axle load. This is the reason that the standard FWD practice is to make the measurements at points where traffic loading is slight (such as between the wheel tracks).

Occasionally, a check is made by measuring the deflections in one of the wheel tracks. If the deflection values measured in the wheel track are significantly larger than those measured between the wheel tracks, this is a definite indication that the pavement is approaching the end of its service life.

For a length of road pavement, the average values of δ_0 and δ_r/δ_0 (Q_r) are calculated (see Figure 8), together with their standard deviations. Next, the 85th percentile value of δ_0 and the 55th percentile value of Q_r are calculated. Then, these and the known value of E_1 are compared with a series of interpretation graphs (of which Figure 9 is an example) to determine the values of E_3 and h_1. The value of h_1 obtained in this way should be interpreted as the effective asphalt-layer thickness ($h_{1\,eff}$).

Formerly, the interpretations of all possible combinations of 15th and 85th percentile values of δ_0 and Q_r were checked in terms of the corresponding required overlay thickness. However, experience has shown that, in nearly all structures, the 15th percentile value of δ_0 in combination with the 85th percentile value of Q_r leads to a safe overlay design.

Interpretation graphs such as that shown in Figure 9 are based on the results of several BISAR computer program calculations.

VALIDATION AND EXAMPLES

There have been several experiments performed with the FWD to check the validity of the results obtained by using the method of interpretation described above. Some of the results are shown in Figures 10 and 11, and further confirmation is provided by the results of a recent experiment (see Table 1). Constructions 1-4 were different sections of the same road, constructions 5 and 6 were located in a second road, and constructions 7 and 8 in a third, all situated relatively close to each other in one municipality. The results of the FWD measurements performed on these pavements are shown in Figure 12.

For constructions 1-6, the deflections measured at a distance r = 1 m from the center of the load were used in the interpretation. In the case of constructions 7 and 8, the deflection bowl was wide and shallow because of the thick layer of hydraulically bound slag and the deflection at r = 2 m was used.

In these interpretations, all the base materials were assumed to be unbound material to keep the interpretation technique simple. The particular properties of any bound base materials manifested themselves in terms of an additional asphalt-layer thickness over and above the actual asphalt-layer thickness. This is obvious for constructions 1 and 4 and even more so for constructions 7 and 8. Also, the difference in effective thickness of constructions 7 and 8 can only partly be explained by the difference in actual asphalt thickness, which means that the slag in the base layers of construction 7 must have reached a higher degree of hydraulic binding. Thus, it may be concluded that there is generally good conformity between the actual pavement thicknesses and the derived effective asphalt-layer thicknesses.

As an additional check, low-frequency (<80 Hz) wave-

Figure 7. Relationship between E-modulus and number of load repetitions.

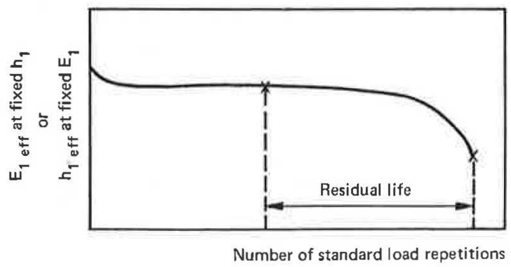


Figure 8. Graphical representation of δ_0 and Q_r.

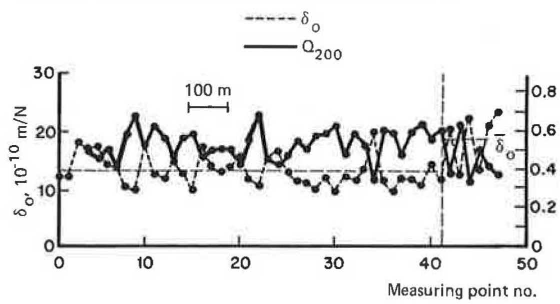


Figure 9. Deflection interpretation chart.

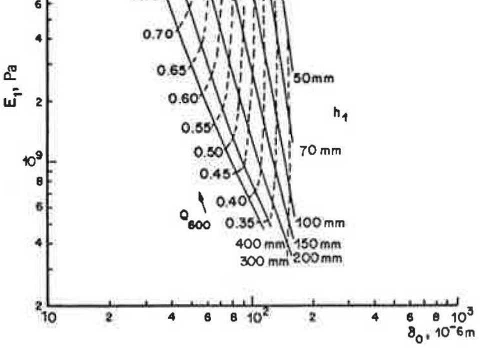


Figure 10. Comparison between subgrade modulus obtained from sinusoidal-wave-propagation measurements (E_{3SW}) and that obtained by FWD (E_{3FWD}) measurements.

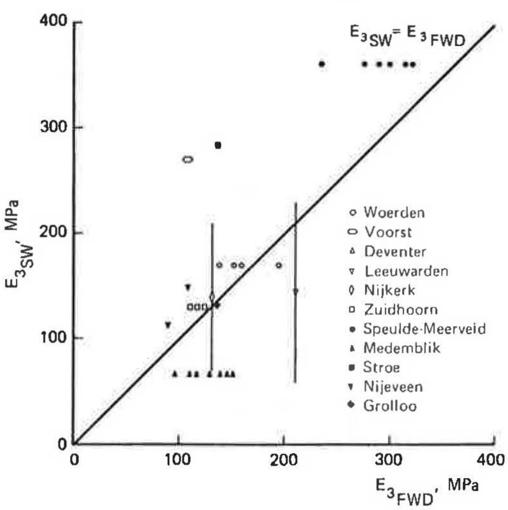


Figure 11. Comparison between asphalt layer thickness determined from FWD deflections ($h_{1\ FWD}$) and actual layer thickness ($h_{1\ actual}$).

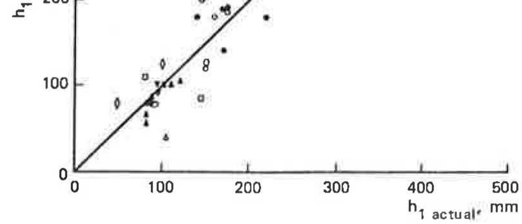


Figure 12. Deflection measurements made with the falling weight deflectometer: constructions (I-VIII).

