Ontario

Ministry of
Transportation and
Communications

Research and
Development
Division

A Performance Prediction Subsystem

Flexible Pavements

RR 200

Ontario Pavement Analysis of Costs

OPAC

Ontario Pavement Analysis of Costs

A Performance Prediction Subsystem

Flexible Pavement

F. W. Jung
Research Officer

Ramesh Kher
Research Officer

W. A. Phang
Head, Pavement Research

Engineering Research and
Development Branch
Research and Development Division

Publication Coordination and Design
by Technical Information Group

Printed by Graphic Services Office

Ontario

Ministry of
Transportation and
Communications

Research and
Development
Division

Published by the Ministry of Transportation and Communications, Ontario.
Hon. John R. Rhodes, Minister
H. F. Gilbert, Deputy Minister

Copies of this report may be obtained by writing to
The Editor
Research and Development Division
Ministry of
Transportation and Communications,
Downsview, Ontario
Canada M3M 1J8.

First Printing August 1975

Summary

A procedure for predicting pavement performance is required for advanced systems of pavement design and management. Long-range highway costs and user benefits can only be established if the future serviceability of pavements can be determined, since programing of construction and resurfacing priorities, finding optimal solutions in design and setting administrative policies all depend on cost-benefit calculations. Thus, there is a need for mathematical models which can predict pavement performance as a function of age.

The approach presented in this paper suggests that pavement performance models can be developed for any geographic and environmental conditions. The approach has been formulated so that local experience gained from successful highway pavements can be used. In principle, the performance of a pavement is affected by two major distress mechanisms—traffic loads and environmental conditions. Traffic generates pavement distress through fatigue and creep of materials which results in cracking and permanent deformation. Environment causes deterioration of pavements, predominantly through differential heave and settlement of the subgrade, resulting in roughness and cracking of the surface.

Pavement distress due to traffic has been modelled using AASHTO Road Test performance data which primarily represent traffic-type deterioration. Successful thickness designs in the Province of Ontario were analyzed using Elastic Layer Theory. This analysis, supplemented by AASHTO Road Test data, led to the development of a model which reflects primarily a deterioration due to traffic. Distress due to environment was modelled using the Brampton Road Test data. This data indicated that environmental type distress can best be quantified by an exponential mathematical function. The resulting submodels for the two distress types were combined to give a final model which predicted fairly accurately the performance of the Brampton Road Test sections.

Further tests have indicated that the final model has successfully predicted pavement lives in close agreement with general experience in the province.

Contents

1 Introduction

A primary objective of a pavement management system is to provide pavement surfaces with acceptable riding quality, at the lowest possible cost. It is widely accepted that under the same traffic loading, a weak pavement structure will deteriorate faster and become rough more quickly than a strong pavement structure. The strong structure will require thicker construction with higher quality materials and initially will be more costly than the weak structure. Thus, the benefits of the longer life attained by stronger pavement must be weighed against the higher construction costs incurred. The stronger pavement will provide a smoother ride for a longer time than the weaker pavement and thus will favorably affect user vehicle operation and time costs. Thus when considering alternative pavement designs, management must be able to predict the probable riding quality of a pavement structure as a function of its age.

The ability to predict riding quality at the design stage has become possible only recently, stemming from the results of the AASHTO Road Test [1]. An improved mechanistic and more generally applicable method has now been developed and is described in this paper. The term *performance*, as it is used here, is the history of the riding quality as it varies over a period of time. Since economic comparison of alternatives is considered within a time frame, the performance of overlays required to restore structural integrity and renew riding quality is also described.

Based on investigations by Phang [2, 3] and Lister [4], the performance of a flexible pavement structure can be predicted from its early deflection response to standard load tests. In an earlier paper Jung and Phang [5] established a relationship between performance and traffic by applying principles of elastic layer analysis to the AASHTO Road Test results. In the present paper, the performance prediction model has been extended to account for the losses in riding quality induced by the geographic and climatic environment, similar to those conceived by Scrivner et al [6, 7]. Further, a method is proposed to account for the deterioration of pavement layers at the end of the performance life when the pavement needs rehabilitation by an overlay.

The subsystem for performance prediction uses principles of linear elasticity to calculate the subgrade surface deflection of a flexible pavement structure when it is acted upon by a standard wheel load. The simple method of calculation as suggested by Odemark [8] produces results which correlate closely with subgrade surface deflection [5] calculated by more rigorous but complex analytical procedures using programs such as CHEVRON and BISTRO. This simple method of calculation makes it practical to investigate a large number of alternative strategies.

The sequence of steps in the subsystem is outlined in Figure 1. In the initial design of a pavement, Step 1, an equivalent gravel thickness is calculated. In Step 2, the calculated (Odemark) subgrade surface deflection for this structure is determined for the standard load and for the specific subgrade. Step 3 shows the traffic load in terms of repetitions N_i of the standard axle at any time Y_i in years. In Step 4a, the traffic N_i is used to determine the Riding Comfort Index (RCI) loss p_{Ti} due to traffic for the subgrade surface deflection calculated in Step 2. In Step 4b, the year Y_i from Step 3 is used to estimate the RCI loss P_{Ei} due to environment for the subgrade surface deflection calculated in Step 2. In Step 4c, the two RCI loss components p_{Ti} and p_{Ei} are summed and plotted against time Y_i. The life Y of the pavement is obtained from this plot where the sum intersects the desired total loss in Riding Comfort Index. A similar step procedure is used to determine the life of the overlay.

Details of each step are given in the paper, including explanations of qualification and calibration according to Ontario experience.

