LAYER ANALYSIS OF THE BRAMPTON TEST ROAD AND APPLICATION TO PAVEMENT DESIGN

Nabil I. Kamel and W. Phang, Ministry of Transportation and Communications, Ontario; and
Jack Morris and R. C. G. Haas, University of Waterloo, Ontario

Considerable attention has been devoted, during the past decade, to developing a pavement design system based on structural analysis concepts. The implementation of a design system requires a structural model that will calculate the serviceability-age history of various alternative strategies. Consequently, it is essential to relate the structural analysis outputs of stresses, strains, and deflections to the subsequent performance of the pavement under different traffic and environmental conditions. One objective of a recent study of the Brampton Test Road was to develop these relations for conditions in southern Ontario. BISTRO and CHEVRON computer programs were used in iterative procedures to calculate pavement structural response of stresses, strains, and deflections of each of the Brampton Test Road sections. The moduli values required for the analysis were derived from laboratory test data provided by The Asphalt Institute. These calculated structural responses were then correlated to observed pavement behavior and performance measurements of the test sections. Charts to predict serviceability losses as a function of traffic, type of base material, and vertical stress level on subgrade surface are developed. A good relation is found between the measured Benkelman beam rebounds and calculated subgrade surface deflection. Equivalencies of various types of base materials are developed based on equal loss of serviceability. Finally, the paper presents a structural design system for flexible pavements based on use of layered analysis techniques. It also discusses, through an example problem, how this fits into overall pavement management systems.

•THE development of comprehensive pavement design and management systems has received considerable attention in Canada and the United States during the past few years (1-12). Currently, authorities in Texas, Ontario, and other jurisdictions are attempting to implement working systems of pavement management (1-3).

A major component of any pavement management system is the structural design phase. Here, the primary outputs that must be predicted by the designer are performance (i.e., serviceability-age history) and the associated cost and benefit implications, as shown in Figure 1. The structural analysis outputs of stress, strain, and deformation must then be linked with serviceability history. Today, a fundamental method of predicting pavement performance under different traffic and environmental conditions is needed.

In southern Ontario, the Brampton Test Road was constructed in 1965 primarily to compare the performance of a variety of base materials. Extensive data on material properties, traffic, climate, and performance were available (13-17). Hence a unique opportunity was provided to relate structural response and performance in a controlled experiment over a period of time. As a result, an investigation (18, 19) was undertake

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

to determine whether structural analysis could be used to predict the serviceability-age histories of the various pavement designs used at Brampton Test Road. The purpose of this paper is to report the findings of this investigation. More specifically it reports the following:

1. The basic material characteristics required for a structural method of analysis;
2. The analysis of stresses, strains, and deflections for the Brampton test sections employing computerized, non-linear-elastic layered techniques;
3. The relations between the structural responses obtained in step 2 and the observed behavior and performance;
4. The establishment of equivalencies for the Brampton materials, based on the criterion of equal terminal serviceability; and
5. The application of the results obtained in steps 3 and 4 to a pavement design and management system.

STRUCTURAL ANALYSIS OF THE BRAMPTON TEST ROAD

Structural Analysis Model

Since the 1960 AASHO Road Test, pavement researchers have devoted considerable attention to developing a "rational" method of pavement design based on stresses, strains, and deflections throughout the pavement system. Ideally, the constitutive equations of the materials should be determined and the pavement modeled accordingly. Up to the present time, it has not been possible to follow this approach because of the complex nature of the pavement structure. The current situation has been summarized by Morris, Kamel, and Haas (20). It is concluded that a non-linear-elastic layered solution is the most convenient and reliable means of obtaining the structural responses within a pavement system. In this study, CHEVRON and BISTRO multilayered elastic computer programs were employed, and iterative procedures were used to account for the nonlinear characteristics of the materials.

Material's Characterization

The Brampton Test Road was constructed during August and September 1965 as a full-scale experiment. Details of the project construction, objectives, and principal findings to date are presented elsewhere (13-15). The experiment consists of 36 test sections and incorporates five types of base as shown in Figure 2.

Repeated load triaxial compression tests on the unbound and stabilized materials were conducted by The Asphalt Institute (16, 17). During the testing, appropriate ranges of temperature, stress, moisture content, density, etc., were selected to simulate the field conditions.

The test sections composed of cement-treated bases suffered from shrinkage fracture early in the experiment and were not considered in this investigation.

The elastic response of unbound and emulsion stabilized materials can be described by the resilient modulus, M_R (21, 22), the ratio of the repeated deviator stress to the rebound or recoverable axial strain. The characteristics of the unbound materials based on The Asphalt Institute's test are shown in Figure 3. In keeping with previous research workers (23, 24), M_R is shown as a function of deviator stress, σ_d, for the clay subgrade and of confining stress, σ_3, in the case of the granular materials. The results of the repeated load triaxial tests on the bituminous stabilized sand at three test temperatures are shown in Figure 4.

The response of an asphaltic material is generally described by the stiffness modulus, S, as defined by Van der Poel (25). This parameter can be obtained either by "direct" means from laboratory measurements or by "indirect" methods based on Van der Poel's nomograph and its modifications (9). For this study, indirect estimation of stiffness was employed using McLeod's modified method (26). The stiffness-temperature relations obtained, based on a loading time of 0.03 sec, which corresponds to traffic moving at 60 mph (27), are shown in Figure 5.

Asphalts are thermoplastic materials; i.e., their stress and strain responses under load are time- and temperature-dependent. Consequently, the structural response of a

Figure 1. Gross output of pavement system in terms of performance and value.

