DEVELOPING STRUCTURAL DESIGN MODELS FOR ONTARIO PAVEMENTS

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This paper describes the applications of layered system analysis in designing Ontario pavements. The Brampton Test Road experiment provided this investigation with various data on material properties, climatic conditions, traffic, and performance. Test sections were analyzed by using Bistro and Chevron computer programs, which predict the structural response of stresses, strains, and deflections in pavement sections. The moduli values required for the analysis were derived from laboratory test data provided by The Asphalt Institute. The calculated structural responses were then correlated to observed field behavior and performance of test sections. Pavement responses, as predicted by the layered system analysis, could be closely related to observed behavior and performance. The analysis shows relationships existing between observed surface rut depth and vertical compressive strain on the subgrade surface. The measured Benkelman beam rebounds and calculated subgrade surface deflections are also correlated. Charts to predict pavement performance for various peak Benkelman beam rebounds are developed, and equivalencies of various types of Brampton base materials are derived based on a criterion of equal loss of serviceability. The paper presents a structural design system for flexible pavements and discusses, through an example problem, how this system is incorporated into the overall pavement management system in Ontario.

•DURING the past 5 years, considerable attention has been devoted in Canada and the United States to developing rational pavement design and management systems (1, 2, 3, 4, 5, 6, 7, 8). A pavement management system includes a number of subsystems, i.e., planning, structural design, construction, maintenance, performance evaluation, a data bank, and research. These subsystems are structured together and aimed at producing the best pavement design and management process.

A major component of any pavement management system is the structural design subsystem. The primary outputs of this subsystem are to predict performance (i.e., serviceability history) and associated cost and benefits of various design strategies. Many researchers have attempted to use structural analysis to estimate stresses, strains, and deflections in pavement structures. The structural analysis alone, however, has no significance if it is not related to pavement serviceability history, which is the designer's primary concern. The development of transformation functions that relate structural responses and performance is a comprehensive task and as yet has not been extensively developed in pavement design technology. Kamel (9, 10) and Kamel et al. (11) attempted to develop such functions for conditions in southern Ontario by examining data from the Brampton Test Road and developing charts to predict serviceability losses with traffic (or pavement age) for various vertical stress levels on a subgrade surface.

For this investigation, the Brampton Test Road experiment provided extensive data on material properties, traffic, climate, and performance. The basic purpose of this

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investigation was to determine whether structural analysis could be used to predict serviceability and age (or traffic) histories of various pavement designs used at Brampton. The analysis was then extended to develop a second generation set of pavement structural design models for Ontario's pavement design and management system (1).

This paper reports the structural analysis of the Brampton Test Road sections by using computerized nonlinear, elastic-layered techniques. It presents several relationships obtained between the calculated structural responses, observed behavior, and measured performance of the in-service test sections. It discusses base layer equivalencies for various Brampton materials and the application of the results to pavement design and management systems in Ontario.

STRUCTURAL ANALYSIS OF THE BRAMPTON TEST ROAD

In August and September of 1965, a full-scale experiment known as Brampton Test Road was constructed on Highway 10 north of Brampton, Ontario. Details of the project, construction, objectives, and findings are discussed by Schonfeld (12) and Phang (13, 14). Figure 1 shows a layout of the experimental pavement sections, which consist of 36 test sections and incorporate 5 types of base materials. The test sections of cement-treated base suffered from shrinkage fracture early in the experiment and were not considered in this investigation.

The structural analysis was performed by means of calculating stresses, strains, and deflections throughout each of the test sections. The details of the structural models used for calculations, material characterization, and structural analysis procedures are given by Kamel, Phang, Morris, and Haas (11). Chevron and Bistro multilayered, elastic computer programs were used to calculate the structural responses, and iterative procedures were used to account for the nonlinear characteristics of the materials. The moduli values required for the analysis were derived from results of repeated load triaxial compression tests, which were conducted on Brampton materials (15, 16).

The stiffness moduli of the asphalt concrete, s, were calculated by using McLeod's modified method (17). The stiffness-temperature relationships were then constructed by using a loading time of 0.03 sec, which corresponds to traffic moving at 60 mph (100 km/h) (18).

The temperature at the middepth of each asphalt concrete layer was estimated by using Southgate's models (19). The corresponding stiffness modulus for each layer was then obtained, and the structural layered analysis was performed for each pavement section. The results are given in Table 1.