2 Flexible Pavement Performance Indicator

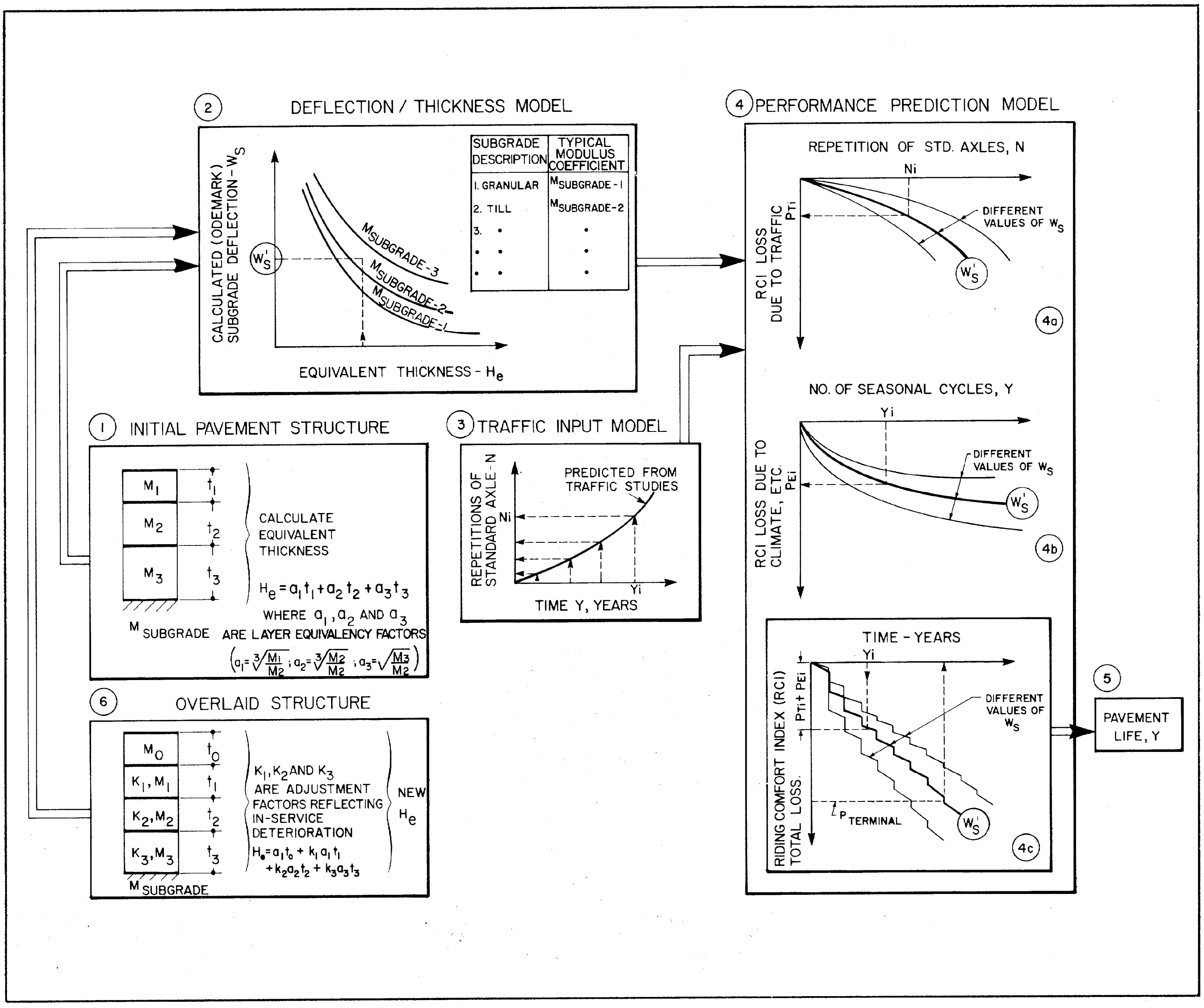
2

Pavement deflection has often been accepted by many design agencies as a fair indicator of the structural strength of a pavement. Empirical formulae have been developed [3, 9, 10] to predict this value which can normally be measured in the field by using a Benkelman Beam. Generally, limiting values of this deflection have been used as criteria for a "successful" pavement.

Recently, much effort has gone into applying elastic layer theory to predict deflections, as well as stresses and strains, and to design pavements based on these predicted responses. However, to arrive at a rational pavement design procedure it is necessary to identify which of these responses will adequately indicate the future performance of a pavement and, further, to know the location in the pavement structure where this response will be critical.

In Ontario, a comprehensive elastic layer analysis of existing pavement design experience [5] has revealed that elastic deformation at the top of the subgrade is the best indicator of pavement performance. The underlying hypothesis is that repeated subgrade deflections result in permanent deformation of the subgrade surface which eventually leads to permanent deformation of the pavement surface and a decrease in riding quality.

Figure 1. Flow Diagram Showing Steps in the Design Subsystem.



3 Performance Prediction Model

Subgrade deflections may be calculated by using the procedure developed by Odemark [8] whereby an elastic layer system can be transformed into an equivalent granular thickness system by means of equivalency factors. The computed equivalent thickness H_e is then used to calculate the deflection *w* at the top of the subgrade as shown in Figure 2. Layer equivalency factors for determining equivalent granular thickness as shown in this figure have been obtained from experience gained at the Brampton Road Test.

As per the Odemark method, certain properties of layer materials named elastic moduli, are required for calculating subgrade deflections [8]. As described in Reference [5], a specific set of these properties was arrived at through a comprehensive analysis of experiences on various classes of roads in Ontario. These are given in Figure 2. Layer equivalency factors as determined at the Brampton Road Test helped to establish the ratios among the values of these layer moduli (Figure 2). Since a fixed value of layer modulus is assigned to each layer, henceforth, these values will be called "layer coefficients" to avoid confusion with the traditional definition of a modulus. A close scrutiny of the deflection equation using various other sets of "layer coefficients" has revealed that absolute values of layer coefficients are immaterial as long as their "ratios" remain constant (since ratios determine layer equivalency factors).

The hypothesis that subgrade deflection is a good indicator of pavement performance and that this deflection can be determined adequately by the set of layer coefficients as given in Figure 2, is substantiated in Figures 3 and 4. Figure 3 shows an excellent correlation between Odemark subgrade deflection calculated for AASHTO Road Test sections (using the set of layer coefficients in Figure 2) and the number of 18-kip (80kN) equivalent applications successfully carried on these test sections to a terminal performance level of 2.5. Figure 4 shows a correlation that was observed at the Brampton Road Test between calculated Odemark subgrade deflections (using the same set of layer coefficients) and the measured Benkelman Beam surface deflections—the response which, from Canadian experience, is considered to be a good indicator of pavement performance [10].

By using the established set of layer coefficients, Figure 5 has been drawn to give a graphical representation of the Odemark subgrade deflection due to a standard 9000-lb. (40 kN) wheel load and for various subgrade layer coefficient values. Inherent in the curves of the figure are the following layer equivalency factors of Surface : Base : Subbase = 2 : 1 : 2/3. The deflection *w* is obtained from this figure after selecting a value for the subgrade layer coefficient (from the table in the figure determined on the basis of Ontario pavement performance experience on a variety of subgrades in the province) and after calculating the value of the total equivalent granular thickness H_e for a pavement structure by using the above layer equivalency factors. Also shown (on the right-hand side of the figure) is a scale from which corresponding peak Benkelman Beam surface deflections can be predicted as determined from the correlation established in Figure 4.

The method presented in the following sections shows how the Odemark subgrade deflection *w* is used to predict performance of a pavement structure.

3

The performance prediction model presented in this report is composed of:

a/a traffic input model which establishes the amount of traffic during the analysis time period (Figure 1, Step 3);
b/a model for predicting the performance loss p_T due to traffic loads (Figure 1, Step 4a);
c/a model for predicting the performance loss p_E due to geographic or environmental influences (Figure 1, Step 4b); and,
d/a method for combining p_T and p_E (Figure 1, Step 4c).

Immediately after construction, the Riding Comfort Index p_o of a pavement is relatively high and depends largely on the conditions during construction. From this value, the losses p_T and p_E must be subtracted to obtain the Riding Comfort Index *p* at any particular time. Thus,

$$p = p_o - P_T - p_E$$

The traffic loss p_T is a function of the number of equivalent standard axle load applications, N. The environmental loss p_E is a function of the number of years or seasons passed, Y. The number N is a function of Y as established by the traffic input model.

A. Traffic Input Model (Figure 1, Step 3). The traffic input model is basically a functional relationship between traffic load N and time Y. The relationship can be established from traffic data as proposed and discussed in Reference [11]. The following method is being used in Ontario.

B. Traffic Related Deterioration

Each project to be designed has a traffic estimate; usually an annual average daily traffic (AADT) in the design (first) year and a predicted AADT at the end of 20 years. First, the total equivalent standard (18-kip) single axle load applications N_f expected during the analysis period A_p are determined by using an average of initial and final estimates of the number of 18-kip (80 kN) axles per day in the design lane as described by Equation (2) in Figure 6. Subsequently, at any number of years Y, the number of corresponding standard load applications N are determined by using Equation (1) in the figure.