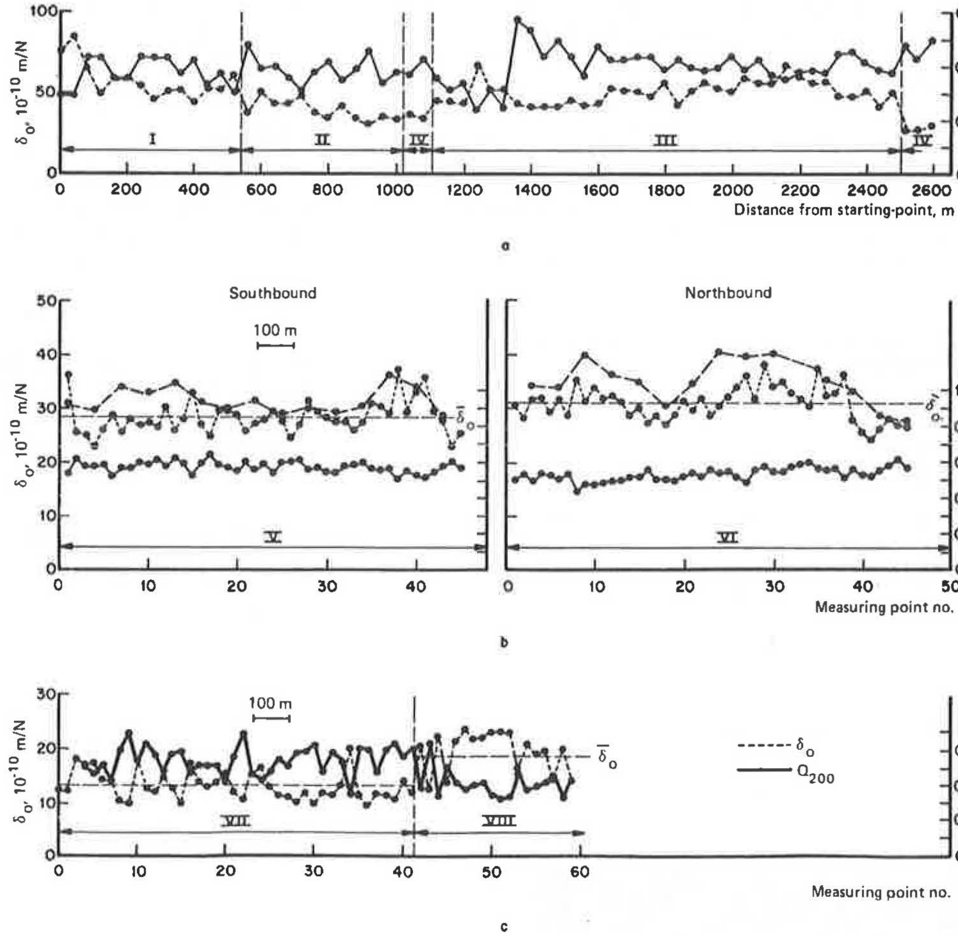


Table 1. Confirmation of validity of interpretation method.

Item	Construction 1	2	3	4	5	6	7	8
Thickness of bitumen-bound material (cm)	7	14	14	14	24	24	15	11
Road base material	~5 cm of old pavement (cracked) on 20 cm of crushed gravel	10 cm of blast furnace slag	20 cm of crushed gravel	Sand cement			5 cm of crushed gravel and 30 cm of blast-furnace slag on 50 cm of sand	5 cm of crushed gravel and 30 cm of blast-furnace slag on 50 cm of sand
Subgrade	Sand	Sand	Sand	Sand	Sand	Sand	Clay and sand	Clay and sand
$h_{1\ eff}$ (FWD) (cm)	12	15	13	17.5	27.5	25	65	35
E_3 (MPa)								
FWD	120	190	150	190	163	182	194	200
RVM							150	150

propagation measurements were performed on constructions 7 and 8 by using the road vibration machine (RVM) to determine the value of E_3. The values obtained in this way (150 MPa) correspond fairly well to those obtained by using the FWD technique. [The somewhat lower value might be explained by the fact that the stresses induced by the RVM in the subgrade are lower (by about 50 percent) than those induced by the FWD.]

SHELL DESIGN CHARTS

The Shell Pavement Design Manual published in 1978 includes a large number of thickness-design charts that can be used in the design of both new pavements and overlays. The design model on which the charts are based (and consequently the model from which the overlay thickness can be calculated) is the three-layer model described above.

The failure criteria that govern the design are threefold (see also Figure 13):

1. The compressive strain at the surface of the subgrade: If this strain is excessive, permanent deformation will eventually occur at the top of the subgrade, re-

sulting in permanent deformation at the pavement surface as well.

2. The tensile strain in the asphalt layer: The maximum tensile strain generally occurs at the bottom of the asphalt layer; if this strain is excessive, cracking of the asphalt layer will occur.

3. The permanent deformation within the asphalt layer: This may lead to rutting to such an extent that the acceptability of the pavement as a riding surface is impaired.

Figure 13. Pavement design model.

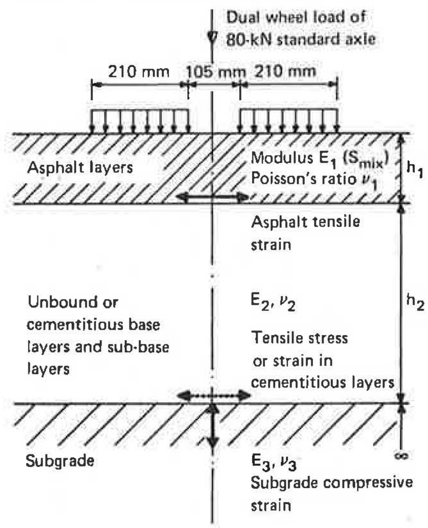
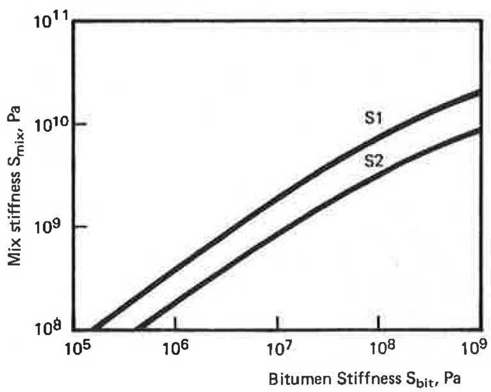


Figure 14. Standard stiffness characteristics.



One of the basic principles of the pavement-design theory on which the manual is based is that the structural design life of a pavement is dependent on either the subgrade-strain criterion or the asphalt-strain criterion.

In determining the thickness required for an overlay, the subgrade-strain and the asphalt-strain criteria should each be considered separately; it is quite possible that the design criterion that did not govern the original pavement design will become limiting for the overlay thickness. Consider, for example, a structure originally governed by the subgrade-strain criterion. If, after a certain proportion of the service life has been consumed, an overlay is applied, the permanent deformation due to both asphalt and subgrade deformation will automatically be eliminated and the original design life will be restored. However, the maximum asphalt strain must also be taken into account. In the original pavement, this occurred at the bottom of the asphalt layer. After the overlay has been applied, it will still occur in the same place, but its magnitude will be less because the pavement is thicker.

The level of this asphalt strain must be low enough to reduce the rate of consumption of the residual fatigue life so that the original pavement will last the future design life without cracking. This can be calculated by using a fictitious design life in interpreting the charts.

The design manual contains close to 300 design charts that were developed on the basis of the three-layer model of the pavement and incorporate the criteria of subgrade strain and asphalt strain. The permanent-deformation criterion is not incorporated in the charts and must be dealt with by a separate procedure.