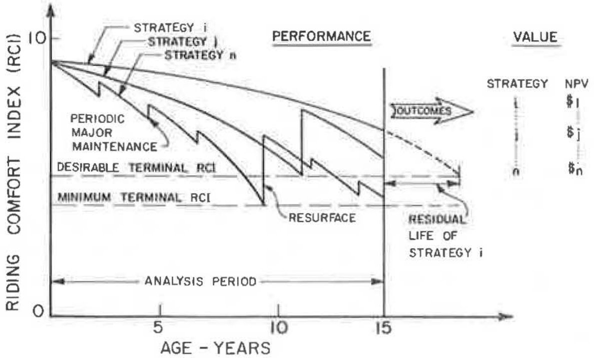
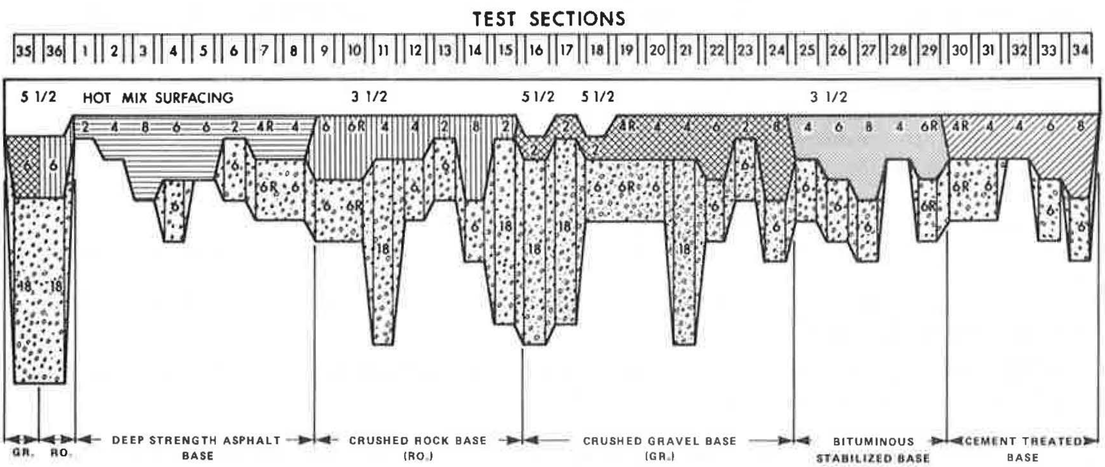


Figure 2. Layout of experimental pavement sections.



EACH TEST SECTION = 200 FT. EACH TRANSITION = 50 FT.

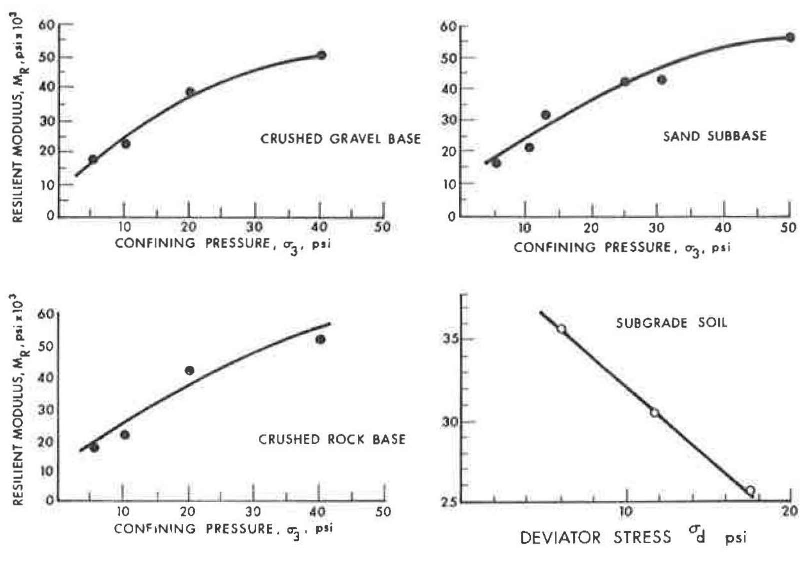
LEGEND

R. = REPLICATED (I.E. DUPLICATED DESIGN)

GR. = CRUSHED GRAVEL BASE

RO. = CRUSHED ROCK BASE

Figure 3. Resilient moduli of Brampton unbound materials.



pavement depends on the temperature distribution within the asphaltic concrete. In recent years, considerable effort has been devoted toward developing realistic models for the determination of temperature distribution within asphaltic concrete pavements (28-30). Southgate's approach (28), which appears to give reasonable results and is easy to apply, was adopted for this investigation.

An earlier study (15) indicated that the pavement of the Brampton Test Road is at its annual weakest condition in the month of June. The average maximum air temperature in June (70 F) was used to determine the temperatures at middepth of the asphaltic concrete surface, binder, and base courses. The corresponding stiffness modulus for each layer was then obtained from Figure 5.

Structural Analyses

The structural analyses of the Brampton test sections were performed primarily by means of the CHEVRON computer program. This approach assumes that each layer is composed of linear-elastic and homogeneous material; but, as shown in the previous section, the moduli of unbound and stabilized materials are stress-dependent. This difficulty can be overcome by using an iterative process as follows:

1. Assume moduli for the layers,
2. Calculate the horizontal and vertical stresses at the middepth of the base and subbase and those at the surface of the subgrade,
3. Use the calculated stresses in step 2 to get the corresponding M_R value for each layer using the material characterization curves shown in Figure 3,
4. Compare the assumed values of M_R in step 1 with the calculated values in step 3, and
5. Repeat the previous steps until the assumed and calculated values of the moduli are approximately equal.

CHEVRON does not provide subgrade strains, so BISTRO was employed for this calculation. The results are given in Table 1.