STRUCTURAL ANALYSIS AND PAVEMENT PERFORMANCE RELATIONSHIPS

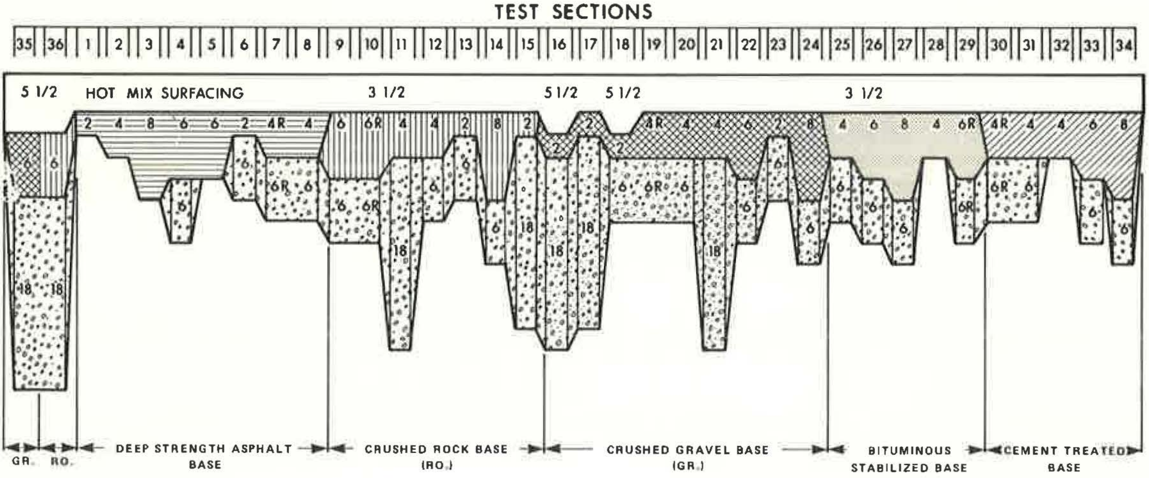
Pavement Serviceability and Performance

At the AASHO Road Test, Carey and Irick (20) developed the best known procedure for defining and obtaining serviceability. Their concept of the 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 the present serviceability index (PSI). The integration of PSI over time or load applications was termed the performance.

The present performance rating (PPR) was developed concurrently in Canada by the Roads and Transportation Association of Canada (RTAC) along similar lines except that it used a 10-point rather than a 5-point scale (21, 22). In 1968, the pavement management committee of RTAC changed the term PPR to riding comfort index (RCI) (3). 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.

At Brampton, RCI was estimated from correlations between RCI and a roughness index profilometer patterned after the British Road Research Laboratory design (23, 24).

Figure 1. 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

Table 1. Results of layer analysis.

Base Type	Section Number	Surface Deflection (in.)	Subgrade Deflection (in.)	Vertical Stresses on Subgrade Surface σ_v (psi)	Vertical Strain on Subgrade Surface ϵ_v (in./in. $\times 10^{-3}$)
Asphalt concrete, no subbase	1	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
Asphalt concrete, 6.0-in. subbase	4	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
Crushed rock, 6.0-in. subbase	9, 10	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
Crushed rock, 18-in. subbase	11	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
Crushed gravel, 18-in. subbase	16	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	5.36	0.150
Crushed gravel, 6-in. subbase	18	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
Bituminous-stabilized, 6-in. subbase	25	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
Bituminous-stabilized, no base	28	0.0241	0.0189	28.60	1.230

Note: 1 in. = 25.4 mm. 1 psi = 6.8948 kPa.

Relationship of Pavement Rutting and Vertical Strain on Subgrade Surface

Measurements of the pavement surface rut depth were taken at Brampton on each test section at 20 wheel-path locations by using a 4-ft (1.2 m) transverse span gauge. The observed surface rutting is related to the vertical compressive strain (ϵ_v) and the vertical compressive stress (σ_v) on the subgrade surface.

The calculated ϵ_v gave better relationships with surface rut depth (RD). Some sample results are shown in Figure 2. Although the data are scattered, it appears that there is a correlation between RD and ϵ_v. In addition, it appears that the full-depth asphalt concrete sections do not follow the same pattern as the conventional sections in that higher strains can be tolerated by the former without excessive rutting.

Trenching was not done at Brampton; consequently it is not conclusively known to what relative extent rutting has occurred in various layers. However, by comparing the RD of the full-depth and the deep-strength asphalt sections, a significant amount of rutting was observed in the subbase. At the AASHO Road Test, an average of 45 percent of the rutting for loops three to six occurred in the subbase (25).

A detailed description of this phase of the analysis is given by Haas, Kamel, and Morris (26). It was emphasized that present technology is only able to recommend criteria for precluding excessive rutting (27, 28); it is not sufficiently developed to be able to confidently predict the actual amount of rutting that might occur for any given design situation. The current state of the art is extensively discussed by Morris and Haas (29).

Relationship of Riding Comfort Index, Traffic, and Vertical Stress on Subgrade

The observed performance, in terms of RCI values, is related to the calculated σ on subgrade surface, and a linear relationship was found at various pavement ages (1965-1970).

The relationships of serviceability loss with traffic at various σ_v levels were subsequently constructed by using the preceding relationships based on the traffic history at Brampton. A typical chart is shown in Figure 3. It can be seen that RCI decreases more rapidly as the vertical stress on the subgrade surface increases. This phase of the analysis is discussed by Kamel (9). These charts can be used to predict pavement serviceability history if traffic history and computed vertical stress on subgrade surface are known. The application of these results to a pavement design system is demonstrated by Kamel, Phang, Morris, and Haas (11).

