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Structural Performance Model and Overlay Design Method for Asphalt Concrete Pavements

A. A. A. MOLENAAR AND CH. A. P. M. VAN GURP

The development of a structural performance model for flexible pavements is described. This model consists of a set of probability-of-survival curves in which the structural deterioration of pavement structures, which are characterized with their equivalent layer thickness, is given with respect to the number of load repetitions. The equivalent layer thickness is calculated according to Odemark's theory. It is shown that the equivalent layer thickness and the survival rate of the pavement can be determined by means of deflection measurements. Furthermore, it is shown how an in situ asphalt concrete fatigue relation can be derived for the construction considered. An overlay design chart based on the equivalent layer thickness concept is given, and an example of how the developed techniques are used for the overlay design of asphalt pavements is presented.

Because the economic recession is restricting the available pavement maintenance and rehabilitation budget, an optimal allocation of this budget for maintenance projects becomes more and more important. It is obvious that, in this situation, engineering judgment alone is not enough to solve overlay design and budget-allocation problems. More emphasis is therefore placed on the so-called rational methods.

This paper describes the efforts of the Laboratory for Road and Railroad Research, Delft Univer-

sity of Technology, on the development of structural performance and overlay design methods. These models are based on deflection measurements that were carried out on several road sections over a three-year period and on a theoretical analysis of three layer pavement systems. An example illustrates the use of these models.

STRUCTURAL PERFORMANCE MODEL: THEORETICAL ANALYSIS

Ninety-three layer structures were analyzed with the BISAR computer program to derive relations between the equivalent layer thickness (h_e) and the maximum strain in the asphalt layer or the vertical compressive strain at the top of the subgrade (1). The equivalent layer thickness is defined as follows:

$$h_e = \sum_{i=1}^2 0.9 h_i \sqrt[3]{E_i/E_3} \quad (1)$$

where

h_e = equivalent layer thickness (m),
 h_i = thickness of layer i (m),

E_i = elastic modulus of layer i (N/m^2), and
 E_3 = elastic modulus of the subgrade (N/m^2).

The geometry of the loading system used in the analysis is shown in Figure 1. The results of this analysis are presented in Figure 2. The equivalent layer thickness was used as an independent variable because it is a simple way to describe the bearing capacity of pavement structures. The equivalent-layer-thickness concept was originally developed by Odemark (2) and has been successfully used by Jung and Phang (3) in the assessment of subgrade deflection.

Figure 1. Geometry of loading system.

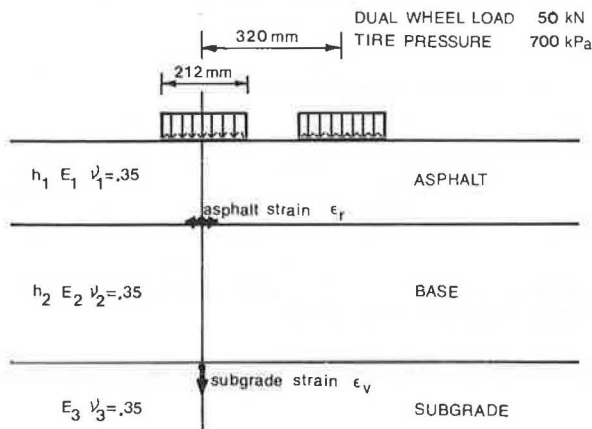
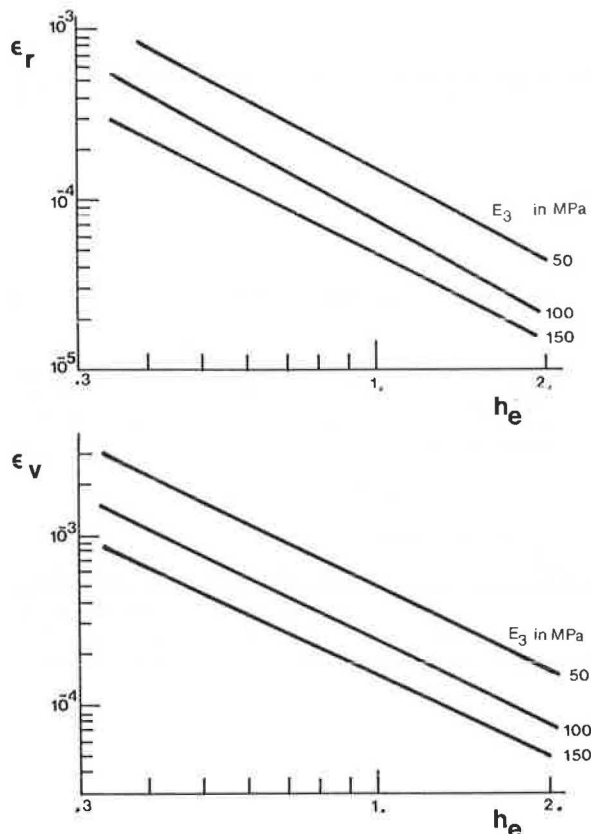