In the design procedure, certain (standard) asphalt-mix types are used. The mix stiffness (E_1 or S_{mix}) is characterized in relation to bitumen stiffness by the curves S1 and S2 as shown in Figure 14, and the fatigue behavior is standardized as F1 and F2 as shown in Figure 15. Two standard grades of bitumen—hardened 50 and 100 pen—are used; their properties are given below.

Bitumen	$T_{800\ pen}$ (°C)	Pen_{25} (0.1 mm)	Penetration Index
50 pen	59	35	0
100 pen	53	60	0

Thus, charts are generally given for eight different standard mix codes (all possible combinations of S1 or S2 with F1 or F2 and 50 or 100 pen bitumen); for example, a mix that has good stiffness behavior (an E-modulus of S1) and good fatigue behavior (F1) made with 45-60 pen bitumen would be represented by the mix code S1-F1-50.

Other input parameters for design are subgrade mod-

Figure 15. Standard fatigue characteristics.

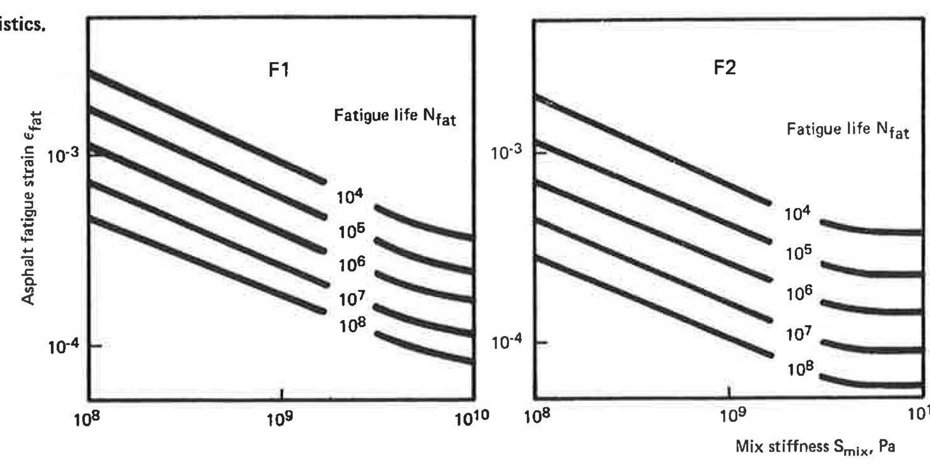


Figure 16. Axle-load conversion.

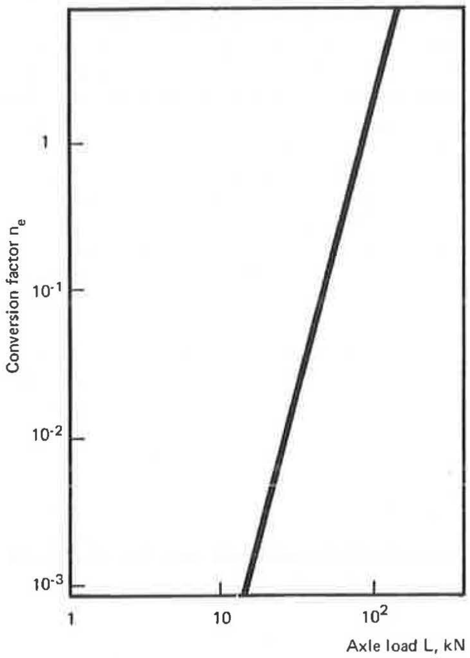
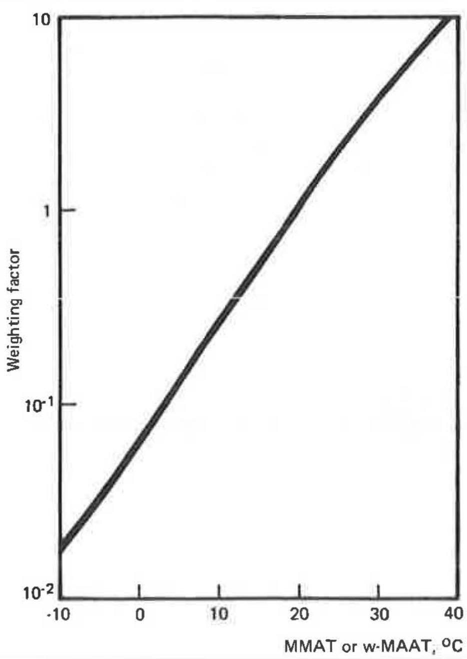


Figure 17. Chart W: temperature-weighting curve.



ulus, design life, and climate.

The design life (N) is expressed as the number of repetitions of a standard axle load of 80 kN. Any given axle spectrum can be converted into an equivalent number of 80-kN standard axle loads by using the graph illustrated in Figure 16. Based on 80 kN = 1.00, this curve represents the relative damage done to the pavement by an axle load different from 80 kN.

The climate is introduced as a weighted mean annual air temperature (w-MAAT) that can be calculated from the mean monthly air temperatures (MMATs) by use of the weighting-factor curve shown in Figure 17 (chart W of the design manual). This weighting curve was obtained by using BISAR calculations for a selection of representative multilayer pavement structures. The asphalt layers of these structures were further subdivided to account for the stiffness gradient that results from a temperature gradient in an otherwise homogeneous asphalt layer. The term "weighted" thus indicates that the daily and monthly temperature gradients in the asphalt layer in a certain climate have been taken into account. For example, by using the data given in Table 2 and Figure 17, the w-MAAT for Washington, D.C., is found to be $17.5 \approx 18°C$.

Table 2. Determination of w-MAAT.

Month	MMAT, °C	Weighting Factor	Month	MMAT, °C	Weighting Factor
January	1.4	0.08	September	20.3	1.1
February	2.2	0.09	October	14.2	0.50
March	6.7	0.18	November	8.1	0.20
April	12.2	0.38	December	2.8	0.10
May	18.1	0.82	Total		8.85
June	22.8	1.5			
July	25.3	2.1	Avg		0.74
August	23.9	1.8			

There are four types of design charts (see Figure 18).

1. Type HN (see Figure 18a): This type of chart shows the relationship between required asphalt thickness and the required thickness of the unbound base layers for different design lives, expressed as N. There are 128 design charts of the type HN in the manual, covering w-MAAT values of 4°C, 12°C, 20°C, and 28°C and subgrade moduli of 25-200 MPa.
2. Type HT (see Figure 18b): This type of chart most clearly illustrates the effect of climate on design. In general, a warm climate requires a thicker asphalt layer. However, this is not always the case; for example, when the asphalt strain is the governing criterion, the larger permissible asphalt strain of the warmer asphalt (see also Figure 15) may reverse this effect. There are 72 charts of the type HT in the manual.
3. Type TN (see Figure 18c): This type of chart shows the effect of climate in a different way, making possible, for example, the extrapolation of an empirically proven construction to a different climate. There are 48 charts of the type TN in the manual.
4. Type EN (see Figure 18d): This type of chart is useful not only for extrapolating asphalt thicknesses to areas that have different subgrade moduli, but also for the determination of overlay thicknesses. There are 48 charts of the type EN in the manual.