Relationship of Riding Comfort Index, Traffic, and Benkelman Beam Rebounds

Benkelman beam rebound measurements have been made periodically at Brampton. The computer values of subgrade surface deflections gave a good relationship with the initial, mean-peak rebound values observed in spring 1966. Figure 4 shows these relationships for (a) full-depth asphalt concrete sections and (b) all other Brampton sections considered. A good linear relationship is obtained for the latter with r^2 equal to 0.936 and a standard error of estimate equal to 0.007 in. (0.18 mm). The relationship for all except the full-depth sections is

$$x = 7\,\delta_s$$

where

x = mean-peak initial Benkelman beam rebound, and
δ_s = computed subgrade surface deflection.

This relationship depends on the moduli values that were used for the layer analysis. For the asphalt concrete layers, these moduli were based on a loading time of 0.03 sec, which corresponds to a speed of about 60 mph (100 km/h). If lower speeds (i.e., longer

Figure 2. Surface RD versus δ_s for some Brampton Test Road sections.

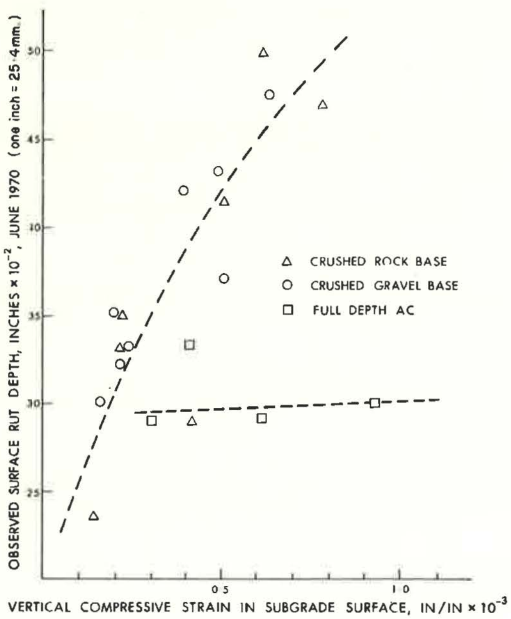


Figure 3. Loss of riding comfort with traffic (or pavement age) for various subgrade stress levels (data from Brampton Test Road).

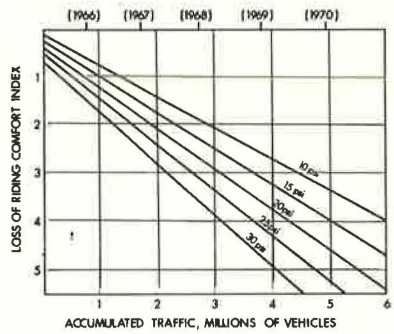
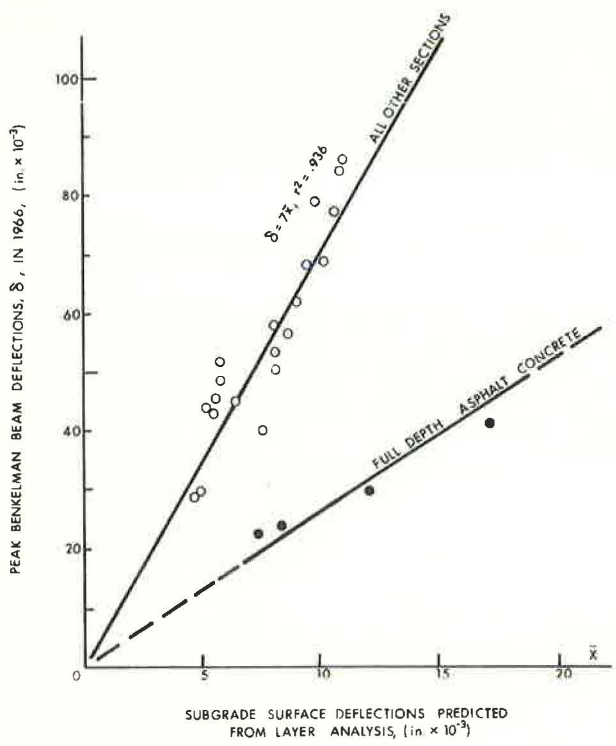


Figure 4. Calculated δ_s versus measured initial mean-peak Benkelman beam rebounds.



loading times) were used, the estimated moduli values would be less and higher subgrade surface deflections should be predicted. In turn, those computed deflections would then more closely correspond in actual magnitude to the rebound measurements of Benkelman beam, which are taken at creep speed.

The computed subgrade surface deflections were also related to the performance of the corresponding in-service experimental pavement sections. Figure 5 shows the good relationships obtained for the full-depth asphalt concrete sections and for all other pavement sections.