B. Traffic Related Deterioration (Figure 1, Step 4a). The calculated Odemark subgrade deflection *w* is used to derive a performance prediction equation based on the main factorial test data of the AASHTO Road Test [1]. The equation (Equation (11) in Reference [5]) has been rescaled from a 5-point index [Present Serviceability Index (PSI)] to a 10-point

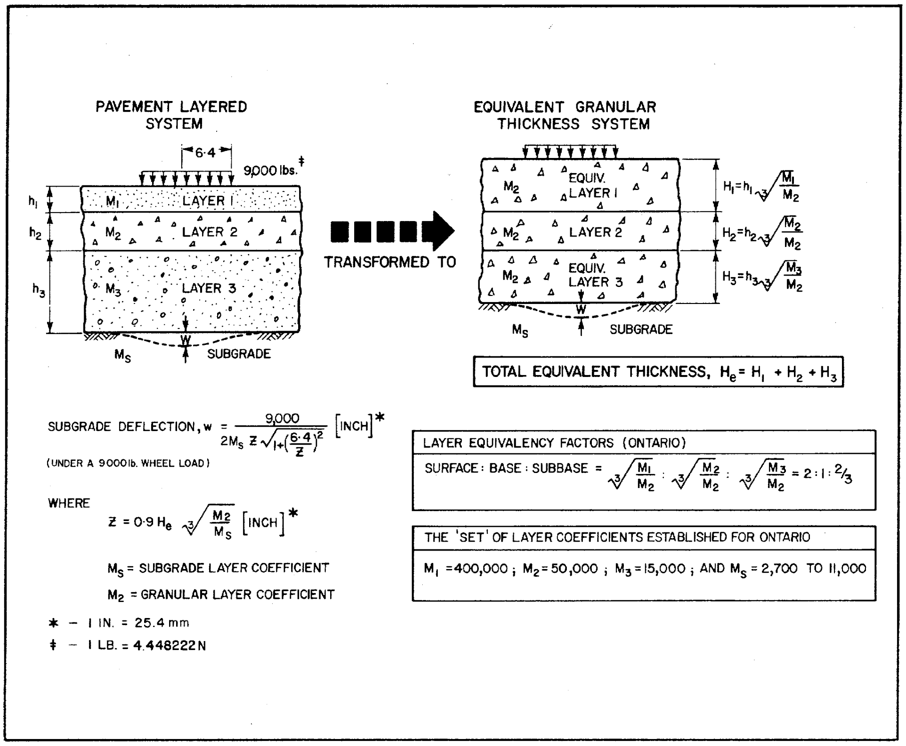


Figure 2. Transformation of Pavement Layered System into Equivalent Granular System, and Calculation of Subgrade Deflection.

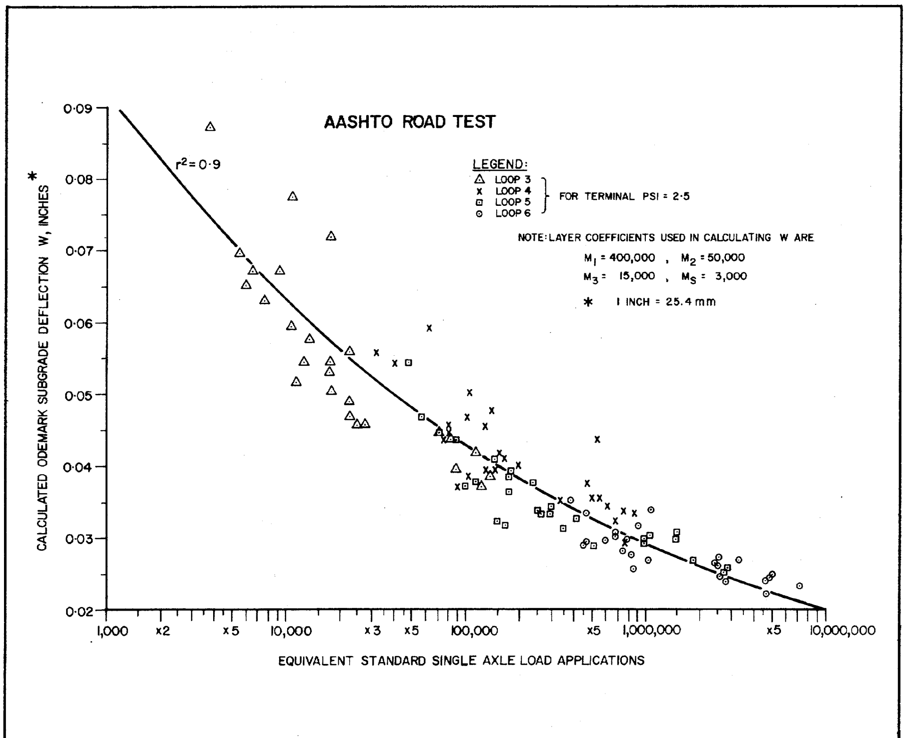


Figure 3. Calculated Odemark Subgrade Deflection Versus Load Repetition at the AASHTO Road Test (After Reference 5).

index [Riding Comfort Index (RCI)]. The modified equation is given below and is plotted in Figure 7.

$$p_T = 2.4455\ \psi + 8.805\ \psi^3$$

where:

- ψ = $1000\ w^6\ \mathrm{N}$ *
- p_T = loss in Riding Comfort Index due to traffic
- w = calculated Odemark subgrade deflection in inches
- N = number of standard (18-kip or 80 kN) single axle load applications

It should be noted that AASHTO Road Test data did not include sections giving Odemark subgrade deflections of less than 0.025 inches (0.635 mm)(i.e. very thick pavements), however, the model can be extrapolated for designs in this range.

C. Environment Related Deterioration (Figure 1, Step 4b). Since the AASHTO Road Test was carried out with accelerated loading and relatively under-designed pavement thicknesses, results are not readily applicable to real-life pavements even if such pavements are influenced by similar geographical and environmental conditions. Due to exposure to climatic cycles, the real-life pavements suffer deterioration over a period of time, even under low traffic volumes, due to differential heaving and settling from frost, freeze-thaw cycles, swelling subgrades or other such influences. Since the AASHTO Road Test sections survived only one or two winter seasons, the test results do not adequately indicate the effect of these influences. This became apparent in Ontario when the traffic related performance model derived from AASHTO Road Test data was applied to the Brampton Road Test over a period of eight years of its performance. The traffic performance model could not reproduce the Brampton results even by applying a proportionality factor and the characteristics of the difference suggested a different kind of functional relationship.

A flexible pavement attains its highest level of serviceability of riding comfort immediately after construction. During its lifetime, the pavement is exposed to forces from axle weights, temperature changes which cause restrained expansion or contraction, and changes of subgrade conditions which cause swelling, shrinkage, freezing—thawing, and heaving—settlement. These forces or influences act in various combinations and proportions on a pavement, causing loss in Riding Comfort Index.