STRUCTURAL ANALYSIS-PAVEMENT PERFORMANCE RELATIONS

Pavement Serviceability and Performance

This section first considers pavement serviceability and performance in general terms and then considers the performance of the Brampton sections.

The best-known procedure for defining and obtaining serviceability was developed at the AASHO Road Test by Carey and Irick (10). Their concept of present serviceability rating (PSR) explicitly recognized the road user by means of a panel rating procedure. PSR was then correlated to a set of physical measurements called present serviceability index (PSI). The integration of PSI over time or load applications was termed the performance.

The Roads and Transportation Association of Canada (RTAC, formerly the Canadian Good Roads Association) concurrently developed the present performance rating (PPR) along similar lines except that it used a 10-point rather than a 5-point scale (31, 32). In 1968, the Pavement Management Committee of RTAC changed the term PPR to riding comfort index (RCI) (33). The new term recognized that serviceability is only an evaluation of riding comfort quality and does not include structural condition or safety characteristics. RCI is employed throughout this paper.

Attempts to model RCI in Canada have not been highly successful. At Brampton, RCI was estimated from correlations between RCI and a roughness index profilometer patterned after the British Road Research Laboratory design (34, 35). A typical set of RCI-time relations obtained by this method is shown in Figure 6.

RCI-Traffic-Vertical Stress on Subgrade Relations

The observed performance, in terms of annual RCI values, is related to calculated structural response, in particular the vertical stress on subgrade surface, σ_v. Vertical subgrade stress has been used as the principal structural response variable because

Figure 4. Resilient modulus-confining pressure relation for bituminous-stabilized sand.

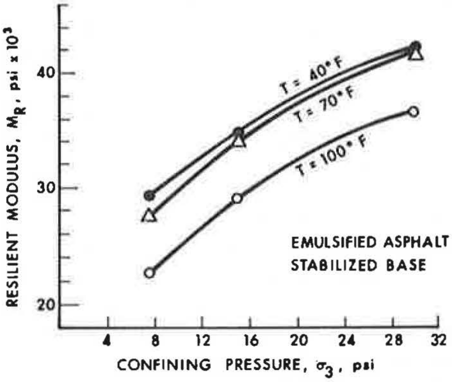


Figure 5. Asphalt concrete stiffness moduli-temperature relations.

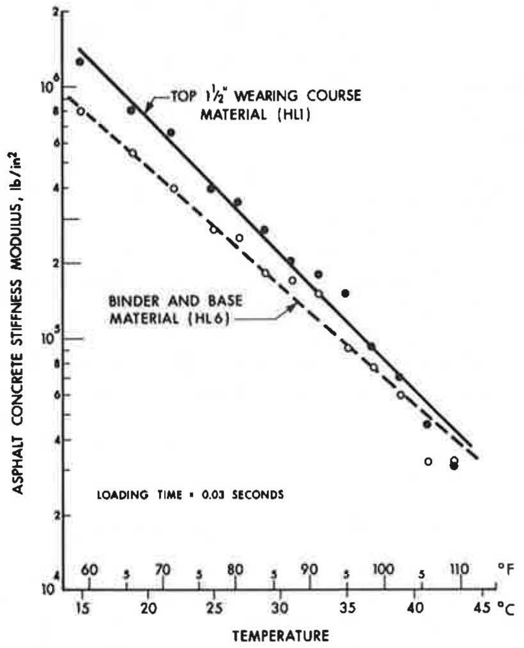


Table 1. Summary of results of layer analyses.

Section Number	Base	Subbase	Surface Deflection (in.)	Subgrade Deflection (in.)	Vertical Stresses on Subgrade Surface (psi)	Vertical Strain on Subgrade Surface (in./in. × 10^{-3})
1	Full-depth asphalt concrete	None	0.0183	0.0172	30.3	0.917
2			0.0134	0.0120	20.7	0.609
3			0.0096	0.0074	12.0	0.292
5			0.0104	0.0088	14.6	0.398
4	Deep asphalt concrete	6-in.	0.0103	0.0064	9.32	0.229
6			0.0172	0.0110	18.00	0 573
7, 8			0.0128	0.0083	12.90	0.352
9, 10	Crushed rock	6-in.	0.0241	0.0095	14.90	0.505
12			0.0226	0.0108	17.20	0.601
13			0.0240	0.0133	20.60	0.792
14			0.0236	0.0083	12.60	0.405
11	Crushed rock	18-in.	0.0249	0.0053	7.04	0.205
15			0.0232	0.0056	8.07	0.227
36			0.0211	0.0046	5.28	0.148
16	Crushed gravel	18-in.	0.0208	0.0053	6.71	0.187
17			0.0227	0.0056	8.04	0.225
21			0.0278	0.0056	7.30	0.214
35			0.0216	0.0047	5.37	0.150
18	Crushed gravel	6-in.	0.0181	0.0099	15.40	0.499
19, 20			0.0240	0.0109	17.50	0.629
22			0.0247	0.0090	14.70	0.580
24			0.0223	0.0080	12.50	0.384
25	Bituminous-stabilized	6-in.	0.0210	0.0102	16.70	0.561
26, 29			0.0211	0.0086	13.40	0.429
27			0.0200	0.0076	11.70	0.342
28	Bituminous-stabilized	None	0.0241	0.0189	28.60	1.230

Note: Using 9-kip single wheel load acting uniformly over a circular area of about 12 in. in diameter. Poisson's ratio of approximately 0.4 was assumed.

it is a good indicator of pavement load-carrying capacity; it also appears to give the best relations. The results indicate that the test sections can be classified into four groups: full-depth asphaltic concrete sections; asphalt concrete base sections that include subbase materials, i.e., deep-strength construction; granular base sections involving crushed-rock and crushed-gravel bases, i.e., conventional construction; and bituminous stabilized base sections.