In principle, all four types of chart can be used for both the determination of overlay thicknesses and the design of new pavement structures. The presentation of a given type of information in different ways means that, whatever the form in which a problem manifests itself, the designer should be able to find the answer in a reasonably direct manner.

USE OF FALLING WEIGHT DEFLECTOMETER DATA WITH THE SHELL DESIGN CHARTS

The charts most frequently used for overlay design are those of the type EN, which are very suitable for the purpose. Because the number of charts had to be limited for practical reasons, there are EN charts for unbound-base-layer thicknesses of 0 and 300 mm only, for climates that have w-MAATs of 12°C, 20°C, and 28°C only. For other thicknesses of unbound-base layers and other

Figure 18. Example design charts: (a) type HN, (b) type HT, (c) type TN, and (d) type EN.

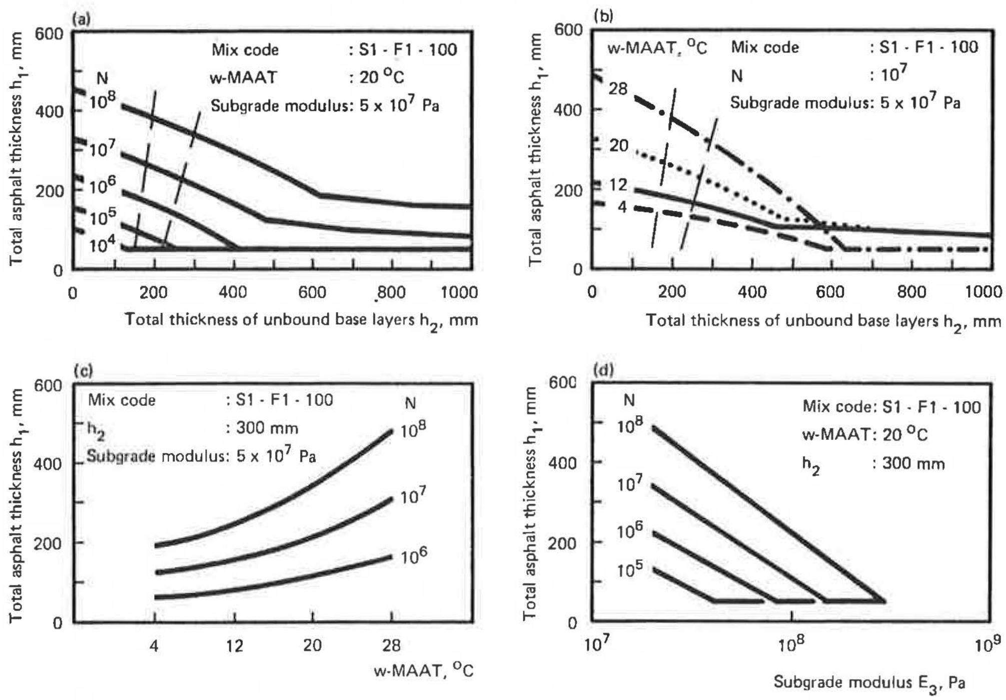
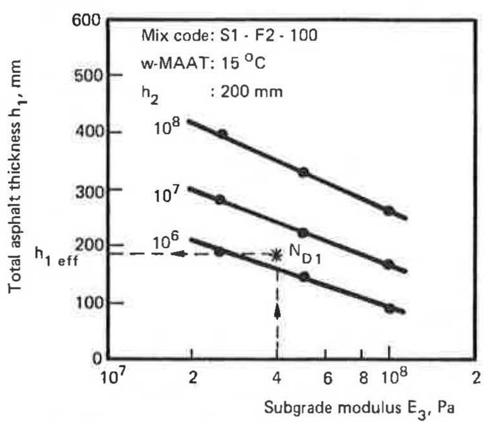


Figure 19. Determination of original design life.



climates, therefore, charts of the type EN must be constructed by interpolation of values read from charts of the type HN, HT, or TN. (The manual makes provision for this by supplying blank EN charts in the form of EN chart grids on a transparency.)

If the value of h_2 is known or can be assumed and the values of E_3 and $h_{1\,eff}$ of the pavement can be established, for example, from interpretation of the FWD measurements, the original design life of the pavement can be determined from a chart of the type EN.

This is illustrated in Figure 19 for a climate that has a w-MAAT of 15°C (which means that interpolation is necessary). If the mix code of the asphalt of the existing pavement is S1-F2-100, h_2 = 200 mm, the calculated E_3 = 40 MPa, and the calculated $h_{1\,eff}$ = 190 mm, then the original design life of that pavement must have been 2 000 000 passes of an 80-kN standard axle load. The number of standard axle passes to date can be calculated from data on the traffic intensity and axle-load distribution by using the weighting factors shown in Figure 16. The residual life of the existing pavement can then be established, and it can be determined whether this residual life suffices or whether an overlay is needed to produce the required future service life.

If an overlay is required, three separate calculation procedures are required:

1. One for an overlay-thickness design based on the subgrade-strain criterion;
2. One for an overlay-thickness design based on the asphalt-fatigue-strain criterion, taking into account the design life already consumed by the traffic to date; and
3. One to check that the thicknesses derived by the first two calculations do not exceed the thickness required when the existing pavement is taken as having deteriorated to such an extent that it must be regarded as an addition to the unbound-base layers only.

If the "check" thickness is found to be less than the larger one resulting from the first two procedures, it should be used because it is the most economical one, while still being adequate from the point of view of structural strength. If it is not less, the larger thickness from the first two procedures should be used.

Because subgrade strain manifests itself as a permanent deformation that is automatically eliminated by any overlay thickness, no allowance need be made for traffic passed to date and the original design life (from the point of view of subgrade strain) is restored and even increased. But if, in the existing pavement, the asphalt strain was not critical, it is possible that, under certain combinations of design parameters, it may become critical in the overlaid structure. The reason for this is that the maximum asphalt strain will occur at the underside of the existing asphalt pavement, both in the original and in the overlaid structure. In both cases, allowance should be made for the traffic passed to date, because this has consumed part of the asphalt fatigue life of the existing pavement.

The way that the consumption of asphalt fatigue life is accounted for is by substitution of a design number (N_{D2}) for the actual number of standard axle-load repetitions (N_{A2}) to be expected in the required future design period. The relationship between N_{D2} and N_{A2} can be derived as follows: If the original total design life (on the basis of asphalt strain) is N_{D1} and the number of standard axle-load repetitions that has occurred to date is N_{A1}, the relative consumption of life is N_{A1}/N_{D1}. The relative residual fatigue life is then $1 - N_{A1}/N_{D1}$.

This relative residual fatigue life can be consumed by the number of future standard axle-load repetitions (N_{A2}). If N_{A2} exceeds the absolute residual fatigue life ($N_{D1} - N_{A1}$), it is clear that an overlay is necessary. However, in the overlaid structure, the maximum asphalt strain will occur in the same place as in the existing structure (i.e., at the underside of the asphalt layer). If N_{A2} were used directly in the charts, the asphalt thickness obtained would be that for a new pavement, as though the existing pavement had not suffered any damage.