* $\psi = 3.7238 \times 10^{-6}\ w^6\ \mathrm{N}$
where w is in millimetres

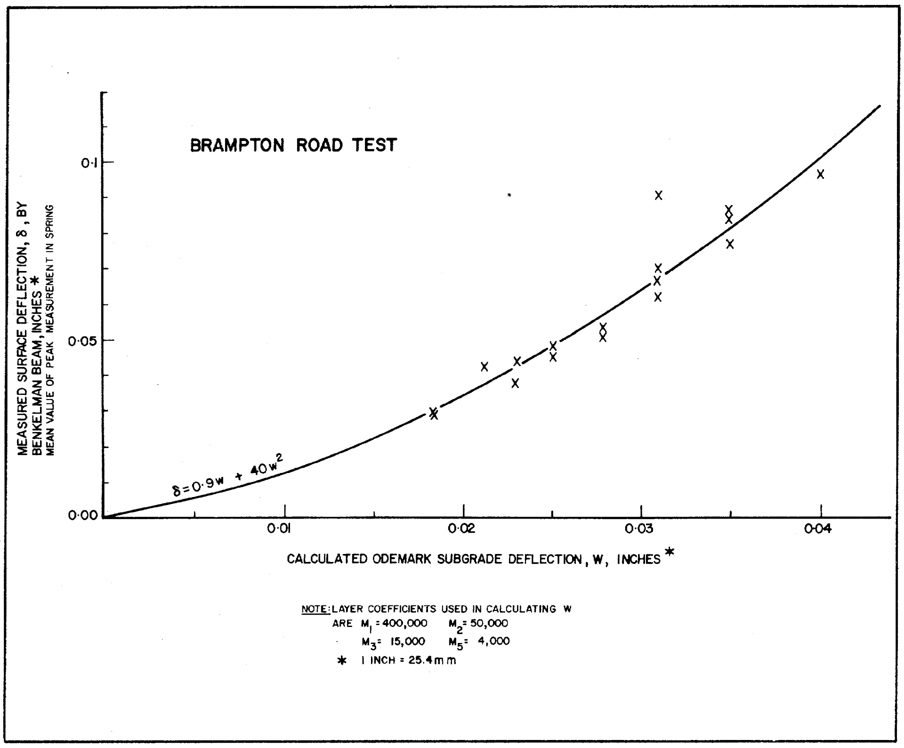


Figure 4. Calculated Odemark Subgrade Deflection Versus Largest Benkelman Beam Deflection in Spring, Brampton Road Test Sections.

The combination of axle weights with soft subgrade conditions in the spring, and the upward migration of water in the subsoils due to freezing (which results in heave or upward expansion in conjunction with subsequent re-consolidation) are noteworthy distress mechanisms. The influence of spring softening observed in the AASHTO Road Test was the reason why the number of axle repetitions were "weighted". The effect of water migration and freezing is a function of the increasing number of years through which a pavement survives. Also, deterioration due to temperature changes (such as transverse cracking) is a function of the number of winter-spring seasons. In light of this time-dependent part of pavement deterioration, it becomes necessary that a loss model, in addition to a traffic loss model, should be developed as a function of number of years or winter-spring cycles.

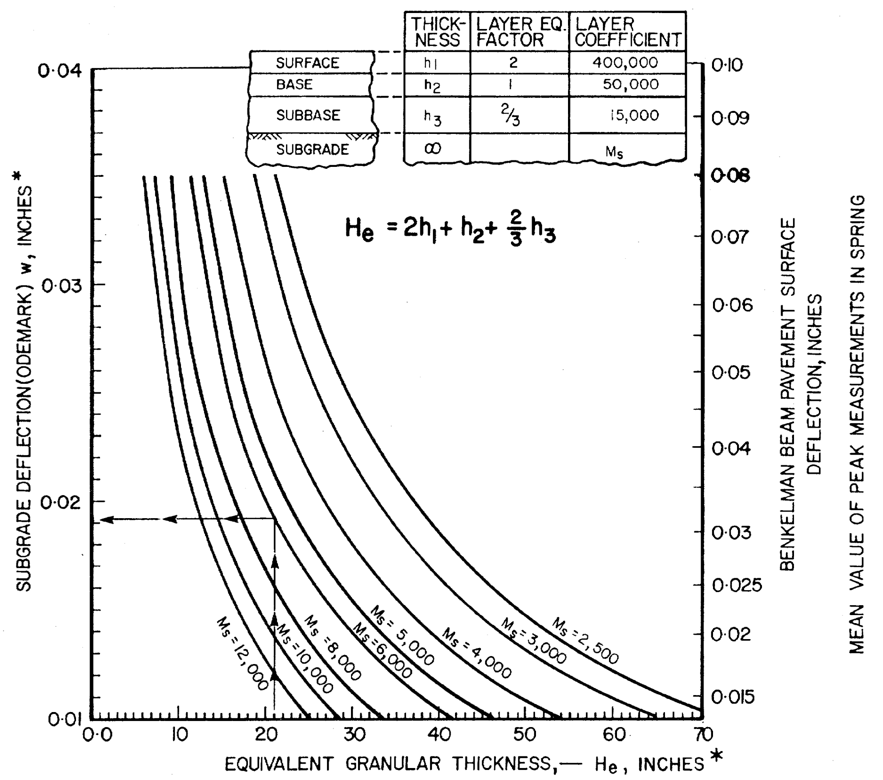
The function which best represents this additional loss term is conceived to have the specific characteristic of decreasing exponentially with each year or season passed. A similar function has been suggested by Kher et al [12] in the development of pavement performance equations for swelling clay subgrades in Texas, a condition which produces similar volume changes as those produced by low-temperature frost effects. The function is shown in Figure 8 and shows that the rate of decrease in p due to environmental forces is at a maximum in the initial years and reduces with time as p reaches a hypothetical ultimate value of p_∞ at infinite time. In other words, the more a pavement deteriorates from its initial smoothness towards its ultimate roughness value of p_∞, the less is the annual rate of "roughening". For regions where a different type of environmental deterioration is observed, the loss function may assume a different form such as, for example, a linear relationship which should be used where the same average amount of environmental deterioration is observed in each year of pavement life.

It is also conceived that the level of p_∞ is larger for stronger pavements, i.e. stronger (thicker) pavements will be less affected by environmental deterioration than weaker pavements. Since the strength of a pavement has been established in the previous section by the subgrade surface deflection criterion, the same response value can be used to establish the resistance of pavements to environmental forces. In other words, p_∞ can be made a function of the Odemark subgrade deflection.

To establish the constants of the exponential deterioration function, it is necessary to obtain experimental data or other experience on real pavements in a particular region. In Ontario, the Brampton Road Test was used for this purpose. The methodology adopted to establish this function is described below. However, it is obvious that each region must rely on local experience to obtain a relevant function with different constants.