Typical results, those obtained for the granular base sections, are shown in Figure 7. These RCI-σ_v-age relations can be drawn in a more meaningful form by plotting RCI-age (or traffic) curves for various levels of σ_v. The results for the four pavements are shown in Figure 8. The relations demonstrate the effect of σ_v on the serviceability history. It can be seen that RCI decreases more rapidly as the vertical stress on the subgrade surface increases. The rate of the decrease depends on the pavement type. The full-depth sections are the least sensitive to increasing in σ_v, and the bituminous stabilized base sections are the most sensitive.

These relations can be used to develop loss of performance curves for the various levels of σ_v. If normal scales are employed, good linear relations, as shown in Figure 9, are obtained. If the lines are extended to the left, then for each pavement type a different but relatively constant serviceability loss occurs at zero load applications. This loss is to be expected because of the effects of climate, foundation movements, and aging.

RCI-Traffic-Benkelman Beam Rebound Relations

The Benkelman beam has been used for many years as a means of measuring the in situ structural response of a pavement to load, and considerable technology has been developed on the basis of the method (1, 31). The Ontario pavement management system (1) employs structural design models that are derived principally from Benkelman beam experience.

Figure 10 shows the good correlations obtained between calculated subgrade surface deflections and measured initial mean peak Benkelman beam rebounds. Two distinct patterns emerge—one for the full-depth sections and the other for the remaining pavement types.

A complete description of this phase of the analysis is given elsewhere (36, 37). Briefly, a linear relation between log RCI and accumulated equivalent 18-kip single-axle loads for various Benkelman beam rebounds was found. Charts that predict performance loss as a function of traffic, type of base material, and pavement strength, as indicated by Benkelman beam rebound, have been developed (37). The type of model obtained in this analysis agrees with that obtained by Painter (38) when analyzing AASHO pavements of various strengths. It was also found that, for all pavement types other than the full-depth sections, sections with an initial peak rebound of 0.05 in. should lose 4.0 units of RCI after some 0.5 million applications of equivalent 18-kip single-axle loads. This result agrees with the findings of the CGRA after an extensive study of Canadian roads (31).

DEVELOPMENT OF BASE LAYER EQUIVALENCIES

The concept of layer equivalency is an integral part of design procedures currently employed by a number of agencies, including the Ministry of Transportation and Communications, Ontario. The RCI-age (or traffic)-vertical subgrade stress relations, previously developed, were therefore utilized as a basis for evaluating the equivalencies of various types of bases.

Basic Criterion

In the past, the equivalencies of various materials have generally been based on equal structural response (e.g., stresses, strains, deflections, etc.). Phang (15) for example established the equivalencies of the Brampton base materials on the basis of a peak Benkelman beam deflection of 0.05 in. Pavement deflection varies with age (1), and therefore equivalencies based on this criterion are also time-dependent. Further-

Figure 6. Time relations for typical Brampton RCI.

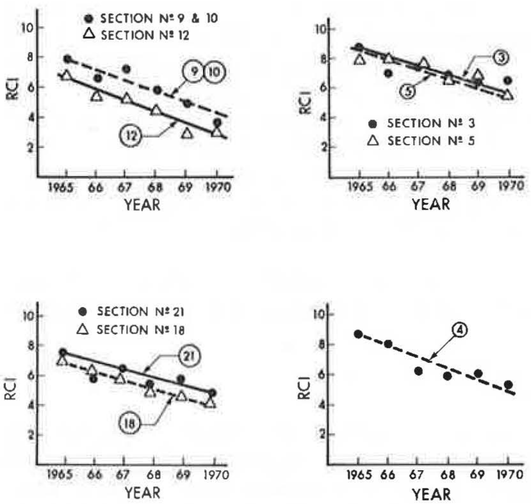


Figure 7. RCI and vertical compressive stresses on subgrade surface for crushed-rock and crushed-gravel base sections.

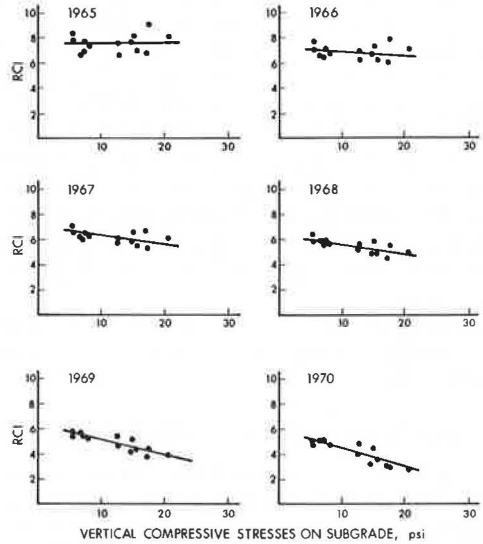
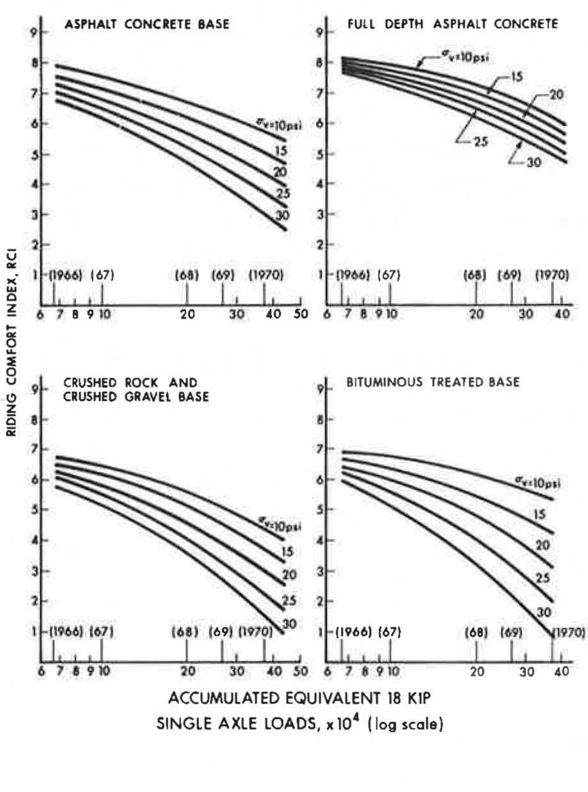


Figure 8. Performance history of Brampton base materials at various vertical stresses on subgrade surface.



more, pavement strength does not vary proportionately with thickness; therefore, if the deflection criterion is varied from 0.05 in. to, say, 0.02 or 0.10 in., significantly different equivalencies may be found.