Figure 2. Relations between equivalent layer thickness (h_e) and maximum asphalt strain (ϵ_r) or subgrade strain (ϵ_v).



From the relation given in Figure 2 and appropriate fatigue relations for the asphalt mix and the subgrade considered, relations between the equivalent layer thickness and the pavement life expressed as the number of equivalent 100-kN axles could be developed. An example is given in Figure 3. Figure 3a shows the equivalent layer thickness versus asphalt strain relation, while Figure 3b shows the fatigue relation for a gravel-sand asphalt mix. Combining Figures 3a and 3b results in Figure 3c, which gives the relation between the equivalent layer thickness and the pavement life based on the asphalt strain criteria. The same procedure is followed to derive relations between the equivalent layer thickness and the pavement life based on the subgrade strain criteria (Figures 3d, 3e, and 3f).

Finally, the combination of Figure 3c and Figure 3f results in Figure 3g, which shows the overall relation between equivalent layer thickness and pavement life. Figure 3g can be used for pavement design purposes, which means that an equivalent layer thickness can be selected if the design number of load applications is known. The equivalent layer thickness determined in this way can then be split up in layer thicknesses and layer moduli by using Equation 1. In fact, alternative designs can be made very easily by using the procedure described here.

It should be noted that the pavement life determined from Figure 3g is the mean pavement life because it is based on the mean value of material properties, layer thicknesses, and strains. The mean pavement life means that the design has a reliability of 50 percent.

By taking into account the variability of layer thicknesses and material properties, survival curves have been derived from design charts as given in Figure 3. These survival curves give the probability of survival of a given design (h_e) in relation to the number of load applications. An example of such a survival curve is given in Figure 4.

The way in which these probability-of-survival curves were derived is fully described elsewhere (1). However, for the sake of completeness it will be repeated here.

First, the variance of the equivalent layer thickness was assessed from the following equation:

$$S_{h_e}^2 = \sum_{i=1}^{L-1} (\delta f / \delta h_i)^2 S_{h_i}^2 + \sum_{i=1}^L (\delta f / \delta E_i)^2 S_{E_i}^2 \quad (2)$$

where

$S_{h_i}^2$ = variance of layer thickness i ,

$S_{E_i}^2$ = variance of the elastic modulus of layer i ,

L = number of layers,

$f = 0.9 \sum_{i=1}^L h_i \sqrt{E_i / E_s}$, and

E_s = elastic modulus of the subgrade.

Next, $S_{\log \epsilon}^2$ was calculated from

$$S_{\log \epsilon}^2 = d^2 S_{\log h_e}^2 + S_{\log f, (h_e - \epsilon)}^2 \quad (3)$$

where

$S_{\log \epsilon}^2$ = variance of the estimated strain,
 d^2 = slope of the equivalent layer thickness versus strain relation, and

$S_{\text{l.o.f. } (h_e - \epsilon)}^2$ = variance due to lack of fit of the equivalent layer thickness versus strain relation.