For each section of the Brampton Road Test, the best fit curves were drawn through the observed data as well as the curve predicted by the traffic performance model. The two curves did not coincide due to the differences contributed by the environmental factors. When the differences were examined for all the sections of the Brampton Road Test, the following three inferences were drawn:

1/Environmental deterioration is an exponential curve of RCI versus the number of years;
2/Exponential deterioration rate α equal to 0.06 applies to all sections of the Brampton Road Test; and



TYPICAL SUBGRADE LAYER COEFFICIENTS M_S IN ONTARIO

SUBGRADE / CONDITION	GRAN. TYPE MATERIALS SUITABLE AS GRAN. BORROW	SANDY SILT AND CLAY LOAM TILL: SILT<40 V.F. Sa and Si <45	SILT 40-50 V.F. Sa and Si 45-60	SILT>50 V.F. Sa and Si >60	LACUSTRINE CLAYS	VARVED AND LEDA CLAYS
GOOD	11,500	7,000	6,000	4,500	5,500	4,500
FAIR	10,500	6,000	5,000	4,000	5,000	3,500
POOR	9,000	5,500	4,500	3,500	4,000	2,500

* 1 INCH = 25.4 mm

Figure 5. Odemark Subgrade Surface Deflections for Different Equivalent Granular Thicknesses and Subgrade Layer Coefficients.

3/The asymptotic value of p_∞ varies for each section and correlates highly with the strength of the section represented by the Odemark subgrade deflection, w. The correlation is expressed as follows:

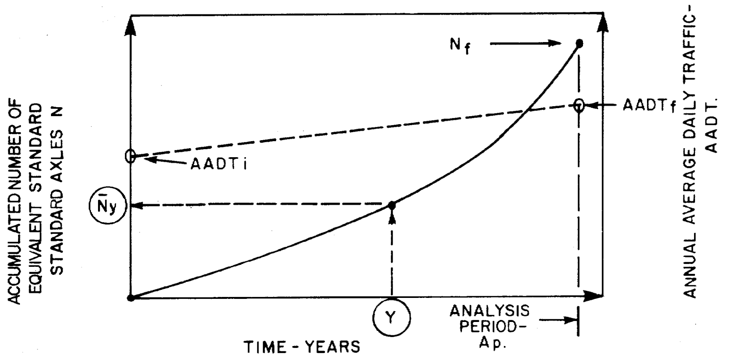
$$p_\infty = \frac{p_o}{1 + Bw}$$

where

p_o = Initial Riding Comfort Index of pavement

B = 100

Figure 9 gives the basic performance data on some of the Brampton Road Test sections, ranging from very weak to very strong pavements. The curves calculated by the equations of the final model which combine deterioration resulting from both traffic and environmental influences (using the above constants) are also shown. These curves demonstrate that the final model predicts the riding comfort measurements of the Brampton Road Test over the entire range of conventional pavements



$$N = \frac{N_f}{A_p}\left[\frac{2AADT_i}{(AADT_i + AADT_f)}Y + \frac{AADT_f - AADT_i}{A_p\,(AADT_i + AADT_f)}Y^2\right] \quad (1)$$

$$N_f = \frac{A_p}{2}\left\{\left(\frac{AADT_i}{2} \times DAYS \times T_i \times LDF_i \times TF_i\right) + \left(\frac{AADT_f}{2} \times DAYS \times LDF_f \times TF_f\right)\right\} \quad (2)$$

WHERE T = PERCENT OF TRUCKS

$\frac{AADT}{2}$ = ONE DIRECTIONAL AVERAGE ANNUAL DAILY TRAFFIC

LDF = LANE DISTRIBUTION FACTOR (1.0 FOR TWO LANE ROADS; 0.8 FOR MOST FOUR LANE ROADS)

TF = TRUCK FACTOR = EQUIVALENT STANDARD AXLES PER TRUCK (18 kip or 80 kN)

DAYS = No. OF DAYS PER YEAR FOR TRUCK TRAFFIC (GENERALLY = 300)

i AND f DENOTE "INITIAL" AND "FINAL" RESPECTIVELY

Figure 6. Traffic Input Model.

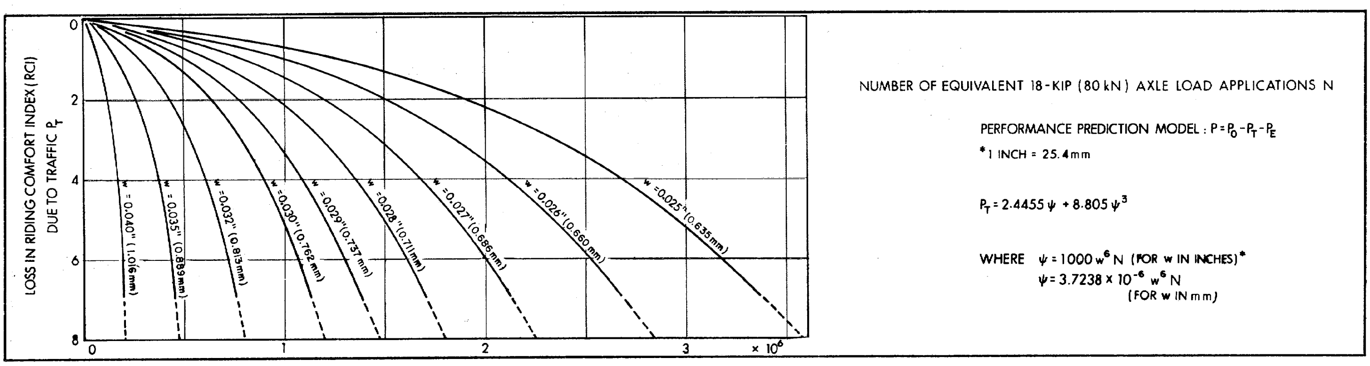


Figure 7. Performance Prediction Model — Traffic Related Contribution (P_t).

