highway projects. This has been done, in substance, by not approving any mixture designs submitted by contractors, unless the calculated asphalt film thickness, at the optimum asphalt content, is approximately 6.5 microns, or higher.

Serious deterioration of the wearing courses has been noticeable on a few Full-Depth asphalt pavements, because the specifications for soundness permitted aggregates to be used that did not always give satisfactory performances. Standard specifications for soundness of the aggregates in the lower classes of wearing course mixtures, permit a maximum freeze-thaw loss in water of 10 and permit the freeze-thaw loss in a water alcohol solution to go as high as 45. Aggregates meeting these soundness requirements have performed satisfactorily in underlying layers. However, for wearing courses, it has been observed, especially for the 3/4-inch mixture size designation, that a satisfactory performance without serious surface deterioration is not obtainable unless a lower maximum water-alcohol freeze-thaw loss is specified for the aggregates in the wearing course mixture.

In this light, for the last two years on primary highway projects involving lower class wearing course mixtures, if there was the possibility that an aggregate might be used that would approach the specified maximum water-alcohol freeze-thaw loss of 45, the state has usually written special provisions for the project, prior to the letting, that require the aggregates in the wearing course mixture to comply with a maximum water-alcohol freeze-thaw loss of 25. For county highway projects, this special provision can also be written for wearing course mixtures, but the initiative has to be taken by the County Engineer or the state Materials Engineer in the district.

Finally, as far as the asphalt-treated base mixtures for Full-Depth asphalt pavements are concerned, the specifications permit the use of almost all natural or crushed local aggregates that will give reasonable performances. There has been no indication that the quality of the aggregates in the bases was not high enough. The mixtures are also desne graded and mostly 3/4 inch in size.

Until the early 1970's, for the most part, a straight 4% asphalt cement content was used in asphalt-treated base mixtures on all Full-Depth asphalt paving projects. To my knowledge, there has been only one or at the very most, possibly two, pavements where it was thought that failures occurred because the 4% asphalt content was too low. However, the asphalt content in the base is considered to be related to the control of spacing between transverse cracks. Not all projects have had transverse cracking, but in general, transverse cracking has occurred on several. The spacings between these cracks have varied among the pavements. On some, the cracks have been very closely spaced, and some of the engineers have been concerned, because it was thought that these cracks could develop into a serious maintenance problem. It was realized that several factors contributed to the cracks, but it was also reasoned that a higher asphalt content in the base mixtures would have a strong influence in controlling the spacing.

In addition to this, Iowa has been very much aware of the thinking for several years among asphalt paving technologists, that asphalt contents for the base mixtures should be determined in the laboratory, using the same criteria that is used for wearing course mixtures. Because of this thinking, and because of the aforementioned failures and variations in the transverse cracking characteristics of the asphalt bases, about three years ago, Iowa adopted the practice of using laboratory tests to designate asphalt contents in the base mixtures of Full-Depth pavements.

So far, these tests have been used to designate asphalt contents that, as near as possible, will give more uniform characteristics to the base course mixtures. These asphalt content percentages for the bases have been somewhat less than test results indicate would be the optimum for wearing course mixtures. The reason for not using the the optimum has mainly been the associated cost increase.

DESIGN AND PERFORMANCE OF ASPHALT CONCRETE BASES IN THE STATE OF WASHINGTON

R. V. LeClerc, Materials Engineer, and J. P. Walter, Bituminous Testing Engineer, Washington State Highway Commission

Asphalt concrete bases have been used in the state of Washington since about 1960. We are generally pleased with the roadway performance of pavement structures using asphalt concrete bases, but it does vary. Although we plan to continue the design and use of asphalt concrete bases, we also plan a concentrated

investigation into the disappointing performances to identify causes and implement corrective measures to prevent repetition.

This report deals with preliminary results from the first few projects we have investigated in which compaction and asphalt quality are suspect. Our data are leading us to believe that asphalt quality is best judged through tests after aging in the Rolling Thin Film Oven. An apparent need for durability tests as well as consistency measurements on asphalts is evident. A modified ductility test at 45 F (7.2 C) is showing some promise to identify asphalts which have given us early hardening.

INTRODUCTION

Asphalt concrete bases have been used in the state of Washington since about 1960. Present standards for heavy duty pavements show structural sections of asphalt concrete base (ACB) varying in thickness from 0.35 ft. to 0.45 ft. (10-14 cm.). These are placed under asphalt concrete pavements (ACP) ranging from 0.25 ft. to 0.35 ft. (7-10 cm.) in thickness. The combination of thicknesses selected for any one job depends upon predicted traffic loadings. Mineral aggregate for the ACB is generally 1 1/4 in. (3.2 cm.) minus dense-graded product with a 50 percent minimum fracture. Both leveling and wearing courses of ACP use 5/8 in. (1.6 cm.) minus dense-graded aggregates with a 75 percent minimum facture.