Finally, $S_{\log N}^2$ was calculated from

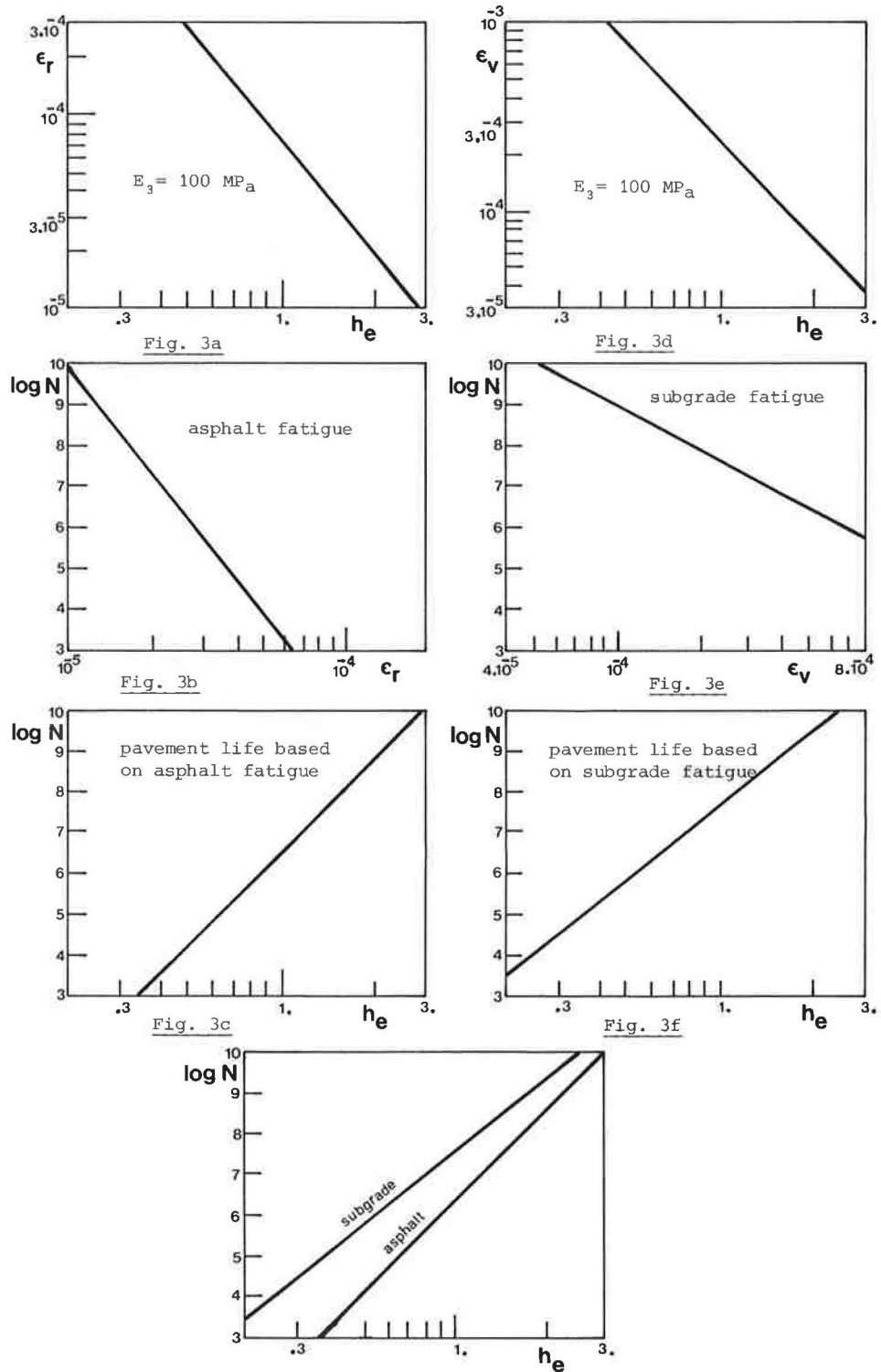
$$S_{\log N}^2 = b^2 S_{\log \epsilon}^2 + S_{\text{l.o.f. } (N - \epsilon)}^2 \quad (4)$$

where

$S_{\log N}^2$ = variance of the estimated number of load repetitions to failure, b^2 = slope of the used fatigue relation, and

$S_{\text{l.o.f. } (N - \epsilon)}^2$ = variance due to lack of fit of the strain versus number of repetitions to failure relation (fatigue relation).