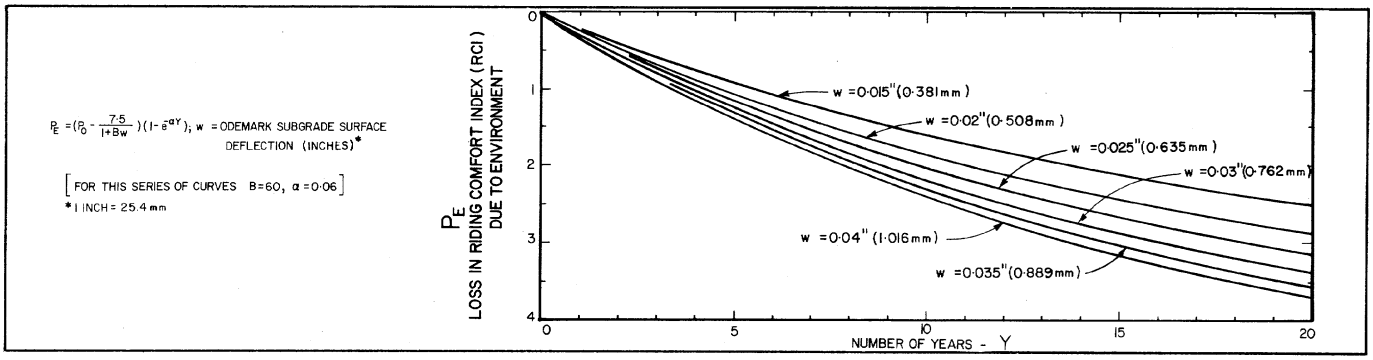
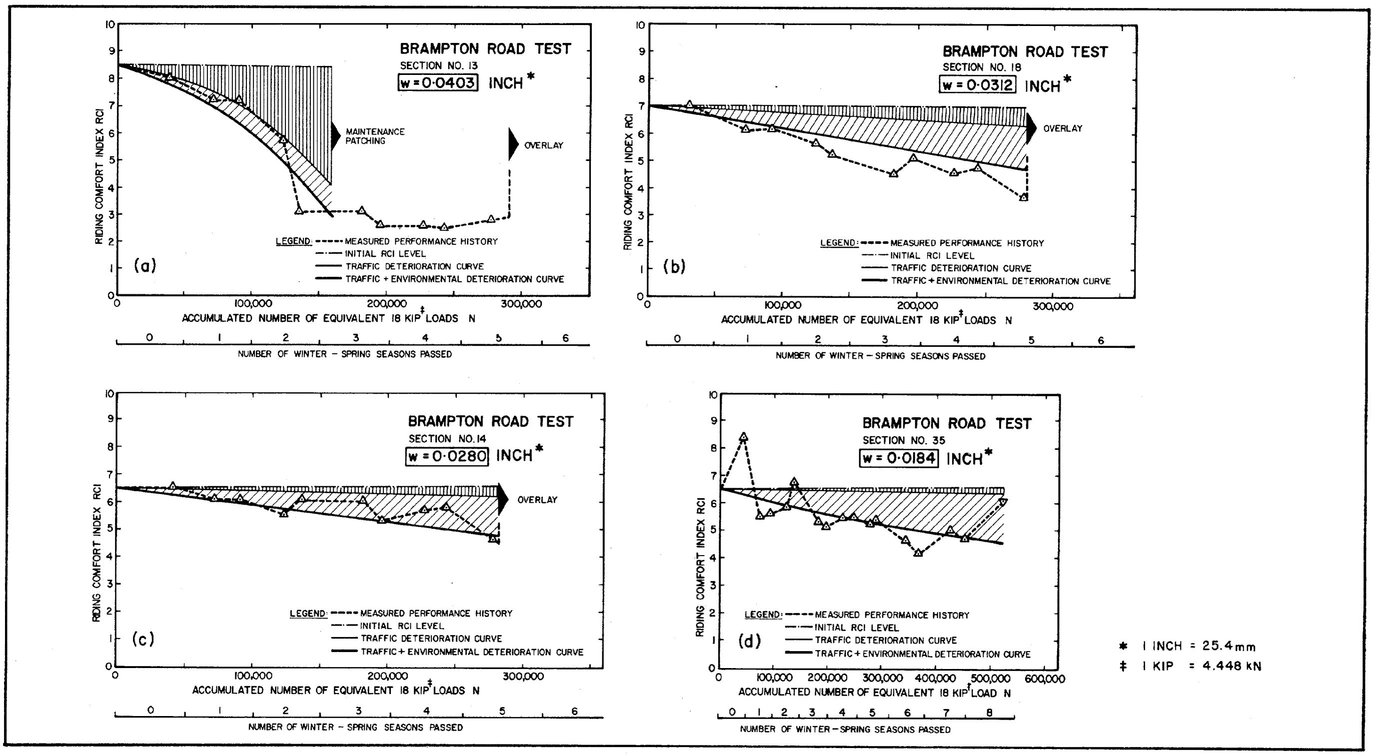


Figure 8. Performance Prediction Model – Environmental Related Contribution.

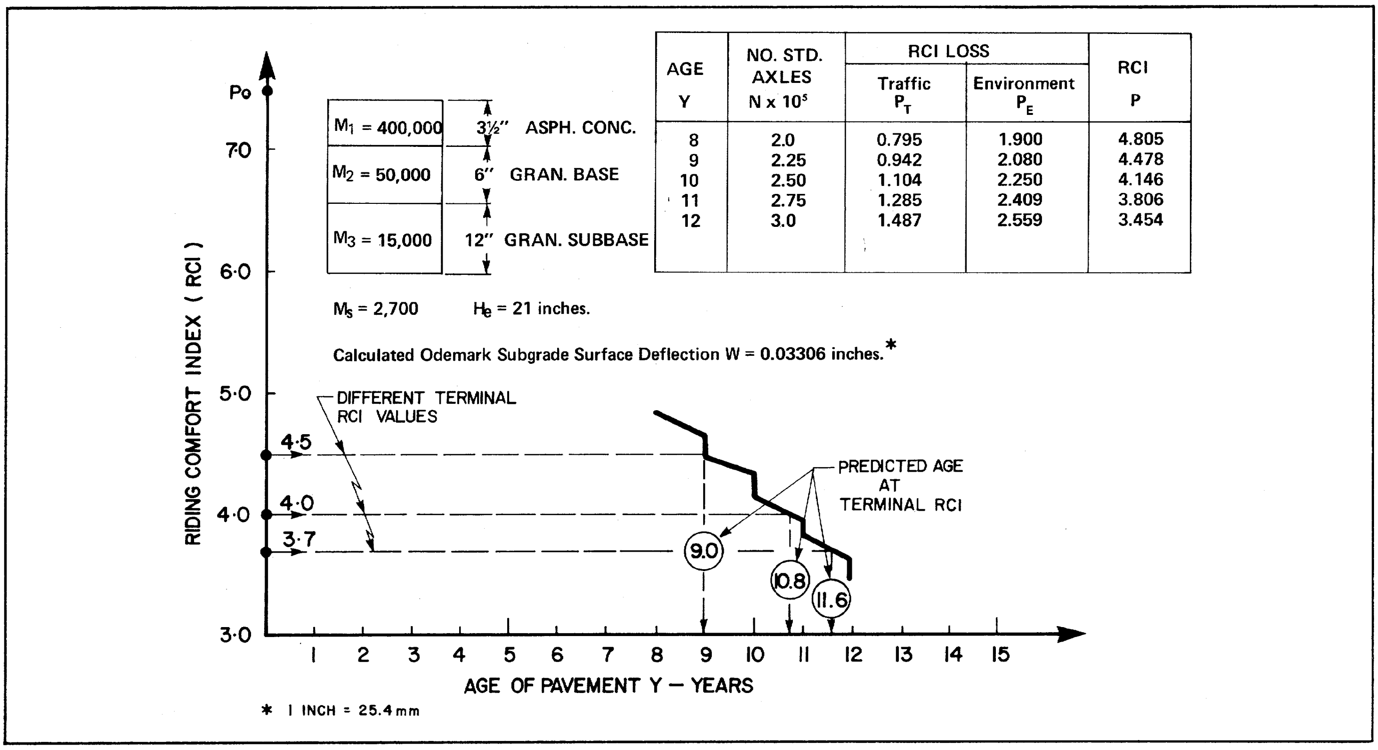
tested. Also shown in these figures are the curves resulting from only the traffic deterioration model.

Data on the values of RCI taken immediately after construction indicated that average p_o for the pavements can be assumed to be equal to 7.5. With p_o at a constant value, the model was tested on a variety of past pavement design experiences in the province. The analysis showed that for B equal to 60, pavement lives matched the province-wide experience.

Figure 9. Brampton Road Test Performance Data Versus the Predicted Performance Using the Model Developed (Including Traffic Plus Environmental Deterioration).



4 Example of Application of the Model



AGE Y	NO. STD. AXLES N x 10^5	RCI LOSS Traffic P_T	RCI LOSS Environment P_E	RCI P
8	2.0	0.795	1.900	4.805
9	2.25	0.942	2.080	4.478
10	2.50	1.104	2.250	4.146
11	2.75	1.285	2.409	3.806
12	3.0	1.487	2.559	3.454

Figure 10. Example Showing Predicted Performance Curve and Predicted Ages at Different Terminal RCI Values.