We are generally pleased with roadway performance of pavement structures using ACB, but it does vary. Some performance is very pleasing, some disappointing, but the average is quite acceptable. Although we plan to continue the design and use of ACB, we also plan a concentrated investigation into the disappointing performances to identify causes and implement corrective measures to prevent repetition.

This report deals with preliminary results from the first few projects we have investigated. Four projects are covered in detail. Two are marginal in performance; immediately adjacent are two others showing better performance. They were selected because they offered good opportunities for comparison. All four projects are on Interstate 90 in Eastern Washington.

PERFORMANCE RATINGS

The performance of highways in the state of Washington is determined by periodic condition ratings conducted in accordance with procedures described in previous reports (1). Basically the rating system consists of an evaluation of two elements -- the pavement ride and the pavement structural condition. The structural capacity is judged by a cataloging of visible defects -- cracking, rutting, raveling, flushing, etc. Each defect is categorized by severity and extent, with negative numbers being assigned to each category. The sum of the negative numbers is subtracted from 100 to obtain the structural rating.

Ride is measured with a Brokaw Roadmeter and the results converted to a scale of 0-10. Utilizing the concept of defects, the zero value represents the best ride and 10 represents the worst. The pavement rating (R_p) is a combination of the ride rating (R_R) and structural rating (R_S). Originally it involved the geometric mean of the two; it has recently been modified to the following equation:

$$R_P = R_S \sqrt{\frac{1 - R_R}{10}}$$

The entire state highway system is rated at least once every two years. Results are tabulated by data processing equipment and made available in a report showing pavement ratings and ride values, as well as type and extent of distress on each one-mile section of highway. The performance curve of a section of highway, or highway project, is a plot of the condition ratings against time in service. Generally speaking, our design contemplates a pavement rating at or above 50 after 10 years.

INVESTIGATION DETAILS

Investigation of the projects involved a compilation and review of design and construction

details together with a study of test data on cores taken from the present pavement. Design data consisting of traffic, climate, and roadway structural sections are identified in the project discussions below and shown in Figures 4, 6, 11, and 17. Construction records were reviewed for identification of any "abnormal" conditions or incidents that might have contributed to poor performance. Asphalt properties and mix compaction data were scrutinized most closely because of their generally recognized importance to serviceability (and because these data were quite readily available). Tests on cores to evaluate present condition of the asphalt concrete pavement concentrated on air void content, asphalt content, and asphalt properties -- mainly penetration at 77 F (25 C) and ductility at 45 F (7.2 C) on the Abson recovered material. Traffic records were also reviewed to determine how much of original design traffic index has been "used up." All data were reviewed in light of the performance curve to identify possible correlation or "causes."

In evaluating asphalts, considerable importance was placed on the percentage of retained penetration after a simulated aging test such as the Thin Film Oven Test or the Rolling Thin Film Oven Test. We inferred that an asphalt which barely met requirements for percent retained penetration would have less durability than an asphalt with retained penetrations well above the minimum requirements, all other things being equal.

Our asphalt concrete mixes are designed to be dense-graded, with a "target" of 15 percent voids in mineral aggregate, and a "final" air void content of 3 percent, after about five years in the roadway. To achieve this, we aim for 6-8 percent air voids at construction. We subscribe to the rationale that void contents above this subject the asphalt films to adverse exposure to the elements with consequent early hardening and embrittlement, the result of which is generally a premature appearance of cracks in the pavement.