In this study, the serviceability-age relations of the Brampton sections have been established in terms of the vertical stresses on the subgrade surface. Therefore, equivalencies can be developed on the basis of equal terminal serviceability. This criterion seems to be more realistic because pavement performance must be the end concern. Equivalencies based on this criterion enable the engineer to generate thickness combinations without having to consider the problem of strength differences.

Equivalencies based on terminal serviceability, however, may also vary depending on the level of terminal serviceability selected. The philosophy of varying the terminal RCI with the function or class of the pavement has been discussed in detail by Haas, Kamel, and Morris (37). Three terminal serviceability levels, 5.5 for major highways and freeways, 4.5 for secondary highways, and 3.5 for township roads, are recommended.

The data available at this time only permit the development of equivalencies for secondary highways, i.e., a terminal serviceability of 4.5 as represented by the Brampton Test Road.

Layer Equivalencies for Secondary Highways

A terminal serviceability of 4.5 has been recommended for secondary highways (37). The initial RCI of most newly constructed pavements lies between 8 and 9. If the initial RCI is assumed to be 8.5, then the basic criterion can be considered as a loss of RCI equal to 4.0. Figure 9 shows RCI loss-traffic relations for various stress levels and pavement types. A horizontal line corresponding to an RCI loss of 4.0 has been drawn, and the lines of different stress levels, σ_v, have been extended to intersect this basic criterion line. The points of intersection provide the relations between accumulated equivalent 18-kip single-axle loads and vertical stresses on the subgrade for a loss of 4.0 units of RCI. The results obtained for each type of base are shown in the upper portion of Figure 11. In constructing the relation for the bituminous stabilized base, a very large accumulated traffic number was found at the 10-psi level. This point resulted from considerable extrapolation and therefore was neglected when the curve was drawn. The lower part of Figure 11 shows the relation between stress level and equivalent base thickness (i.e., actual base thickness plus transformed subbase) for each pavement type. Subbase thicknesses have been converted into equivalent base thicknesses, using the following equivalencies suggested by Phang (14): 1 in. of subbase = 0.57 in. of granular base = 0.57 in. of bituminous stabilized base = 0.23 in. of asphalt concrete base.

The relations shown in Figure 11 are based on sections that have a constant thickness of asphalt concrete surface equal to $3\frac{1}{2}$ in.

Canadian studies (31) show that secondary pavements may reach a terminal RCI of 4.5 after 0.5 million applications of equivalent 18-kip single-axle loads. The Brampton Test Road provides similar evidence. If the upper portion of Figure 11 is entered at 0.5 million load applications, then the stress levels corresponding to 4.0 units of RCI loss can be determined. The example shown in Figure 11 indicates that a granular base pavement will lose 4.0 units of RCI if the subgrade surface is stressed to 7.5 psi. The lower portion of Figure 11 can then be used to determine the corresponding equivalent base thickness. For the example considered, 13.5 in. of base thickness is required to maintain a subgrade surface stress of 7.5 psi. The equivalent base thicknesses for other pavement types are 12.5 in. of bituminous stabilized base, 6.7 in. of asphalt concrete base (with subbase), and 4.0 in. of asphalt concrete (without subbase, i.e., full depth). The equivalencies based on these results are given in Table 2.

The values obtained are very close to those employed by the Ministry of Transportation and Communications of Ontario at the present time, except for the full-depth asphalt concrete. Here, the superior behavior of the asphaltic concrete base without subbase is reflected in a granular base equivalency of 3.4 compared to 2.0 when the subbase was included.

Figure 9. Loss of RCI for various base materials and vertical stresses on subgrade surface.