For the purposes of overlay design, therefore, the notional design number (N_{D2}) is introduced. This design number can be regarded as a fictitious number of expected future standard-axle-load repetitions that incorporates an allowance for the proportion of the fatigue life of the existing pavement that has already been consumed.

The value of N_{D2} is derived from the known data on the basis that the relative consumption of fatigue life by N_{A2}, expressed as N_{A2}/N_{D2}, is equal to the relative residual fatigue life ($1 - N_{A1}/N_{D1}$). Thus,

$$N_{D2} = (N_{D1} \times N_{A2})/(N_{D1} - N_{A1}) = N_{D1} \times N_{A2}/N_R \qquad (3)$$

Subsequently, N_{D2} is handled in the same way as N. Once the maximum, required, total future asphalt thickness is known, the overlay thickness can be obtained simply by subtracting the existing asphalt-layer thickness therefrom.

The check calculation should be made to ensure that the overlay thickness thus obtained not only enables the pavement to carry the future traffic but also involves no unnecessary application of asphalt. This situation could arise in an overlay design governed by the asphalt-strain criterion in which the existing pavement is quite close to the end of its fatigue life, i.e., N_{A1} is approaching N_{D1}. In this case, the overlay thickness necessary to limit strains occurring at the underside of the existing pavement to such a degree that it will last out the future number of load repetitions may be excessively large.

EXAMPLES OF OVERLAY DESIGN

The procedure described above can best be explained by examples.

Example 1

Assume that FWD or wave-propagation measurements have shown that an existing pavement has an E_3 of 60 MPa and an $h_{1\,eff}$ of 250 mm and that investigation of a core taken from the pavement indicates that the mix code can be designated as S1-F2-100, and that this is also the code for the type of mix to be used for the overlay. The climate of the location can be represented by a w-MAAT of 18°C. Old records indicate an h_2 of approximately 200 mm.

Because the design manual gives specific charts of the type EN for w-MAATs of 4°C, 12°C, 20°C, and 28°C only, it will be necessary to develop a chart of the type EN for a w-MAAT of 18°C by interpolation of data given in charts of the type HT at a w-MAAT of 18°C. This should be done for the two design criteria (subgrade strain and asphalt strain) separately for various design lives in terms of a number of standard axle passes (see Figure 20). Figure 20a shows the interpolated design chart that gives the required asphalt thicknesses based on the subgrade-strain criterion, and Figure 20b shows the design chart based on the asphalt-strain criterion.

Inserting the FWD results (E_3 = 60 MPa and $h_{1\,eff}$ = 250 mm) into Figure 20a gives point A, which indicates that the original design life based on the subgrade-strain criterion of the pavement (N_{D1s}) was 18 000 000 standard 80-kN axle loads. It can be calculated from traffic data that N_{A1} is 15 000 000. This means that the residual life in terms of standard axle passes (N_{Rs}) based on the subgrade-strain criterion will be 18 000 000 - 15 000 000 = 3 000 000.

Let it be assumed in this example that the road authority requires an overlay that will make the road pavement last for another 30 000 000 standard axle loads (i.e., N_{A2} = 30 000 000). It is obvious that the residual life of the existing pavement is insufficient. Point B in Figure 20a shows that a total asphalt thickness of h_1 = 290 mm will be required. Because there is already an $h_{1\,eff}$ of 250 mm and any overlay will automatically eliminate all ill effects of the old surface, an overlay thickness (h_0) of 290 - 250 = 40 mm will be sufficient from the point of view of the subgrade-strain criterion.

In the old construction, the maximum asphalt strain occurred at the bottom of the asphalt layer, and this will still be the case after the overlay has been applied. It must now be checked whether this asphalt strain will become the governing criterion during the future life of the pavement. For this purpose, the same FWD results are used in Figure 20b. This gives point C, the original asphalt fatigue life (N_{D1a}) of 30 000 000 standard axle loads, which is thus not the governing criterion. Therefore, the residual asphalt fatigue life (N_{Ra}) will be 30 000 000 - 15 000 000 = 15 000 000 standard axle loads, which is insufficient for the future design period.

Thus, a fictitious number of standard axle loads for the future design period (N_{D2}) must now be calculated by using Equation 3: $N_{D2} = N_{D1a} \times N_{A2} \div N_{Ra}$ = 30 000 000 × 30 000 000 ÷ 15 000 000 = 60 000 000 standard axle loads. The asphalt thickness required for a design number of 60 000 000 can then be determined from Figure 20b (point D) and is found to be 280 mm. Thus, the required overlay thickness from the point of view of the asphalt-strain criterion (asphalt fatigue) is 280 - 250 = 30 mm, which is less than that required on the basis of subgrade strain. This leads to the conclusion that the subgrade strain is the governing criterion and that the required overlay thickness is 40 mm.

Because the subgrade strain remains decisive, it is not necessary to check the approach in which the old asphalt layer is regarded as having deteriorated to the extent that it can be taken as part of the unbound base. Had this check been made by using, for example, an HT chart, a required (overlay) thickness of more than 200 mm would have been found, proving that the old pavement has a significant residual value. The overlay thickness required thus remains 40 mm.

Example 2

Assume that deflection measurements have shown that an existing pavement has an E_3 of 70 MPa and an $h_{1\,eff}$ of 200 mm. This time the mix code of the existing pavement and the intended overlay can be designated as S1-F2-50, and h_2 is again 200 mm. The climate is represented by a w-MAAT of 18°C. By interpolation from HT or HN charts, charts of the type EN can be developed as shown in Figure 21. Figure 21a shows the required asphalt thicknesses based on subgrade strain, and Figure 21b shows those based on asphalt strain.

Inserting the FWD data (E_3 = 70 MPa and $h_{1\,eff}$ = 200 mm) in Figure 21a gives a value of N_{D1s} of 30 000 000 standard axle loads. If the traffic to date is again assumed to be 15 000 000 standard axle passes, the re-

Figure 20. Interpolated design chart: example 1—(a) based on subgrade-strain criterion and (b) based on asphalt-strain criterion.

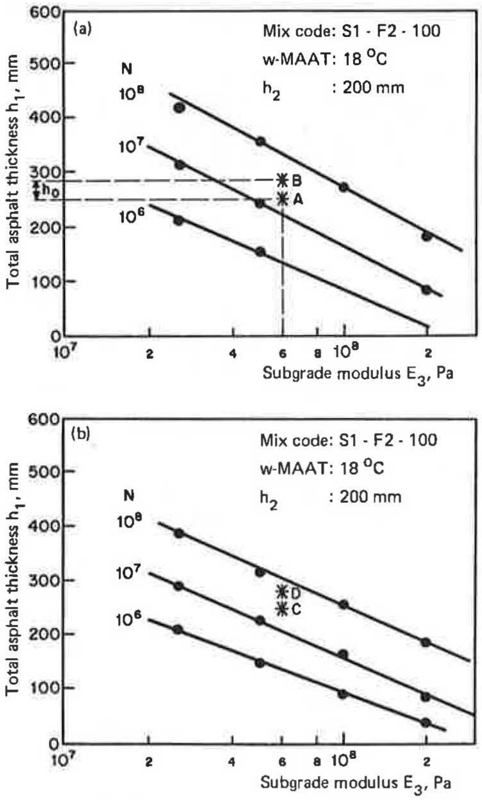


Figure 21. Interpolated design chart: examples 2 and 3—(a) based on subgrade-strain criterion and (b) based on asphalt-strain criterion.