Figure 3. Pavement design curves.



The number of load repetitions to a certain level of reliability (N_p) can now be calculated from

$$\log N_p = \log \bar{N} - U_p S_{\log N} \quad (5)$$

where \bar{N} is the number of load repetitions that have all variables set at their mean value and U_p is the value taken from the normal tables that corresponds with the desired confidence level P . Figure 4 is derived by using an $S_{\log N}$ value of 0.4.

From the foregoing, it will be clear that any value for $S_{\log N}$ can be used by knowing the variations in layer thicknesses and material properties.

Figure 4. Probability-of-survival curves for a given set of equivalent layer thicknesses.

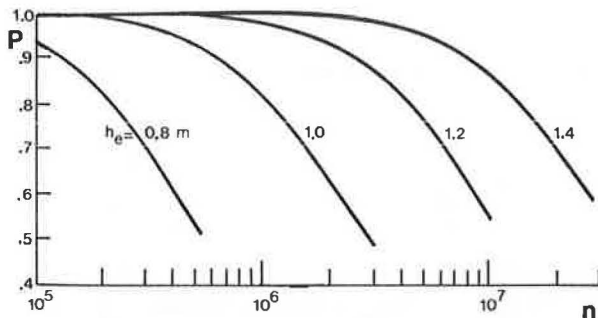


Figure 5. Basic set of survival curves.

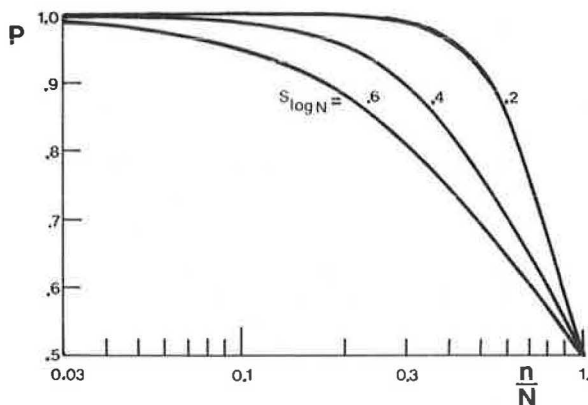
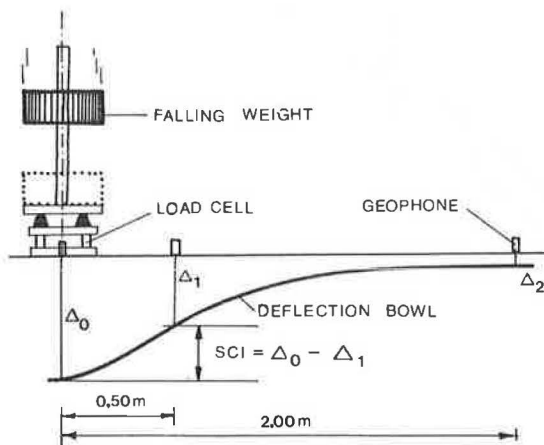


Figure 6. Principles of falling-weight deflection measurements.



Because the shape of the survival curves depends on the material properties and their variations, a basic set of survival curves was derived, which is shown in Figure 5. Figure 5 was derived simply by using different values for $S_{\log N}$ in Equation 5. Now the probability of survival is given as a function of the ratio of applied number of load applications to allowable number of load applications and $S_{\log N}$.

Later it will be shown how a value for $S_{\log N}$ can be determined from deflection measurements. Then it will be shown how Figure 5 will be used for residual life assessment.

ASSESSMENT OF EQUIVALENT LAYER THICKNESS FROM DEFLECTION MEASUREMENTS

Deflection measurements in this study were taken by means of a falling-weight deflectometer. The basic principle of this device is shown in Figure 6. From BISAR calculations, the following relation could be derived between the surface curvature index measured with the falling-weight device and the equivalent layer thickness:

$$\log h_e = -1.117 - 0.486 \log E_3 - (0.556 - 0.021 \log E_3) \log (21 \times 10^{-6} + 0.6 \text{ SCI}) \quad (6)$$

where SCI is the surface curvature index measured with a falling-weight deflectometer ($P = 50 \text{ kN}$, $t = 0.02 \text{ s}$) (m), and E_3 is the elastic modulus of the subgrade (MN/m^2).