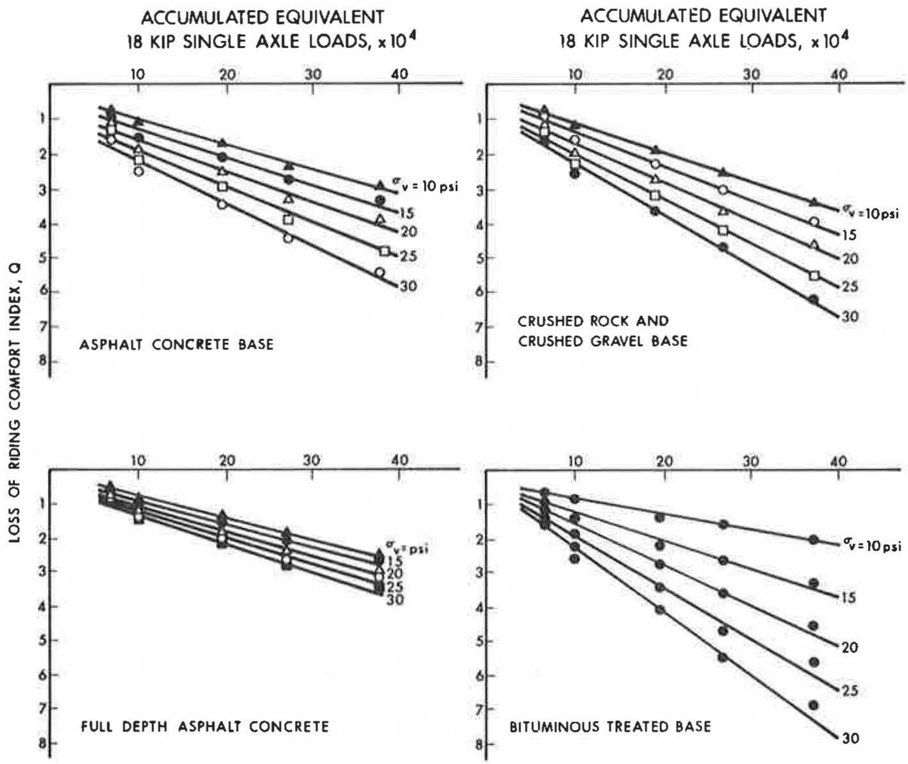


Figure 10. Calculated subgrade surface deflections and measured initial mean-peak Benkelman beam rebounds.

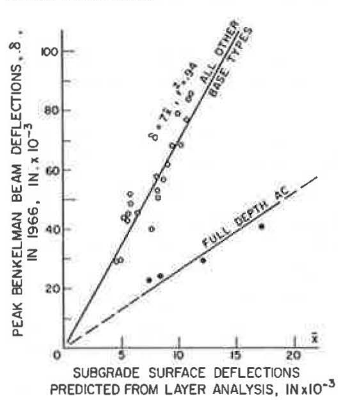
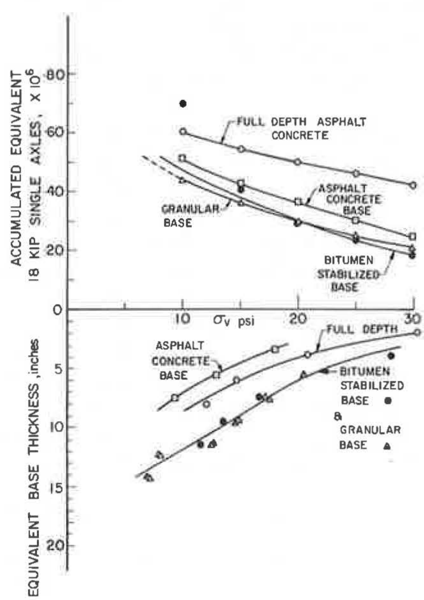


Figure 11. Brampton Test Road materials relations.



APPLICATION OF RCI-AGE (OR TRAFFIC) RELATIONS TO THE PAVEMENT DESIGN SYSTEM

In the previous section, it was shown that the RCI-age (or traffic) relations for various pavement strategies involving Brampton materials could be based on two approaches: the calculated vertical stress on the subgrade surface and the initial peak Benkelman beam rebound (37).

The application of these results to a pavement design system is shown in Figure 12. The diagram shows a simplified flow chart of the major steps required in the design process and includes the two alternative approaches. The method employed for a particular design situation should be governed primarily by the size and importance of the project.

The second approach, based on Benkelman beam rebound, will not be detailed here. The method has been developed to give a set of "second-generation" structural models for the Ontario pavement management system (1) and is described elsewhere (37).

The first alternative employing calculated vertical stress on the subgrade surface is illustrated by the following example problem.

A two-lane highway is to be designed (using a flexible pavement) for a design traffic number (DTN) of 200 in both directions. The location is in a region similar to that of Brampton, and similar materials are to be used. Repeated load triaxial compression test results on samples simulating the field conditions (in terms of density, water content, etc.) are as shown in Figures 3 and 4.

The stiffness modulus for the asphalt concrete, for a range of temperatures and for a loading time of 0.03 sec, is shown in Figure 5. Temperature records show that the average maximum air temperature at the project is about 70 F in June, and the corresponding pavement surface temperature is about 85 F.

In accordance with Figure 12, the designer proceeds in the following steps:

1. Consider the available base and subbase materials. For the purposes of this example, assume that this was restricted to an asphalt concrete base and a sand subbase.
2. Generate an array of possible layer thickness combination alternatives within established constraints for minimum and maximum layer thicknesses. Even for one base and subbase alternative, this can result in a very large number of arrays. A computerized approach is desirable. For the purposes of this example, consider the following thicknesses as one possible combination: surface layer, 3½ in. of asphalt concrete; base layer, 2 in. of asphalt concrete; subbase, 6 in. of sand; and subgrade, natural clay soil of "infinite" depth.
3. Perform the structural analyses as detailed in earlier sections of this paper. The final output after iterating the CHEVRON program gives a vertical stress on the subgrade surface, σ_v, of 18 psi.
4. Predict the performance of each alternative. For this example, assume that the initial RCI is 8.5 and the terminal RCI is set at 4.5. This corresponds to a loss of 4.0 units of RCI to the first overlay. Figure 9 shows that the pavement type considered here will lose 4.0 units of RCI at a σ_v level of 18 psi after approximately 4×10^5 accumulated equivalent 18-kip single-axle loads. For a DTN of 100 (in one direction), the expected design life would be $(4 \times 10^5)/(100)\ (365) = 11$ years.
5. The design problem is not complete until a final design strategy has been recommended. This paper has concentrated on those components of the design phase of pavement management that deal primarily with the structural models and their inputs. However, as shown in Figure 12, overlay designs must be completed for each alternative to the end of the analysis period, and an economic evaluation must be conducted (i.e., expected materials, construction, and maintenance costs must be estimated for each strategy). The designer can then recommend an optimal strategy.