Figure 6, a summary of Figure 5 relationships, shows the effect of the magnitude of the subgrade surface deflections δ_s and pavement age A on pavement performance. Pavement age in such relationships reflects the effects of a variety of variables, i.e., climate, aging, subgrade movements, etc. The relationships clearly indicate that RCI decreases as δ_s increases and as A increases. The relative influence of δ_s and A varies more in the case of full-depth asphalt concrete than in all other sections. The influence is less in the former case.

Figure 6 also shows that the loss of RCI between any 2 consecutive years, for any particular level of δ_s, seems to be fairly constant for each of the two behavior patterns. The annual loss is comparatively smaller for the full-depth sections. This suggests that pavements with equal δ_s may be subject to relatively equal annual loss of RCI, where such annual losses increase as δ_s increases.

The foregoing relationships of Figure 4 were used to determine the subgrade surface deflections corresponding to Benkelman beam rebounds of 0.02, 0.04, 0.06, 0.08, and 0.10 in. (0.5, 1.0, 1.5, 2.0, 2.5 mm). These deflection values were then used to determine the corresponding RCI values at various pavement ages from Figure 6. These derived RCI values were found to give the best correlations with Brampton's traffic if plotted on a log scale (Fig. 7) against traffic on arithmetic scales.

Figure 7 clearly shows the serviceability history of the different pavement sections and the behavior under traffic for various pavement strengths. Although these relationships primarily apply to the Brampton Test Road conditions, they should give a general picture of the basic performance characteristics of flexible pavements in similar climatic regions.

DEVELOPMENT OF BASE LAYER EQUIVALENCIES

The equivalencies of various materials have generally been based on equal structural response. In this study, equivalencies are developed on the basis of equal terminal serviceability or equal loss of serviceability. This criterion seems to be more realistic because pavement performance must be the end concern. Equivalencies based on this criterion enable the designer to generate thickness combinations without having to consider the problem of strength differences.

The step-by-step development of the base layer equivalencies is discussed by Haas, Kamel, and Morris (26). Brampton base layer equivalency values (in inches of the granular base) for 1 in. (25.4 mm) of each material type are as follows:

1. Granular base (crushed gravel or crushed rock), 1.0;
2. Sand subbase, 0.6;
3. Bituminous-stabilized base, 1.1;
4. Asphalt concrete base (with subbase), 2.0; and
5. Full-depth asphalt concrete base, 3.4.

That is, 1 in. (25.4 mm) of granular base ≈ 1 in. (25.4 mm) of bituminous-stabilized base ≈ 2 in. (50.8 mm) of sand subbase ≈ ½ in. (12.7 mm) of asphalt concrete base ≈ ⅓ in. (8.4 mm) of full-depth asphalt concrete. The values obtained are very close to those presently employed by the Ministry of Transportation and Communications of Ontario.

APPLICATION OF ANALYSES TO PAVEMENT DESIGN IN ONTARIO

Haas and Hudson (30) proposed that two levels of terminal serviceability be applied to pavements: a desirable terminal level and a minimum acceptable level at which the

Figure 5. Relationship between RCI and calculated δ_s.