The elastic modulus of the subgrade can be assessed directly from the deflection measured 2 m from the loading center with the following equation:

$$\log E_3 = 3.869 - 1.009 \log \Delta_2 \quad (7)$$

where Δ_2 is the deflection measured at 2 m from the loading center ($P = 50 \text{ kN}$, $t = 0.02 \text{ s}$).

Temperature corrections should be applied on the calculated h_e in order to get the h_e at the reference temperature of 11°C . This temperature was selected as the reference temperature because it is the weighted mean annual air temperature of 11°C for Dutch conditions (4). It has been shown elsewhere (5) that the temperature correction should be applied in the following way:

$$h_{e,t=11^\circ\text{C}} = h_{e,t} - (11 - t) \cdot 0.014 \quad (8)$$

where

$$h_{e,t} = h_e \sqrt[3]{100/E_3} \quad (9)$$

and t is the air temperature at the measurements.

The temperature-corrected $h_{e,t=11^\circ\text{C}}$ is calculated with the following equation:

$$h_{e,t=11^\circ\text{C}} = h_{e,t=11^\circ\text{C}} \sqrt[3]{100/E_3} \quad (10)$$

OBSERVED STRUCTURAL PERFORMANCE

By using the techniques described in the previous sections, the decrease of the h_e in time was determined for a number of road sections by means of deflection measurements that were carried out over a three-year period. Because extensive measurements during and after completion of the construction were carried out by others on a couple of sections (6), the decrease of the h_e from the opening of those sections to traffic could be determined. An example of such a deterioration curve is given in Figure 7a. Figure 7b gives the same deterioration curve, but

Figure 7. Decrease of h_e with respect to (a) time (years) and (b) number of load applications.

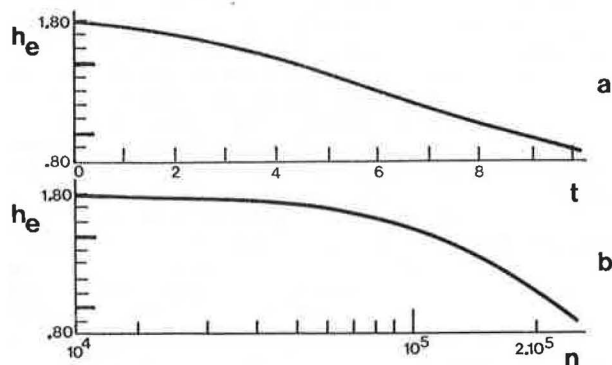


Figure 8. Decrease of K with respect to ratio n/N and β .

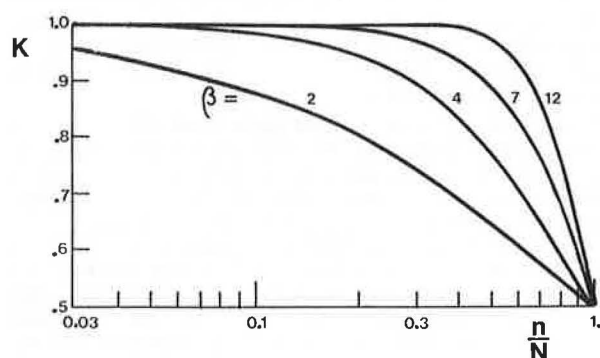
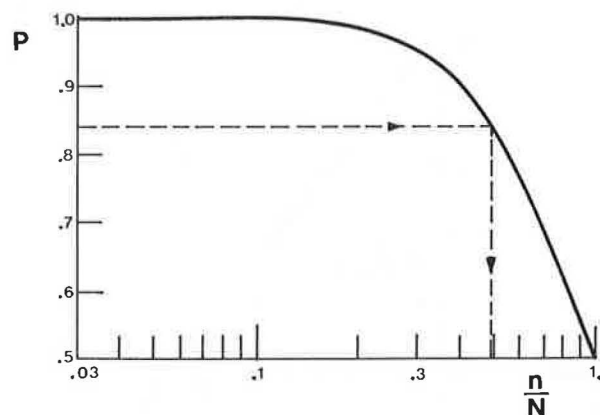


Figure 9. Determination of residual life $(1 - n/N)$ from P and $S_{\log N}$.



now with respect to the number of equivalent 100-kN single axles that have passed the section considered. For almost all of the sections that were studied, a comparable decrease of h_e was observed. On some sections, the expected decrease of h_e could not be determined.