PROJECT DESCRIPTIONS

Yakima River to West Ellensburg

This 4-mile (6.4 km.) section of four-lane divided highway was completed in July of 1967. The subgrade was essentially a river gravel embankment keyed with 0.25 ft. (7.6 cm.) of crushed surfacing top course (5/8 in. max. (1.6 cm.)). The pavement section, Figure 1, consists of 0.45 ft. (13.7 cm.) of asphalt concrete base (ACB) topped with leveling and wearing courses of asphalt concrete pavement (ACP) totaling 0.35 ft. (10.7 cm.) in thickness. Asphalt concrete Class "E" (1 1/4 in. minus aggregate) was used for ACB; asphalt concrete Class "B" (5/8 in. minus aggregate) was used for ACP. Specifications for these dense graded mixes are shown in Table 1. The climate in this area, while essentially dry (11 in. (28 cm.) of rainfall per year), suffers from temperature extremes on both ends of the scale. Average highs are 101 F (38 C); average lows are -7 F (-22 C), with short duration maximums and minimums exceeding these values appreciably. Traffic counts show ADT of 10,700 with 15 percent trucks, most of which are five-axle transports. The performance curve, Figure 2, shows an initial rating of 80, which has deteriorated to 52 in the intervening six years. The present condition of the roadway testifies to this rating -- obvious structural cracking in the wheel paths. However, we found these structural cracks penetrated only the wearing and sometimes the leveling course, but not the base course. This leads us to believe these were not base failures, but structural deficiencies in the asphalt concrete. Design contemplates at least 10 years before reaching anything close to this condition. Traffic census data show that the predicted volumes and classifications were essentially correct -- no under-design for traffic loading.

Asphalt used on this project met all specification requirements; however, the retained penetration was not too far above minimum requirements, Figure 4.

Core data, Figure 3, showed densification of all courses, with decrease in air voids greatest in wearing and least in the base. Pneumatic breakdown was used in compaction of the lower courses; steel-wheel in the wearing course. Tests of the asphalt recovered from the roadway cores, Figure 4, showed low penetration values in the wearing courses, and little if any ductility at 45 F (7.2 C), Figure 5.

West Ellensburg to Bull Road

This 2.6-mile (4.2 km.) section of four-lane divided highway was completed in July of 1968. The subgrade embankment, crushed surfacing top course, and asphalt concrete were essentially the same thickness and of the same material as used on the previous section,

Figure 1. Roadway: traffic, climatic, and structural section, Yakima River to West Ellensburg.

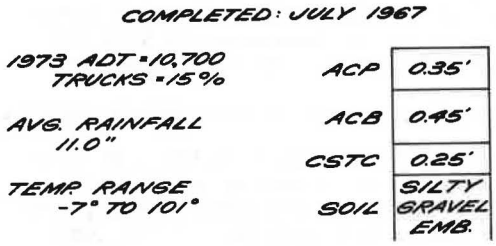


Figure 2. Roadway performance curve, Yakima River to West Ellensburg.

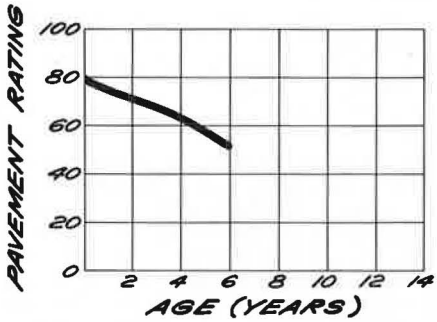


Table 1. Washington State Highway Department asphalt concrete specifications.

		Class B	Class E
Grading			
% Passing	1 1/4 in.		100
	1 in.		90-100
	5/8 in.	100	67-86
	1/2 in.	90-100	60-80
	3/8 in.	75-90	-
	1/4 in.	55-75	40-62
	No. 10	32-48	25-40
	No. 40	11-24	10-23
	No. 80	6-15	6-14
	No. 200	3-7	2-9
% Asphalt		4.0-7.5	3.5-7.0
Fracture (min.)		75%	50%
Sand Equivalent (min.)		45	45
Degradation Factor (min.)		30	20

Figure 3. Percent air voids, as constructed and after service, Yakima River to West Ellensburg.

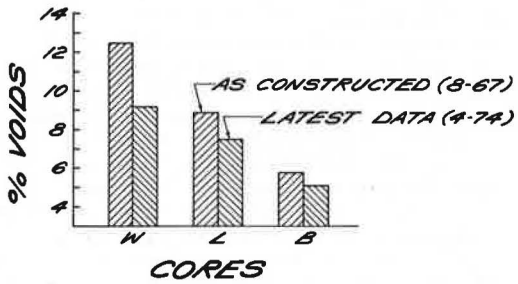


Figure 4. Penetration, original and after service, Yakima River to West Ellensburg.

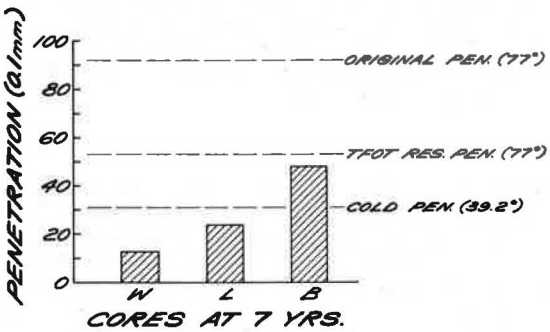


Figure 5. Ductility of recovered asphalt after service, Yakima River to West Ellensburg.