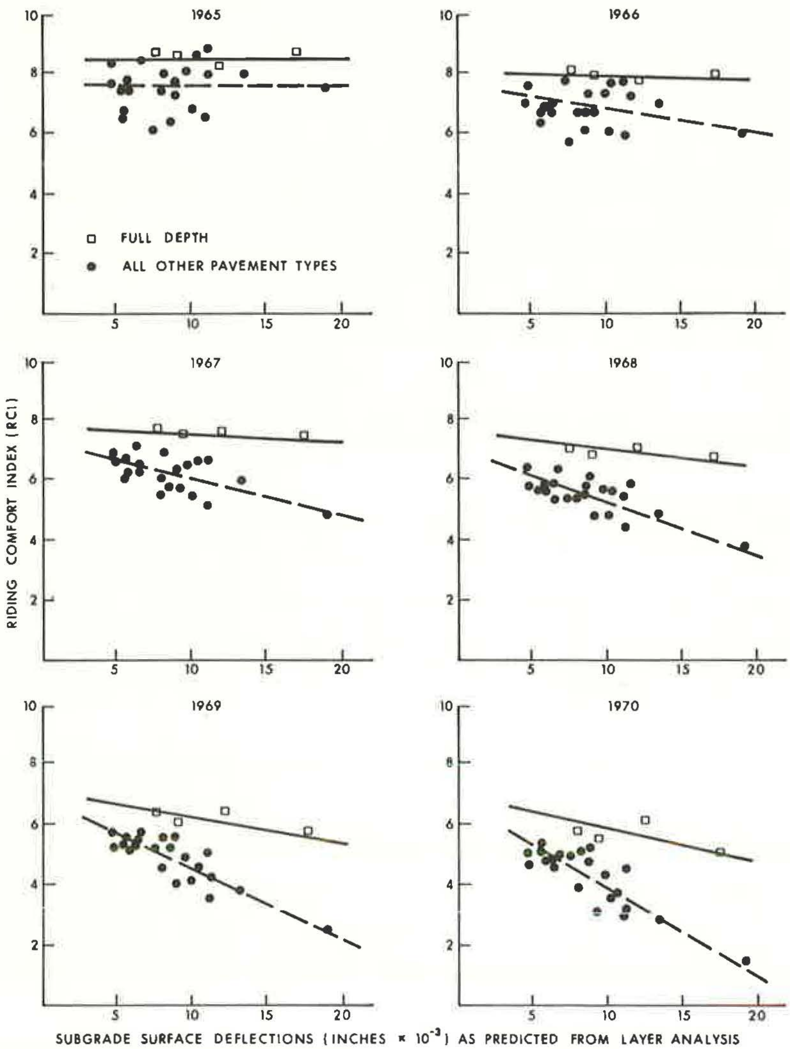
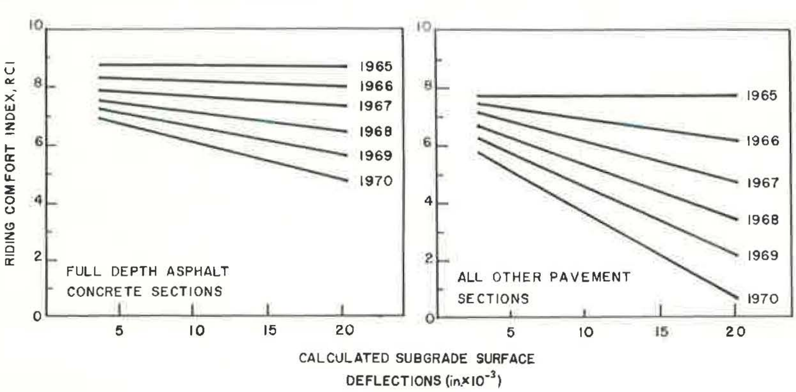


Figure 6. Summary of relationship between RCI and calculated δ_s.



riding quality is so bad that it may affect the normal operation of a passenger car. In this study, an RCI of 4.5 is used by the pavement design and evaluation committee of the Canadian Good Roads Association (3, 21, 22).

Development of Deflection-Based Design Curves

The minimum acceptable RCI of 4.5 on the RCI, traffic, and Benkelman beam curves is shown in Figure 7. The diagram for all pavement sections but the full depth shows that pavements having 0.05-in. (1.3 mm) maximum rebound reach a terminal RCI of 4.5 after about 6 million vehicle passes. This agrees well with the extensive data examined in the Canadian Good Roads Association studies (21).

The relationships of Figure 7 were used to develop the deflection-based design curves of Figure 8. The accumulated traffic value corresponding to the intersection of each performance line of peak rebound with the terminal RCI line was determined. The determined traffic value was then transformed into average daily traffic (ADT) numbers for various pavement ages and was plotted against the corresponding peak Benkelman beam rebound. The deflection-based design curves of Figure 8 are for all pavement sections except the full depth. Similar design curves for full-depth asphalt concrete can be developed by using the upper part of Figure 7.

Pavement Design Approach

Table 2 gives the Ontario flexible pavement design thickness guidelines used by the Ministry of Transportation and Communications for different traffic and highway classes.

Jung and Phang (31) assigned moduli values, E, to various Ontario subgrade soils and developed a thickness design chart that basically reflects the thicknesses given in

Figure 7. Performance relationships for various peak Benkelman beam rebounds.