4

The following example is presented to demonstrate how the effects of traffic and environmental loss in RCI are combined and used to predict the age at which terminal performance is attained.

1/Assume a pavement with a 3½-inch (88.9 mm) asphalt surfacing, a 6-inch (152.4 mm) granular base and a 12-inch (304.8 mm) granular subbase constructed on a soft clay subgrade.

2/The estimated number of standard 18-kip (80 kN) axle load repetitions at 12 years is 300,000.

The first step in the procedure is to calculate the total granular equivalent thickness H_e of the pavement structure. This is $(3\frac{1}{2} \times 2) + (6 \times 1) + (12 \times 2/3) = 21$ inches (533.4 mm) when appropriate layer equivalency values are applied to each layer

For the second step, the subgrade layer coefficient which is selected from the table in Figure 5 as being consistent with the description of the material is $M_s = 2{,}700$. The Odemark subgrade surface deflection w for this pavement under the standard 9-kip (40 kN) wheel load P (contact radius, $a = 6.4$ inches (162.6 mm) is either calculated by:

$$w = \frac{9000}{2\,M_s\,Z} \times \frac{1}{\sqrt{1 + \left(\frac{a}{z}\right)^2}}$$

where

$$Z = 0.9\,\sqrt[3]{\frac{M_2}{M_s}} \times H_e$$

or, may be read directly from the graph in Figure 5.

For this example $M_s = 2{,}700$; $M_2 = 50{,}000$ (adopted for the subsystem and $H_e = 21$ inches (533.4 mm). Z is calculated as 50 inches (1270 mm) and w is calculated as 0.03306 inches (0.84 mm).

For the third step, the number of standard axles N at any time Y must be known. For the example N = 300,000 at Y = 12 years. The loss in RCI, p_T, due to traffic is determined from the following equations:

$$p_T = 2.4455\,\psi + 8.805\,\psi^3$$

where:

$$\psi = 1000\,w^6 \times N\ *$$

In this example $\psi = 0.3917$ and $p_T = 1.487$. The RCI loss may also be read from the curves in Figure 7.

* $\psi = 3.7238 \times 10^{-6}\,w^6\,N$ where w is in millimetres

5 Overlays

For the fourth step, the loss in RCI due to environmental influences, p_E, is calculated by the following formula:

$$p_E = (p_0 - p_\infty)(1 - e^{-\alpha Y})$$

where:

$$p_\infty = \frac{A}{1 + Bw}$$

For this example, as is the case with most new construction, the initial RCI value p_0 is taken as 7.5. The constants in these equations are taken as:

$p_0 = 7.5 = A$
$B = 60$
$\alpha = 0.06$

} use of these values for southern Ontario appears to predict lives which fit experience

For this example $p_\infty = 2.514$ and $p_E = 2.559$. This may also be found from Figure 8.

For the next step, the RCI losses p_T and p_E are deducted from the initial RCI (p_0) to determine the RCI at 12 years, or $p_{12} = p_0 - p_T - p_E = 7.5 - 1.487 - 2.559 = 3.454$. This predicted RCI at 12 years forms one point on the performance curve for this pavement structure. Other points are similarly calculated for different times Y and their corresponding traffic loads N so that the performance curve may be defined. When this is done as is illustrated in Figure 10, the age of the pavement may be determined for any value of RCI which is considered to be the terminal value or the value at which rehabilitation becomes necessary. For example, at a terminal RCI of 4.0, the corresponding predicted age taken from the performance curve is 10.8 years.

5

Repeated traffic loads and environmental influences result in gradual deterioration of pavement layers. Cracks develop in the asphaltic layers and increase in number over the years; concurrently, the granular layers are weakened by various mechanisms such as aggregate degradation and contamination.

Thus, although the life of a newly constructed pavement is predicted based on its initial value of calculated subgrade deflection w, this deflection value does not remain constant but increases as the strength of the pavement layers decreases. This increase in deflection was observed at the Brampton Road Test [13] and demonstrated by Lister in his observations on deflection of more than 300 experimental sections [4]. For the design of an overlay, the reduction in the strength of the existing layers can be accounted for by reducing their layer equivalency factors. The reduction coefficients which are used at present in the subsystem are given in Figure 11.

The premise underlying the use of layer equivalency reduction coefficient K_1 for asphaltic layers is that repeated loading decreases stiffness [14] thus, the coefficient K_1 decreases as RCI decreases to values slightly below that of poor gravel. A straight line relationship was considered most suitable. Figure 11 similarly presents layer equivalency reduction coefficients K_2 and K_3 for granular base and subbase materials. The coefficients are larger for stronger pavements because such pavements are designed to withstand heavier traffic loads and/or more seasonal cycles. The layer equivalency reduction coefficients, while based on limited data, serve not only as indicators of how the modelling of the overlay problem is approached, but their usage results in overlay lives which are acceptable to practicing design engineers in Ontario.

An overlay rehabilitates the pavement in two respects. First, it raises the riding quality to a more comfortable level (see Figure 12) and, secondly, it re-establishes the strength of the pavement. As a general guideline, overlay thickness provided should compensate for some of the loss in strength of the old pavement so that the total granular equivalent thickness after the overlay is comparable with the thickness of the initial construction. The equivalent thickness after overlay is determined as follows:

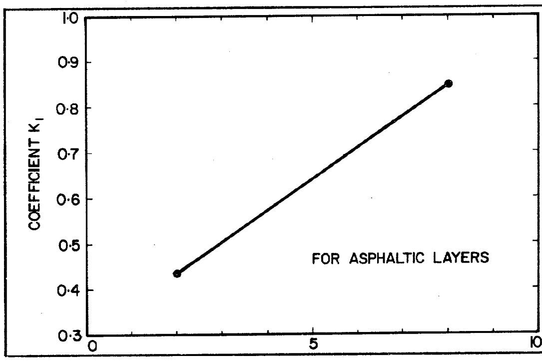
$$H_e = a_1 h_0 + k_1 a_1 h_1 + k_2 a_2 h_2 + k_3 a_3 h_3$$

where:

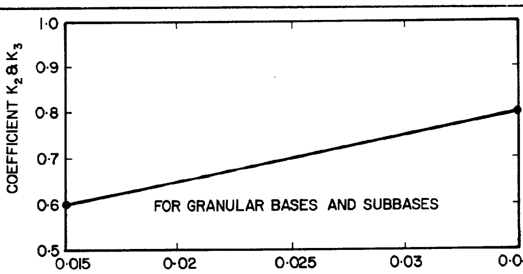
h_0 = overlay thickness
a_1, a_2, a_3 = layer equivalency factors for asphalt surfacing, base and subbase, respectively
h_1, h_2, h_3 = layer thicknesses of surfacing, base and subbase of initial pavement, respectively
k_1, k_2, k_3 = layer equivalency reduction factors (Figure 11) for surfacing, base and subbase, respectively

To predict the life span of an overlaid pavement, the new equivalent thickness value replaces the initial equivalent thickness value and performance prediction procedure is repeated.

Figure 11. Reduction Coefficient for Layer Equivalency Factor.