The decrease of h_e with respect to the ratio n/N (Figure 7b) can be written as follows:

$$K = 1/[1 + \exp(\beta \log n/N)] \quad (11)$$

where

$$K = h_{e_n} / h_{e_0}$$

h_{e_n} = equivalent layer thickness after n load applications,
 h_{e_0} = equivalent layer thickness at the beginning of the pavement life ($n = 0$), and
 β = a constant.

A graphical representation of Equation 11 is shown in Figure 8. If one compares Figure 8 (observed structural deterioration) with Figure 5 (theoretically derived structural deterioration), one notices the striking resemblance. Therefore, it is concluded that K (Figure 8) and P (Figure 5) are exchangeable with each other and that P can be determined by means of deflection measurements. Furthermore, it is concluded that a proper value of $S_{\log N}$ can be determined for each construction from a comparison of β and $S_{\log N}$ with each other (compare Figures 8 and 5). If the h_e is known for at least two values of n , β can be determined by means of regression techniques.

From our study, we concluded that β (and so $S_{\log N}$) is dependent on the type of construction. For rather rigid constructions (i.e., constructions with a cement-stabilized base), $S_{\log N}$ is about 0.3. For full-depth asphalt constructions $S_{\log N}$ is about 0.4, and for constructions with an unbound granular base $S_{\log N}$ is about 0.6. Next we will show how one or the other is used in the assessment of the residual life of pavement structures.

RESIDUAL LIFE ASSESSMENT

For reasons of costs and time, it will not be possible to perform regular deflection measurements in order to follow the decrease of h_e in time. These measurements are mostly taken at the moment when one expects that an overlay should be applied on the construction considered. In order to be able to make a proper estimation of K , it is therefore recommended to perform deflection measurements in the wheel tracks and between the wheel tracks. The value of h_e calculated from the measurements between the wheel tracks is thought to be a reasonable estimate for h_{e_0} , since this part of the

structure is hardly subjected to traffic loading.

Furthermore, a value for $S_{\log N}$ can be selected from the type of construction (see previous section). Knowing K and $S_{\log N}$, the corresponding value for the ratio n/N can be determined (Figure 9), and the pavement life N can be calculated if the applied number of load applications n is known. The residual life, which is defined as $N - n$, can then easily be calculated. Once again it should be noted that the ratio $n/N = 1$ corresponds to the mean pavement life and that ratios greater than 1 can occur.

OVERLAY DESIGN

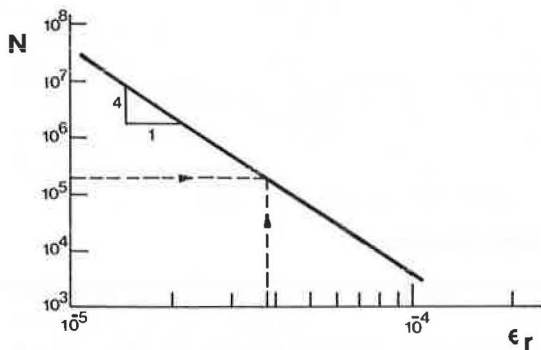
In order to be able to make life predictions for a strengthened construction, one should know the fatigue behavior of the asphalt materials used in the existing construction. Normally, laboratory-determined fatigue relations are used, which are corrected for beneficial effects as rest periods, etc. It is believed, however, that the fatigue behavior in situ can be very different from laboratory-determined fatigue behavior. Therefore, it is suggested that for each construction considered, an in situ fatigue relation should be determined. This is done in the following way.

From the h_{e_0} , which is determined according to the procedure outlined in the previous section, the

initial maximum tensile strain in the asphalt layer is determined by using Figure 2. Because the pavement life N can also be calculated (see previous section), the combination of the calculated ϵ and N will result in a point in the fatigue diagram (Figure 10).