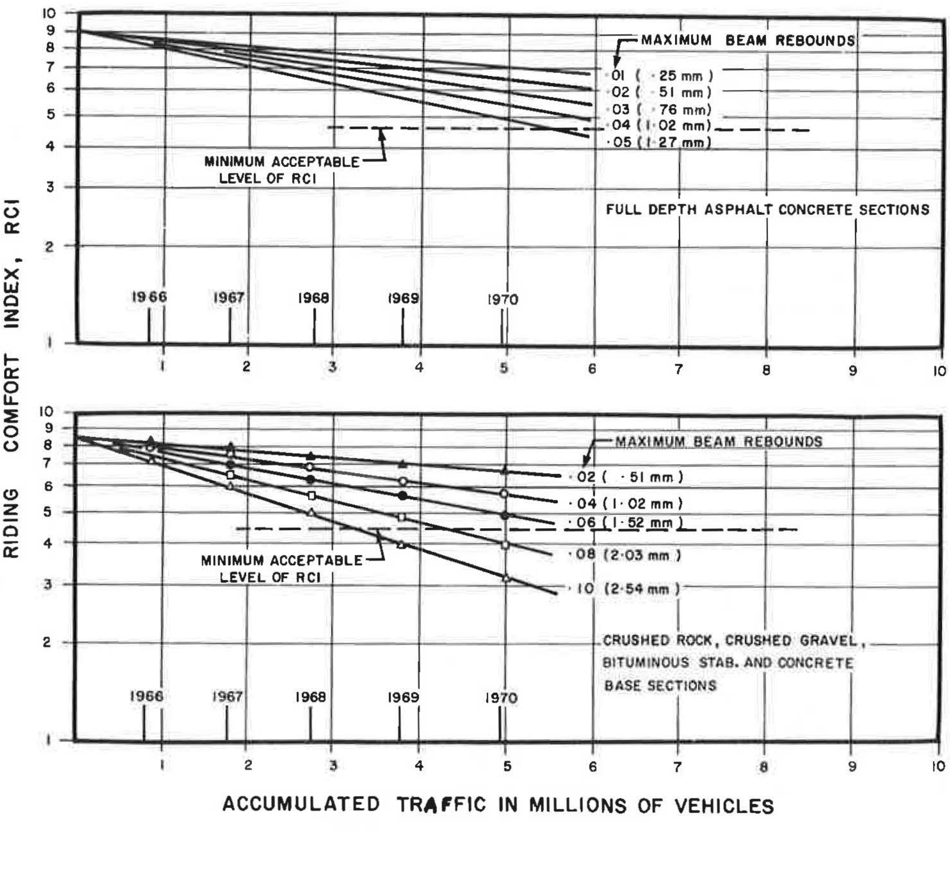


Figure 8. Deflection-based design curves.

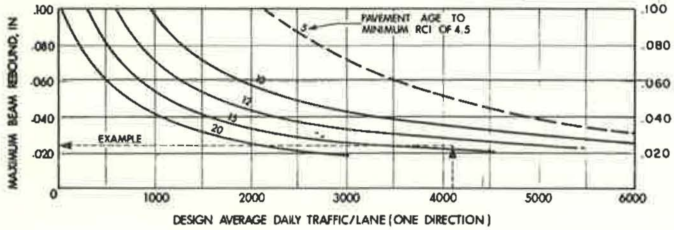


Table 2. Flexible pavement design thickness guidelines.

Road Class	Standard Surface Thickness of Hot Mix (in.)	Total Pavement Thickness of Subgrade[a]: Granular Type of Material Suitable as Granular Borrow	Sandy Silt and Clay Loam Till: Silt <40, Very Fine Sand and Silt <45	Silt 40-50, Very Fine Sand and Silt 45-60	Silt >50, Very Fine Sand and Silt >60	Lacustrine Clays	Varved and Leda Clays	Design ADT (both directions)[b]	Maximum Beam Rebound[c] ($\bar{x} + 2\sigma_{\bar{x}}$)
Multilane									
>8,000 AADT	5½	17-20	25-29	29-33	33-37	29-33	29-45	12,500	0.022
Two lanes									
6,000 to 8,000 AADT	5½	17-20	25-29	29-33	33-37	29-33	29-45	9,000	0.025
4,000 to 6,000 AADT	5	16-19	24-28	28-32	32-36	28-32	28-44	6,500	0.030
2,000 to 4,000 AADT	4	14-17	22-26	26-30	30-34	26-30	26-38	4,300	0.038
1,000 to 2,000 AADT	2½	11-14	19-23	23-27	23-31	19-27	23-31	2,100	0.055

Note: 1 in. = 25.4 mm.

[a]In inches of granular base A (i.e., 1 in. of granular base A = ½ in. of hot mix = 1½ in. of granular subbase C).

[b]Design ADT = AADT $\left(1 + \text{percentage of traffic growth per year} \times \frac{\text{initial life}}{2}\right)$, assumed traffic growth = 4.5 percent per year, lane factor = 80 percent, and initial pavement life = 13 years.

[c]Calculated from maximum rebound of CGRA study.

Figure 9. Relationship of pavement thickness and maximum Benkelman beam rebound relationship for various Ontario subgrade soils.

SUBGRADE MODULI VALUES (Em),psi

GRAN. TYPE MATERIALS	SANDY SILT AND CLAY LOAM TILL: SILT < 40 V.F. SA. AND SILT < 45	SILT 40-50 V.F. SA. AND SILT 45-60	SILT > 50 V.F. SA. AND SILT > 60	LACUSTRINE CLAYS	VARVED AND LEDA CLAYS
9000 - 12000	4500 - 6500	3500 - 5500	3000 - 4500	4000 - 6000	2500 - 4000