RIDING COMFORT INDEX P_T
TERMINAL VALUE BEFORE OVERLAY



ODEMARK SUBGRADE DEFLECTION W IN INCHES *
AFTER INITIAL CONSTRUCTION
* 1 INCH = 25.4mm

6 Concluding Remarks

6

A model has been described which predicts the performance life of a pavement structure. Alternatively, if several pavement structures are proposed for a facility, the most suitable may be selected based on performance life and cost implications.

The model has been formulated such that local experience gained from successful highway pavements can be used. Basically, the performance of a pavement has been considered to be affected by two major distress mechanisms: traffic loads and environmental effects. Traffic-associated distress has been derived from the AASHTO Road Test, which is primarily a traffic-oriented test. Environment-associated distress has been derived from the Brampton Road Test: a test which gives the long-term effects of traffic and the environment. The two tests have been analyzed with reference to past pavement design experience which has been given prime consideration in the curve-fitting process.

The model has undergone considerable testing with its application for designing pavement structures throughout the province. The lives predicted by the model are found to be in close agreement with the general pavement performance experience of the design engineers.

As an empirical formulation, the model contains significant "lack of fit" error due to the very nature of pavement performance data which inherently varies even among identical pavement sections. The "lack of fit" generally implies that more variables affect pavement performance than considered in the study; however, the model does predict average life expectancy and the "lack of fit" is even smaller than that found on the AASHTO Road Test. In the range of extremely low deflections (i.e. extremely thick pavement structures), AASHTO Road Test data did not exist. However, the use of the extrapolated curve outside of the AASHTO range is considered appropriate at the present time.

The procedure described in this paper can be used to develop performance models for other regions or provinces.

Since pavement deterioration which is generated by traffic alone is less significant than that caused by environment, the traffic deterioration model as presented in this paper can be adapted to other regions. Research effort can then be concentrated on updating the environmental model based on local experience.

The model is by no means final and updating of the submodels, such as the environmental deterioration model, is continuing as more data become available.

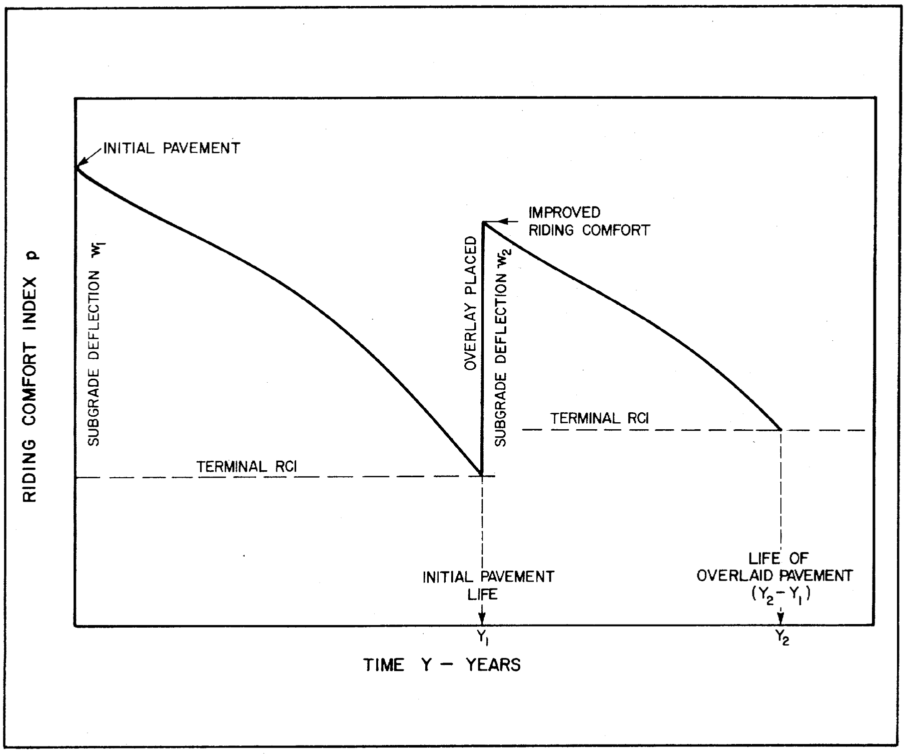


Figure 12. Overlay Performance Concept.

7 References

[1] *The AASHTO Road Test*, Report 5, Pavement Research Publication, No. 954, Transportation Research Board, Washington, D.C., U.S.A., 1962.

[2] W. A. Phang, *Four Years' Experience at the Brampton Road Test*, Research Report 153, Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1969, and *Highway Research Record No. 311*, Transportation Research Board, Washington, D.C., U.S.A., 1970, pp. 68-90.

[3] W. A. Phang and R. Slocum, *Pavement Decision Making and Management System*, Research Report 174, Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1971.

[4] N. W. Lister, "Deflection Criteria for Flexible Pavements and the Design of Overlays," proceedings of 3rd. Int. Conf. on the Structural Design of Flexible Pavements, London, England, 1972.

[5] F. W. Jung and W. A. Phang, *Elastic Layer Analysis Related to Performance in Flexible Pavement Design*, Research Report 191, Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1974, or "Elastic Layer Analysis Related to Performance in Flexible Pavement Design," Transportation Research Record No. 521, Transportation Research Board, Washington, D.C., U.S.A., 1974, pp. 14-29.

[6] F. H. Scrivner et al, *A Systems Approach to the Flexible Pavement Design Problem*, Research Report 32-11, Texas Transportation Institute, Texas A & M University, Texas, U.S.A., 1968-1969.

[7] F. H. Scrivner et al, *Flexible Pavement Performance Related to Deflections, Axle Applications, Temperature and Foundation Movements*, Research Report 32-13, Texas Transportation Institute, Texas A & M University, Texas, U.S.A., 1968-1969.

[8] N. Odemark, *Investigations as to the Elastic Properties of Soils and Design of Pavements According to the Theory of Elasticity*, Statens Vaeginstitut, Stockholm, Sweden, 1949.

[9] N. I. Kamel, J. Morris, R. C. G. Haas and W. A. Phang, "Layer Analysis of the Brampton Test Road and Application to Pavement Design," *Highway Research Record No. 466*, Transportation Research Board, Washington, D.C., U.S.A., 1973, pp. 113-126.

[10] *A Guide to Structural Design of Flexible Pavements in Canada*, CGRA (Canadian Good Roads Association), Pavement Design and Evaluation Committee, September 1965.

[11] *Evaluation of AASHTO Interim Guides for Design of Pavement Structures*, NCHRP Report 128, Transportation Research Board, Washington, D.C., U.S.A., 1972.

[12] R. K. Kher, W. R. Hudson and B. F. McCullough, "A Working Systems Model for Rigid Pavement Design," *Highway Research Record No. 407*, Transportation Research Board, Washington, D.C., U.S.A., 1972.

[13] W. A. Phang, *The Effect of Seasonal Strength Variation on the Performance of Selected Base Materials*, Internal Report 39, Ministry of Transportation and Communications, Downsview, Ontario, Canada, April 1971.

[14] B. F. Kallas and V. P. Puzinauskas, "Flexure Fatigue Tests on Asphalt Paving Mixtures," Fatigue of Compacted Bituminous Aggregate Mixtures, ASTM STP 508, American Society for Testing and Materials, 1972, pp. 47-65.