To get a fatigue relation, a line with a slope of -4 is drawn through this fatigue point (Figure 10). A value of -4 is selected for the slope of the fatigue line because it is a reasonable value compared with slope values for in situ fatigue relations reported elsewhere (7,8). This fatigue relation, determined in the above-described way, is used to predict the pavement life N for lower values of ϵ (i.e., higher values of h_e). An example of the process described above is given next.

Figure 10. Determination of an in situ fatigue relation.



From a construction that consists of a 100-mm-thick asphalt layer on a 350-mm unbound granular base layer, the ratio of h_e in the wheel tracks to h_e between the wheel tracks is determined to be 0.6. Because the construction consists of a rather thick unbound base layer, the value of $S_{\log N}$ is set at 0.6. From Figure 11a it is determined that the ratio n/N is 0.7. Because the number of equivalent 100-kN single axles that have loaded the construction (n) equals 2.1×10^5 , the allowable number of equivalent 100-kN single axles is 3×10^5 .

The equivalent layer thickness between the wheel tracks is 1000 mm; from Figure 11b it was determined that the initial tensile strain (ϵ_i) is 7.3×10^{-5} . The combination of the calculated ϵ_i and N resulted in the fatigue relation for the construction considered, which is given in Figure 11c. The pavement life for higher values of h_e was calculated, and pavement survival curves were derived for those higher h_e values, assuming that $S_{\log N}$ for these constructions is the same as the $S_{\log N}$ for the construction considered (0.6). These curves are shown in Figure 11d. For the construction considered, it was determined that it should sustain another 7×10^5 equivalent 100-kN single-axle repetitions.

From Figure 11d it can be seen that if the minimum acceptance level for the survival rate is set at 0.65, a construction with $h_e = 1270$ mm is capable of sustaining the design number of load repetitions. The needed increase in h_e is therefore 270 mm.

Figure 12 shows the overlay design chart that is used to determine by which overlay the needed increase in h_e can be achieved. This figure was derived from BISAR calculations. As can be seen from Figure 12, the overlay in the example can be

Figure 11. Illustration of example problem.