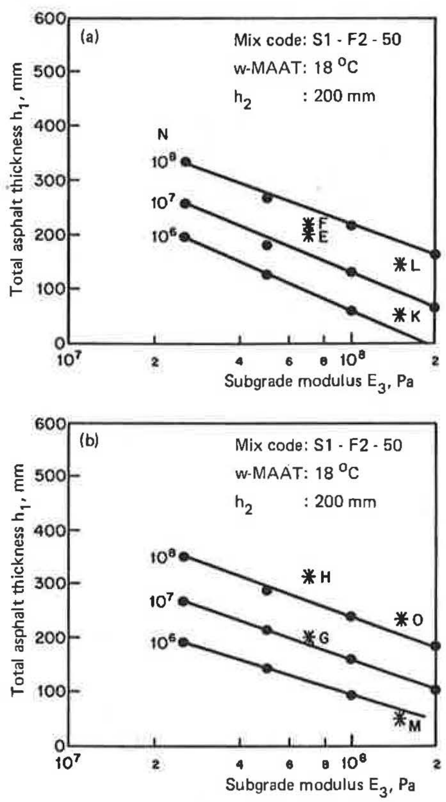
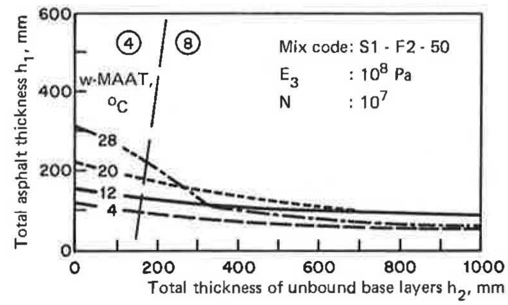


Figure 22. Calculation of overlay thickness by using chart HT 58 and assuming complete failure of existing pavement: example 2.



sidual life (N_{Rs}) is 15 000 000, which, for a required future life (N_{A2}) of 40 000 000 standard axle passes, gives a required total asphalt thickness of 210 mm (point F) and hence a required overlay thickness of 210 - 200 = 10 mm, from the point of view of the subgrade-strain criterion.

Inserting the same FWD data in Figure 21b (point G) gives a value of N_{A1a} of 17 000 000 standard axle passes, which thus is not decisive in the design of the original pavement. Therefore, N_{Ra} = 17 000 000 - 15 000 000 = 2 000 000 standard axle loads, and $N_{D2} = N_{D1a} \times N_{A2} \div N_{Ra}$ = 17 000 000 × 40 000 000 ÷ 2 000 000 = 340 000 000 standard axle loads. The required total asphalt thickness for the design number N_{D2} = 340 000 000 can be determined from point H in Figure 21a and is 320 mm. The required overlay thickness from the point of view of the asphalt strain is thus 320 - 200 = 120 mm. This means that the asphalt strain becomes decisive for the overlaid structure, even though this was not the case in the original pavement.

This time the checking procedure must be carried out to establish whether it would be more economical to consider the old pavement as having deteriorated to the point of failure and to regard the old asphalt layer as part of the unbound-base material. A construction that has 400 mm of base requires an asphalt thickness of more than 120 mm, as can be determined from chart HT 58, shown in Figure 22, which is the standard chart most closely corresponding to the design parameters of the example. Apparently, it is advantageous to use the residual (fatigue) life of the old pavement; the required minimum overlay is 120 mm.

Example 3

Assume, this time, an E_3 of 150 MPa, an $h_{1\,eff}$ of 50 mm, an h_2 of 200 mm, and a climate that has a w-MAAT of 18°C. The interpolated EN charts used for example 2 can again be used (Figure 21). Assume also that N_{A1} = 600 000 standard axle loads and that N_{A2} = 40 000 000 standard axle loads.

Inserting the FWD data (E_3 = 150 MPa and $h_{1\,eff}$ = 50 mm) into Figure 21a shows that N_{D1s} = 3 000 000 standard axle loads (point K). Thus, N_{Rs} = 3 000 000 - 600 000 = 2 400 000 standard axle loads and, from Figure 21a, the required total asphalt thickness for the future design life is 150 mm (point L). Therefore, the required overlay thickness from the point of view of the subgrade-strain criterion is 150 - 50 = 100 mm.

Inserting the same FWD data into Figure 21b shows that N_{D1a} = 800 000 standard axle loads (point M). This means that $N_{Ra} = N_{D1a} - N_{A1}$ = 200 000 standard axle loads. The fictitious design number (N_{D2}) for an N_{A2} of 40 000 000 standard axle passes can be calculated as follows: N_{D2} =

Figure 23. Effect of writing off existing pavement on overlay thickness: example 3.

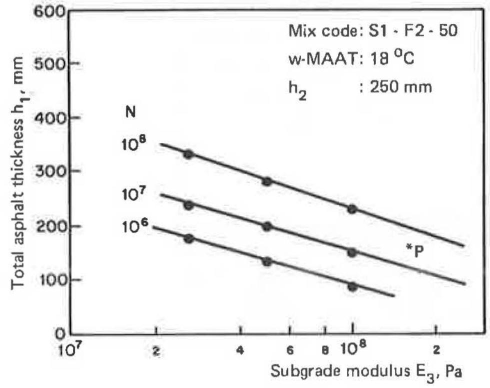
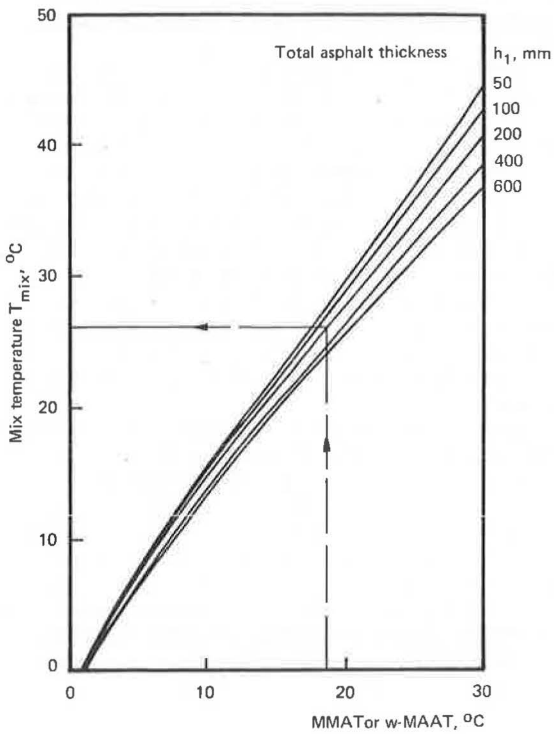


Figure 24. Determination of effective asphalt temperature by using chart RT.