SUMMARY AND CONCLUSIONS

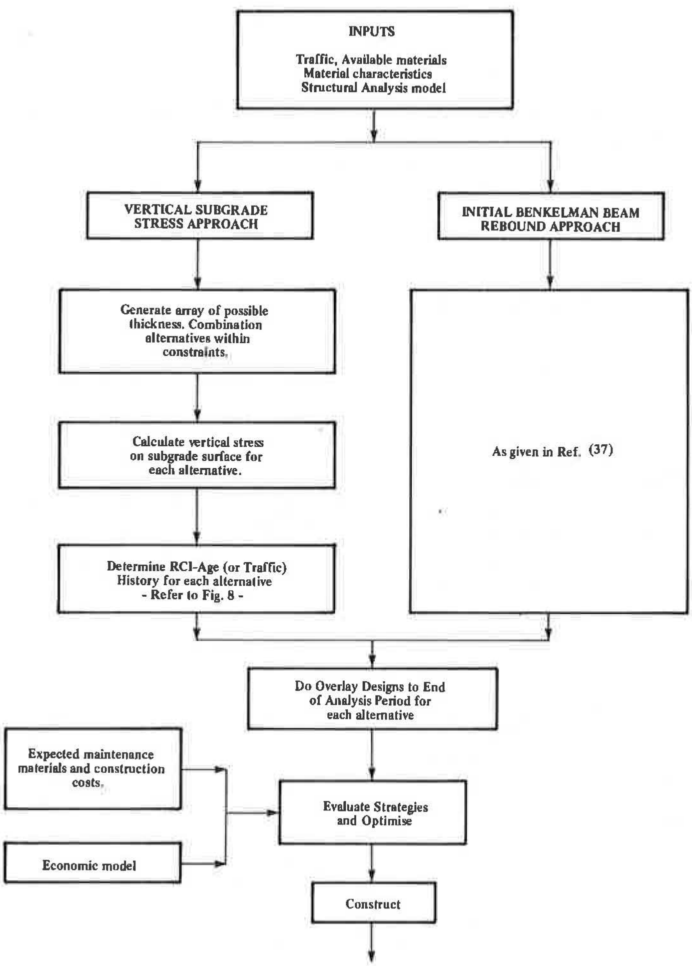
The purpose of this investigation was to determine whether structural analysis could be applied to the design of pavements in terms of predicting serviceability-age history.

Table 2. Brampton Test Road layer equivalencies.

Type of Material	Equivalencies of Granular Base (in.)
1.0 in. of granular base (crushed gravel or crushed rock)	1.0
1.0 in. of sand subbase	0.6
1.0 in. of bituminous-stabilized base	1.1
1.0 in. of asphalt concrete base (with subbase)	2.0
1.0 in. of asphalt concrete base (without subbase, i.e., full depth	3.4

Note: 1 in. of granular base $\simeq$ 1 in. of bituminous stabilized base, $\simeq$ 2 in. of sand subbase, $\simeq$ ½ in. of asphalt concrete base, and, $\simeq$ ⅓ in. of full-depth asphalt concrete.

Figure 12. Pavement design system.



This goal was fulfilled through layer analyses of the various pavement designs employed at the Brampton Test Road, in the following manner:

1. The basic properties of the five pavement materials under consideration, for input to the layer analyses, were established from laboratory test results on the original materials.
2. The structural responses of the various pavement sections in terms of stresses, strains, and deflections under simulated traffic loading, were calculated by iterative, linear-elastic, computerized, layered-system models known as CHEVRON and BISTRO.
3. The calculated structural responses of step 2 were related to the observed performance [in terms of the serviceability-age (or traffic) histories] and to the measured behavior (in terms of Benkelman beam rebounds).
4. It was demonstrated that layer equivalency values could be developed using layer system analysis and a criterion of equal terminal serviceability. The results indicated that, for full-depth pavements, 1 in. of asphalt concrete can conservatively be considered equivalent to 3 in. of granular base.
5. The relations developed in step 3 were demonstrated to be applicable for use in designing pavements; in other words, layer analysis results can be used by the designer to predict the serviceability-age (or traffic) history of pavement strategy.

ACKNOWLEDGMENTS

The work reported in this paper was sponsored primarily by the Ministry of Transportation and Communications, Ontario. The authors wish to thank The Asphalt Institute for making the laboratory test data on the Brampton materials available. The CHEVRON and BISTRO computer programs were provided through the courtesy of the Chevron Asphalt Company and Koninklijke/Shell-Laboratorium, Amsterdam, respectively.

REFERENCES

1. Phang, W. A., and Slocum, R. Pavement Investment Decision Making and Management System. Ministry of Transportation and Communications, Ontario, Rept. RR174, Oct. 1971.
2. Hudson, W. R., et al. A Systems Approach Applied to Pavement Design and Research. Texas Highway Department; Texas A&M Univ.; and Center for Highway Research, Univ. of Texas at Austin, Res. Rept. 123-1, March 1970.
3. Brown, J. L., Buttler, L. J., and Orellana, H. E. A Recommended Texas Highway Department Pavement Design System User's Manual. Texas Highway Department; Texas A&M Univ.; and Center for Highway Research, Univ. of Texas at Austin, Res. Rept. 123-2, March 1970.
4. Haas, R. C. G., and Hutchinson, B. G. A Management System for Highway Pavements. Proc., Australian Road Research Board, 1970.
5. Pavement Management Committee, Roads and Transportation Assn. of Canada. Output Measurements for Pavements Management Studies in Canada. Third Internat. Conf. on Structural Design of Asphalt Pavements, London, Sept. 1972.
6. Kasianchuk, D. A., Terrel, R. L., and Haas, R. C. G. A Design System for Minimizing Fatigue, Permanent Deformation and Shrinkage Fracture Distress of Asphalt Pavements. Third Internat. Conf. on Structural Design of Asphalt Pavements, London, Sept. 1972.
7. Haas, R. C. G., and Hudson, W. R. The Importance of Rational and Compatible Pavement Performance Evaluation. HRB Spec. Rept. 116, 1971, pp. 92-109.
8. Haas, R. C. G., Hudson, W. R., McCullough, B. F., and Brown, J. L. Developing a Pavement Feedback Data System. Highway Research Record 407, 1972, pp. 162-172.
9. Haas, R. C. G. A Working Method for Designing Asphalt Pavement to Minimize Low-Temperature Shrinkage Cracking. Asphalt Institute, Dec. 1971.
10. Carey, W. N., Jr., and Irick, P. E. The Pavement Serviceability Performance Concept. HRB Bull. 250, 1960.