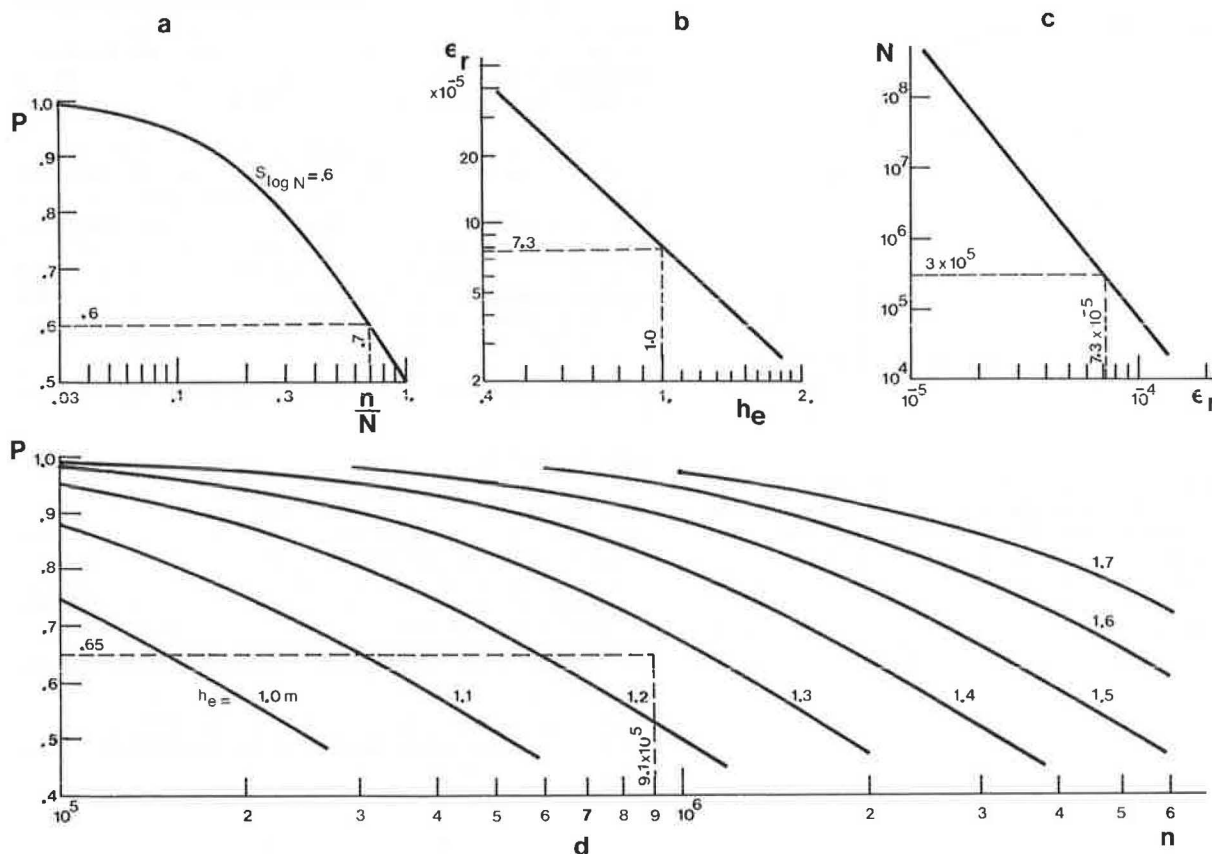
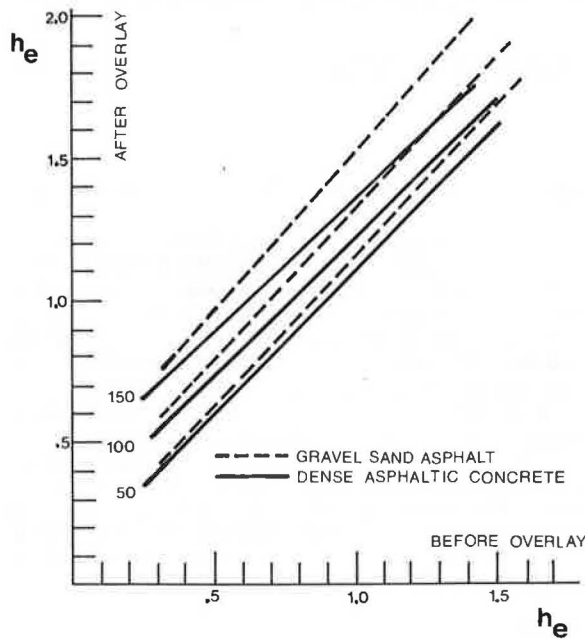


Figure 12. Overlay design graph (overlay thickness in millimeters).



either a 75-mm gravel-sand asphalt layer or a 110-mm dense asphaltic concrete layer. Because of better skid resistance and crack-propagation properties (slower crack propagation), the dense asphaltic concrete solution is selected.

As will be noted, the minimum acceptance level for the survival rate has a rather high influence on the overlay thickness. A value of 0.65 was selected, because accepting lower values of the survival rate will increase the need of routine maintenance because of a worse condition of the pavement surface, especially if rather severe winter conditions occur (9).

CONCLUSIONS

Based on the results of the study presented in this paper, the following conclusions can be drawn:

1. Because the equivalent layer thickness correlates well with the maximum tensile strain in the asphalt layer or the vertical compressive strain at the top of the subgrade, it is an adequate parameter to judge the pavement bearing capacity.
2. The equivalent layer thickness can be easily determined by means of deflection measurements.
3. The probability-of-survival curves derived from observed structural deterioration of pavements can be described with an e-power relation that shows very much resemblance with the theoretically derived probability-of-survival curves.
4. The shape of the survival curve seems to be

dependent on the type of construction.

5. The survival rate of the pavement can be determined by means of deflection measurements in and between the wheel tracks.

6. For each pavement section considered, an appropriate in situ fatigue relation can be determined from the equivalent layer thickness determined from deflection measurements taken between the wheel tracks. The precondition is, however, that no cracking be visible in this area.

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