ONE INCH = 25.4 mm

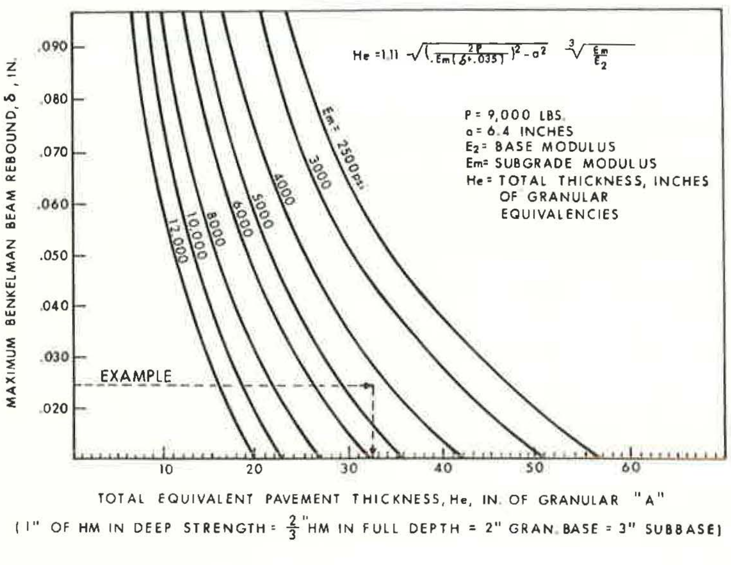


Table 2. In this investigation, the thickness design chart was related to maximum Benkelman beam rebounds (Table 2) for different traffic and highway classes. The chart is shown in Figure 9.

The application of the foregoing results to a pavement design system is shown in Figure 10, which is a simplified flow chart of the major steps required in the design process.

Example Problem

A four-lane divided highway is designed to carry a present AADT of 8,000 vehicles. An average simple growth rate of traffic is assumed to be 4 percent. The two pavement widths are 24 ft (7.3 m) each with 10-ft and 6-ft (3.0 and 1.8 m) outside and inside shoulders respectively. More than 90 percent of the project is constructed on fill materials (clay). The project is in a new location and extended for about 5 miles (8 km).

The designer follows these steps (Fig. 10):

1. Assume that the analysis period = 20 years and E-subgrade clay = 4,500 psi (310 MPa).
2. Calculate the design ADT per lane. Design ADT is defined as the average daily traffic per lane at the middle of the initial life span. Assume the initial life is 14 years and the lane factor is 80 percent. This would result in a design ADT of about 4,100 vehicles per day per lane.
3. Consider the available materials. Assume that this is restricted to an asphalt concrete (HL1-8) and granular type A materials.
4. Generate alternative pavement initial lives and calculate the corresponding maximum rebound for each by using Figure 8. Consider one initial life alternative at 14 years. The deflection-based design curves of Figure 8 show that a pavement section carrying a design ADT per lane of 4,100 and initial life of 14 years should be designed for a maximum rebound of 0.025 in. (0.6 mm). This is only one alternative; the designer could generate other designs of various strengths and initial life spans.
5. Derive the required pavement thickness from Figure 9. About 33 in. (840 mm) of equivalent to granular A material is required for the subgrade clay soil. This total pavement thickness of granular base A could be converted to different alternative combinations of material thickness by using the base layer equivalencies developed previously (i.e., 1 in. of asphalt concrete = 2 in. of granular base A = 3 in. of subbase). The following combination may be used. The designer may choose $1\frac{1}{2}$ in. (38 mm) of asphalt concrete surface HL 1, which is equivalent to 3 in. (76 mm) of granular base A; $10\frac{1}{2}$ in. (267 mm) of asphalt concrete binder HL 2-8, which is equivalent to 21 in. (229 mm) of granular base A; and 9 in. (535 mm) of granular base A—a total equivalency of 33 in. (840 mm). This, of course, is one possible thickness combination. Many other alternative combinations can be considered in the analysis.

Figure 11 shows a cross section of the deep-strength design under consideration as well as the expected serviceability age history of the pavement strategy. Because this design problem deals with design of a major highway, it is recommended that the serviceability level should not drop below RCI = 6. Consequently resurfacing may be required at 10 years (Fig. 11).

The design problem is not complete until a final design strategy has been recommended. This paper concentrated on the parts of the design phase of Ontario pavement management that deal primarily with the structural models and their inputs. As indicated in Figure 10, overlay design must be completed to the end of the analysis period. This may be done in the same manner as described by Phang and Slocum (1). Economical evaluation must also be conducted after estimation of expected materials, construction, and maintenance costs for each strategy. The designer can then, along with other considerations, recommend a final strategy for construction.

This example is an actual work project in the 1973-77 construction program of the Ministry of Transportation and Communications, Ontario. The construction is for Highway 406, St. Catharine's area, Hamilton district. The pavement selection committee of the Ministry, in its recommendation for the most economical design for the

Figure 10. Flow chart of pavement design system.