800 000 × 40 000 000 ÷ 200 000 = 150 000 000 standard axle loads. This design number (Figure 21b) means that a total asphalt thickness of 230 mm (point O) is required and that the required overlay thickness from the point of view of the asphalt-fatigue criterion will be 230 - 50 = 180 mm.

Thus, the asphalt strain was the governing criterion in the original pavement and will remain so in the overlaid structure. This is self-evident because the maximum strain occurs in the same place both before and after overlaying, i.e., at the underside of the original asphlat layer. In this case, therefore, the separate determination of the required overlay thickness from the point of view of subgrade strain could actually have been omitted.

What remains to be done is to check whether it is advantageous to regard the old pavement as having failed completely. If it is so regarded, the total thickness of the unbound base becomes 200 + 50 = 250 mm. Another interpolated EN chart can now be developed, this time for an h_2 value of 250 mm. This chart (see Figure 23) is found, not surprisingly, to be based on the asphalt strain. The subgrade-strain criterion simply is not decisive under these circumstances. The required asphalt thickness is 170 mm (point P).

Hence it is economical, although not very much so, to write off the old pavement completely and apply an overlay of 170 mm, as compared with the 180 mm that would be needed to prevent the existing pavement from cracking.

OVERLAYS OF DIFFERENT MIX TYPES

There will be many cases in which the overlay will be of a different mix type from that used in the existing pavement. This does not immediately invalidate the approach that assumes one mix type for the entire construction, i.e., existing pavement and overlay together. If, for example, the difference is one of composition only, the combination of different grades of bitumen could very well produce a mix that has nearly the same stiffness level at the temperatures present in the pavement in the given climate.

For example, a structure approximately 200 mm thick in a climate that has a w-MAAT of 18°C has an effective asphalt temperature of about 26°C (this value was obtained from chart RT of the manual, as shown in Figure 24). At 26°C, a mix that has a stiffness characteristic S2 but is made with 50 pen bitumen produces the same stiffness level as a mix that has a stiffness characteristic of S1 but is made with 100 pen bitumen (see Figure 25).

Furthermore, if the fatigue characteristic of the overlay mix differs from that of the existing pavement, it will normally be possible simply to use one fatigue characteristic because the maximum asphalt strain after the overlay has been applied will still occur in the same place, i.e., at the underside of the existing pavement. For example, the thickness of an overlay of an S1-F1-100 mix on an existing pavement of an S2-F2-50 mix can be calculated by assuming the mix code S2-F2-50 for the entire construction.

If the stiffness level of the overlay mix differs significantly (by a factor greater than 2) from that of the existing pavement, the necessary overlay thickness is first determined as described above, on the basis of the mix code of the existing pavement. Then, equivalency factors are calculated that indicate the thickness of the overlay mix that would be needed to replace a given thickness of the mix type of the existing pavement. Briefly, the procedure is as follows:

Because the subgrade strain depends not only on the stiffness level of the pavement on top of it but rather on the combination of stiffness level and layer thickness of the pavement, the procedure for calculating on the basis of the subgrade-strain criterion is fairly simple and straightforward.

Consider, for example, a design whereby a 50-mm overlay is to be laid on top of an existing pavement 180 mm thick and assume a mix code for both of S1-F2-100, so that the total depth of asphalt will be 230 mm (based on the subgrade-strain criterion). The asphalt mix envisaged for the overlay, however, has a mix code of S1-F1-50. For a mix of this type, the total depth required to limit the subgrade strain to the level given by that criterion will be 180 mm, which leads to the conclusion that 180 mm of S1-F1-50 replaces 230 mm of S1-F2-100. The required overlay thickness of 50 mm of S1-F2-100 can thus be replaced by 180 × 50 ÷ 230 = 40 mm of S1-F1-50.

Figure 25. Derivation of mix stiffness from mix temperature by using chart M-2.

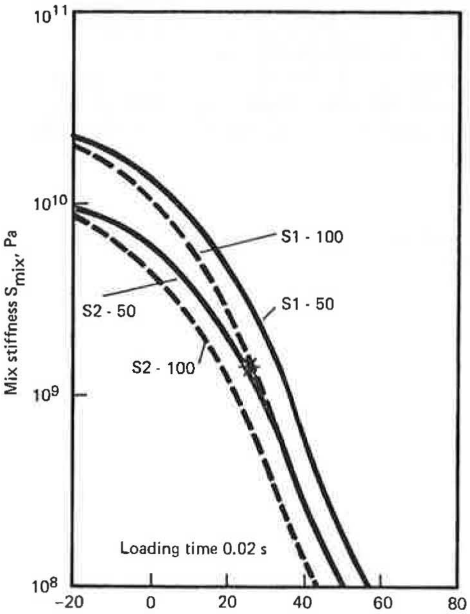
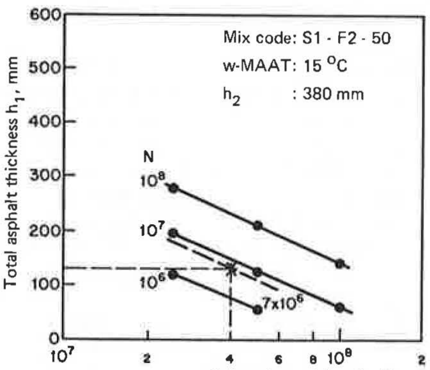


Figure 26. Determination of overlay thickness assuming complete failure of existing pavement (overlay of different mix type).



Again, it must be checked whether the asphalt strain may become the governing factor. Here the matter is complicated by the fact that, on the one hand, the level of permissible strain is governed by the combination of pavement thickness and stiffness level but that, on the other hand, it also depends on the stiffness level itself. Therefore, the equivalency factor must be determined at the same stiffness level. This can be done by comparing the pavements at different temperatures, choosing these temperatures in such a way that both mixes have the same stiffness level and thus the same permissible asphalt strain.

Let us assume that, on the basis of asphalt fatigue life, a total depth of 250 mm is required of a mix that has a code of S1-F2-100 and the existing pavement thickness is 180 mm of a mix that has a code of S1-F2-100. However, the mix intended for the overlay has a code of S1-F1-50, with which the required future design life could be obtained by using a total asphalt depth of only 130 mm. The effective temperature in the pavement can be determined from the w-MAAT and the approximate thickness of the existing pavement plus the overlay (Figure 24).