11. Haas, R. C. G., and Anderson, K. O. A Design Subsystem for the Response of Flexible Pavements at Low Temperatures. Proc. AAPT, 1969.
12. Jain, S. P., McCullough, B. F., and Hudson, W. R. Flexible Pavement System—Second Generation, Incorporating Fatigue and Stochastic Concepts. Texas Highway Department; Texas Transportation Institute, Texas A&M Univ.; and Center for Highway Research, Univ. of Texas at Austin, Res. Rept. 123-10, Dec. 1971.
13. Schonfeld, R. Construction of a Full-Scale Experiment as Part of a Unit-Price Contract. Ministry of Transportation and Communications, Ontario, Rept. RR-119, 1969.
14. Phang, W. A. Four Years Experience at the Brampton Test Road. Department of Highways, Ontario, Rept. RR-153, Oct. 1969.
15. Phang, W. A. The Effect of Seasonal Variation on the Performance of Selected Base Materials. Department of Highways, Ontario, Rept. IR-39, April 1970.
16. Kallas, B. F. The Brampton, Ontario Experimental Base Project. Interim Rept., unpublished, Feb. 1967.
17. Kallas, B. F. The Brampton, Ontario Experimental Base Project. Interim Rept., unpublished, Sept. 1967.
18. Kamel, N. I. Performance Analyses of the Brampton Test Road Using Elastic Layered Theory. Univ. of Waterloo, MASc thesis, Dec. 1971.
19. Kamel, N. I. Pavement Design System in Ontario—Second Generation Incorporating Use of Elastic Layer Theory. Transport Group, Civil Engineering Dept., Univ. of Waterloo, July 1972.
20. Morris, J., Kamel, N. I., and Haas, R. C. G. The Development of Design Procedures for Ontario Flexible Pavements. Symposium on Applications of Solid Mechanics, Univ. of Waterloo, June 1972.
21. Morris, J. An Experimental Investigation of the Vertical Stresses in Two-Layered Systems. Univ. of Melbourne, MEngSc thesis, 1961.
22. Terrel, R. L. Factors Influencing the Resilient Characteristics of Asphalt Treated Aggregates. Univ. of California, Berkeley, PhD dissertation, 1967.
23. Dehlen, G. L. The Effect of Non-Linear Material Response on the Behavior of Pavements Subjected to Traffic Loads. Civil Engineering Dept., Univ. of California, Berkeley, PhD thesis.
24. Seed, H. B., Chan, C. K., and Lee, C. E. Resilient Characteristics of Subgrade Soils and Their Relation to Fatigue Failures in Asphalt Pavements. Institute of Transportation and Traffic Engineering, Univ. of California, 1963.
25. Van der Poel, C. A General System Describing the Visco-Elastic Properties of Bitumens and Its Relation to Routine Test Data. Shell, Bitumen Reprint 9, 1954.
26. McLeod, N. W. Prepared Discussion of Ste. Anne Test Road. Canadian Technical Asphalt Assn., 1969.
27. Barksdale, R. D. Compressive Stress Pulse Times in Flexible Pavements for Use in Dynamic Testing. Highway Research Record 345, 1971, pp. 32-44.
28. Southgate, H. F. An Evaluation of Temperature Distribution and Its Relationship to Pavement Deflection. Kentucky Department of Highways, 1968.
29. Kallas, B. F. Asphalt Pavement Temperatures. Highway Research Record 150, 1966, pp. 1-11.
30. Barber, E. S. Calculation of Maximum Pavement Temperatures From Weather Reports. HRB Bull. 168, 1957.
31. A Guide to the Structural Design of Flexible and Rigid Pavements in Canada. Road and Transportation Association of Canada, 1965.
32. Manual on Pavement Investigations. Canadian Good Roads Assn., Tech. Publ. 11, 1959.
33. Wilkins, E. B. Outline of a Proposed Management System for the C.G.R.A. Pavement Design and Evaluation Committee, Proc. Canadian Good Roads Assn., 1968.
34. Chong, G. Measurement of Road Rideability in Ontario. Department of Highways, Ontario, Rept. IR-29, 1969.
35. Scott, W. J. Roads and Their Riding Qualities. Jour. Institute of Civil Engineers, Road Engineering Design, Road Rept. 25, 1947.

36. Kamel, N. I. Applying Structural Analysis of the Brampton Test Road to Pavement Investment Decision Making and Management System. Ontario Joint Transportation and Communications Research Program, Univ. of Waterloo, 1972.
37. Haas, R. C. G., Kamel, N. I., and Morris, J. Brampton Test Road: Analyses of Performance by Elastic-Layer Theory and Application to Pavement Design and Management System in Ontario. Ontario Joint Transportation and Communication Research Program, Final Rept., July 1972.
38. Painter, L. J. Analyses of AASHO Road Test Data by the Asphalt Institute. Proc. First Internat. Conf. on Structural Design of Asphalt Pavements, 1962.