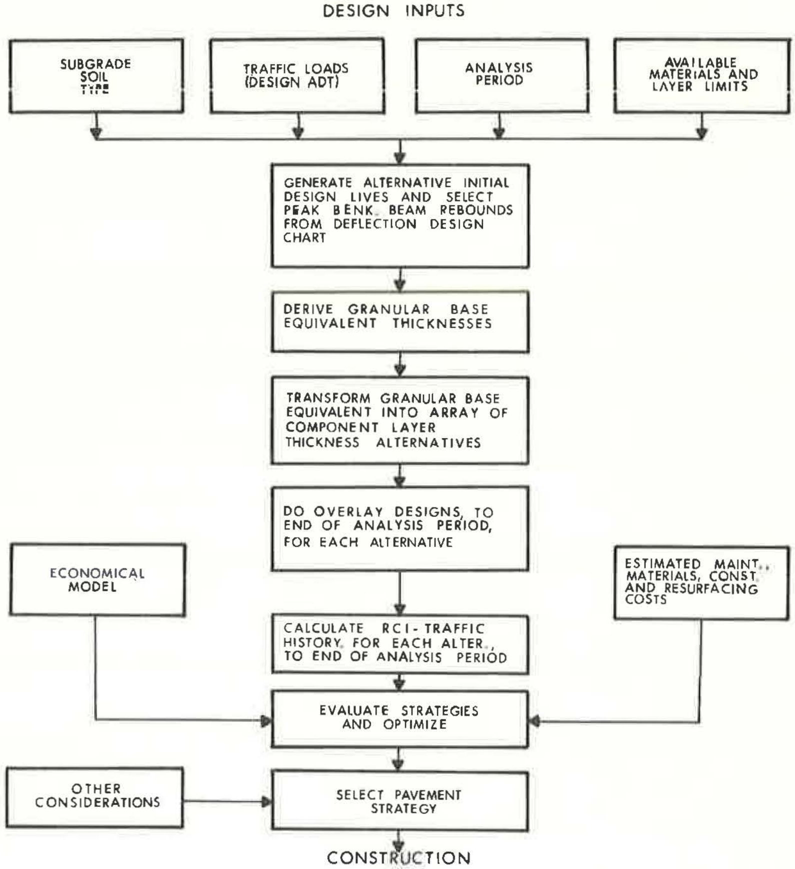
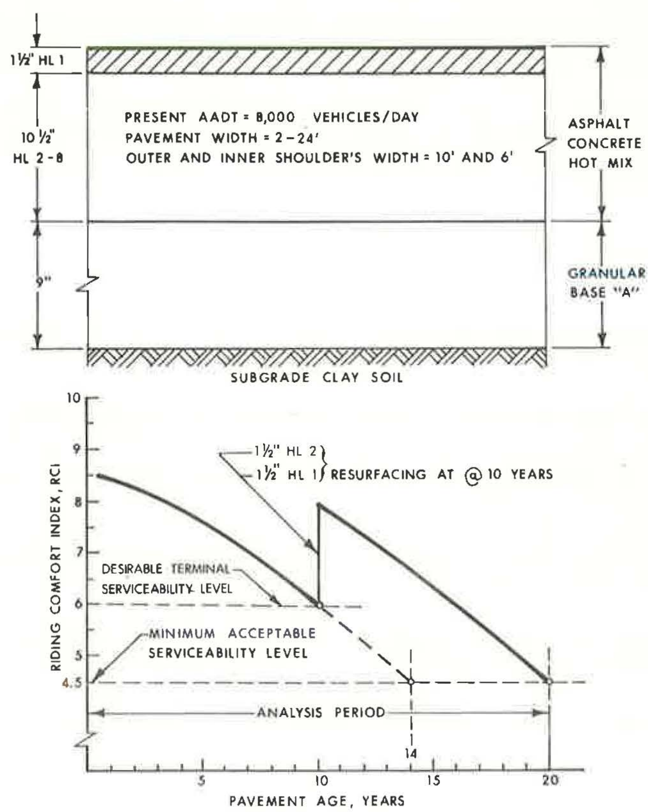


Figure 11. Recommended deep-strength design strategy.



project, has referred to a deep-strength asphalt concrete pavement section having the same total granular equivalency of 33 in. (840 mm) with 1½ in. (38 mm) sand asphalt, and 1½ in. (38 mm) HL 1 resurfacing after 10 years. Figure 11 shows the recommended pavement strategy by the pavement selection committee.

CONCLUSIONS

The general purpose of this investigation was to determine whether layered system analyses could be applied to designing Ontario pavements. This goal was fulfilled through analyses of the Brampton Test Road in the following steps:

1. The basic properties of the five pavement materials under consideration for input to the layered system analyses were established from laboratory testing of the original materials in simulated field conditions.
2. The structural response of the various pavement sections in terms of stresses, strains, and deflections under simulated traffic loading were calculated by iterative, linear elastic, and computerized, layered system models: Chevron and Bistro.
3. The calculated structural responses were related to the measured performance (in terms of the serviceability age or traffic histories) and to the measured behavior (in terms of surface rutting and Benkelman beam rebounds).
4. The calculated responses and analyses were explicitly placed within the context of the Ontario pavement management system described by Phang and Slocum (1). It was demonstrated that a second generation set of design curves for this management system could be developed. The layer equivalency values for the materials of the Brampton Test Road could be developed by using layered system analysis results and a criterion of equal loss of serviceability. These layer equivalencies correspond quite well with those currently used in Ontario.

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