Suppose that, for this particular pavement, the effective pavement temperature is 22°C. As shown in Figure 25, the mix that has a code of S1-F2-100 has the same stiffness level at 16°C as the mix that has a code of S1-F1-50 has at 22°C. The effective temperature of 16°C in the given approximate pavement thickness corresponds to a w-MAAT of 11°C. According to the charts, this gives a required total thickness of a mix coded S1-F2-100 (at a w-MAAT of 11°C) of 220 mm (based on asphalt fatigue). The equivalency factor is thus 220 ÷ 250 = 0.88, and the required overlay thickness of 250 - 180 = 70 mm of S1-F2-100 mix can be replaced by 0.88 × 70 ≈ 60 mm of S1-F1-50 mix.

In this example, the overlay thickness based on the subgrade-strain criterion was decisive. It still needs to be checked, of course, whether the approach whereby the old asphalt layer is taken as having deteriorated to failure would give a more economical overlay thickness. Figure 26 illustrates that, for the circumstances of this example, a thickness of 130 mm of S1-F1-50 is required on top of an unbound base layer of 200 + 180 = 380 mm, so this is clearly not the case.

CONCLUSIONS

Deflection measurements made with a falling weight deflectometer can provide the road engineer with meaningful data on a pavement structure. From these data, the state of the pavement (e.g., in terms of residual life) can be evaluated in an analytical way, and, if necessary, the structural restrengthening measures (e.g., in terms of overlay thickness) that should be undertaken can be determined. The data provided by the FWD are sufficiently accurate to tailor the design to the individual circumstances but, at the same time, are produced quickly enough for routine investigations.

The charts in the Shell Pavement Design Manual can be used by the designer, without resort to computer calculations, to determine analytically the overlay thicknesses required for a variety of circumstances and can even introduce a large measure of refinement if this is required.

ACKNOWLEDGMENT

I wish to offer sincere thanks to R. Ditmarsch, C. W. M. Voermans, and J. Voorwinden for their invaluable help in the preparation of the necessary examples and to J. M. Edwards, J. W. Klinkenberg, J. van Schooten, and W. Visser for their many useful suggestions and constructive remarks. The cooperation of the authorities of the municipality of Terneuzen in permitting the use of evaluation data pertaining to some of their roads is gratefully acknowledged. A pavement evaluation service that operates a falling weight deflectometer is available from Pavement Consultancy Services B. V.

REFERENCES

1. Shell Pavement Design Manual: Asphalt Pavements and Overlays for Road Traffic. Shell International Petroleum Company, London, 1978.
2. P. J. Van de Loo. Practical Approach to the Prediction of Rutting in Asphalt Pavements: The Shell Method. TRB, Transportation Research Record 616, 1976, pp. 15-21.
3. A. I. M. Claessen, J. M. Edwards, P. Sommer, and P. Ugé. Asphalt Pavement Design: The Shell Method. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977, pp. 39-74.
4. A. I. M. Claessen and R. Ditmarsch. Pavement

Evaluation and Overlay Design: The Shell Method. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977, pp. 649-661.
5. A. I. M. Claessen, C. P. Valkering, and R. Ditmarsch. Pavement Evaluation With the Falling Weight Deflectometer. Proc., AAPT, Vol. 45, 1976, pp. 122-157.
6. C. P. Valkering. Pavement Evaluation by Measuring Deflection and Shape of Deflection Bowl. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 2, 1972, pp. 353-356.
7. D. L. De Jong, M. G. F. Peutz, and A. R. Korswagen. Computer Program BISAR: Layered Systems Under Normal and Tangential Surface Loads. Koninklijke/Shell-Laboratorium, Amsterdam, External Rept. AMSR.0006.73, 1973.
8. M. G. F. Peutz, H. P. M. Van Kempen, and A. Jones. Layered Systems Under Normal Surface Loads. HRB, Highway Research Record 228, 1968, pp. 34-45.

Publication of this paper sponsored by Committee on Pavement Rehabilitation Design.

Mechanistic Method of Pavement Overlay Design

Harvey J. Treybig, ARE Inc., Austin, Texas

This paper presents a synopsis of a comprehensive procedure for the rehabilitation design of overlays of both flexible and rigid pavements. The procedure includes an evaluation of the existing pavement by using modern nondestructive deflection testing, condition surveys, and materials sampling and testing. The analytical model on which the computer method is based is elastic-layer theory. This model is used in both the pavement-evaluation and the overlay-thickness analyses. This design and evaluation analysis is unique for various categories of pavement condition. The final overlay thicknesses are selected on the basis of fatigue and rutting criteria where applicable. The entire procedure is automated in a series of three computer programs.

This paper describes the use of a universal procedure for the design of pavement overlays. The detailed development of the criteria for the procedure is discussed and documented in several reports (1-4).

The procedure covers flexible overlays of flexible pavements and both flexible and rigid overlays of rigid pavements. It includes both jointed and continuous rigid pavements and both bonded and unbonded overlays. It covers existing pavements that have remaining life, those that are substantially cracked, and those that are so badly deteriorated that they could be broken mechanically into small pieces. The procedure infers that overlay materials and construction specifications will not differ significantly from those currently used in highway construction. However, it does include some nonconventional materials testing methods.

The comprehensive procedure provides for rehabilitation of existing portland cement concrete (PCC) and asphalt concrete (AC) pavements and is divided into three basic steps: (a) evaluation of the existing pavement, (b) determination of the design inputs, and (c) overlay-thickness analysis. The procedure is illustrated in flow-chart form in Figure 1. Evaluation of the existing pavement is accomplished by a condition survey and deflection measurements. This information enables the designer to distinguish among different segments of the existing pavement based on their condition. Each segment becomes a design section and is analyzed separately. Thus, the most economical rehabilitation may involve varying the overlay thickness along the roadway according to the existing pavement condition.

The design inputs include both past and future traffic, environmental considerations, and materials testing and analysis. The results of deflection measurements also serve to aid in establishing properties of the subgrade material.

The overlay-thickness analysis is based on the concepts of failure by excessive rutting (flexible pavements) and by excessive fatigue cracking (rigid and flexible pavements). Stresses and strains in the pavement are computed by using linear elastic-layer theory (5). The overlay life is determined by entering these stresses into a fatigue or rutting equation that relates stress or strain magnitude and repetitions to failure. The overlay thickness that satisfies the fatigue and rutting criteria is selected as the design thickness.

The design procedure is automated in the form of three separate computer programs—PLOT2, TVAL2, and POD1. The programs require the designer to make only minor hand computations, and these are only aids in determining computer-program input data.

GENERATION OF DESIGN-PROCEDURE INPUTS

The design procedure requires input from the following three areas: deflection testing, condition surveys, and traffic data.

Deflection Testing

Deflection testing is used to measure the response of the in-service pavement to load. From this behavior pattern, areas that have equal or similar performance and materials properties can be determined.

1. Type of equipment: Any type of deflection-measuring equipment (such as the Dynaflect or the road rater) that gives satisfactory deflection results can be used (6). This type of equipment lends itself to rapid testing, thus making it economically possible to investigate the pavement structure thoroughly. Deflections measured with a Benkleman beam and an 80-kN (18 000-lbf) single-axle load can also be used.