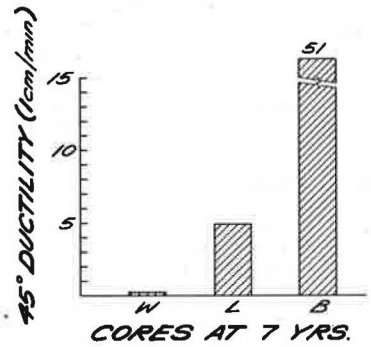
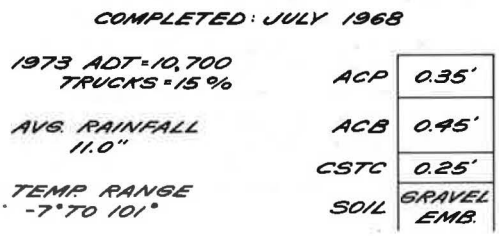


Figure 6. Roadway: traffic, climatic, and structural section, West Ellensburg to Bull Road.



Yakima River to West Ellensburg, Figure 6. The climatic conditions and ADT counts are also the same because the projects are adjoining. The performance curve, Figure 7, shows an initial rating of 79, which has deteriorated to 59 in the intervening five years. The present condition of the roadway is good. A very slight amount of structural cracking in the wheel paths was observed; however, no transverse cracking was observed. Traffic census data show that the predicted volumes and classifications were essentially correct.

Asphalt used on this project met all specification requirements rather handily, Figure 9.

Core data, Figure 8, showed densification of all courses to be the same as in the previous section, i.e., decrease in air voids greatest in wearing and least in base. Pneumatic rollers were used in breakdown compaction of the lower courses; steel-wheel rollers on the wearing. Tests of the asphalt recoverd from the roadway cores, Figure 9, showed low penetration values and little if any 45 F (7.2 C) ductility in the wearing course, Figure 10. However, the penetration and 45 F (7.2 C) ductility in the wearing course, Figure 10. However, the penetration and 45 F (7.2 C) ductilities were progressively better deeper in the pavement and at levels higher than in the adjoining location.

Renslow to Ryegrass

This 5.1-mile (8.2 km.) section of divided highway was completed in July of 1967, but was not opened to traffic until November of 1968. The subgrade was essentially a fragmentary rock keyed with 0.25 ft. (7.6 cm.) of crushed surfacing top course. The pavement section consists essentially of the same combination of asphalt concrete base and asphalt concrete pavement as the previous two sections, Figure 11. The climatic conditions are in most respects the same as the previous sections. Traffic counts show ADT of 8100 with 14 percent trucks, most of which once again are five-axle transports. The performance curve, Figure 12, shows an initial rating of 84, which has deteriorated to 40 in the intervening six years. The present condition of the roadway testifies to this rating -- much structural cracking in the wheel paths and extensive transverse cracking across both lanes and shoulders. However, as in the previous section showing wheel path cracking, the structural cracks extend through the wearing and sometimes into the leveling course, but not into the base course, again leading us to believe these were not base failures but structural deficiences in the asphalt concrete.

Asphalt used on this project met all specification requirements, Figure 14, but retained penetration was not too far above minimum requirements. Samples of asphalt obtained from the refinery representing the asphalt used on this project showed low 45 F (7.2 C) ductilities after artificial aging in the Rolling Thin Film Oven.

Core data showed densification of all courses, Figure 13, with decrease in air voids greatest in the wearing and least in the base. Breakdown compaction was by pneumatic rollers in the lower courses; steel-wheel in the wearing. Tests of the asphalt recovered from the roadway cores showed low penetration, Figure 14, and very little if any 45 F (7.2 C) ductility in al courses, Figure 15.

Ryegrass to Vantage

This 11.7-mile (18.7 km.) section of four-lane divided highway was completed in September of 1968. The subgrade, crushed surfacing top course, and asphalt concrete were practically the same material and identical thicknesses as used on the previous section, Renslow to Ryegrass, Figure 17. The climatic conditions and the ADT counts are identical since the projects are adjoining. The performance curves, Figure 18, show an initial reading of 80, which has deteriorated to 62 in the intervening five years. The present condition of the roadway is good with very little structural damage or transverse cracking present. Traffic census data show that the predicted volumes and classifications were realized with no significant deviation.

Asphalt used on this project met all specification requirements, Figure 20, with retained penetration far above the minimum requirements.

Core data showed densification in all courses except the base course, Figure 19. The decrease in void percentage was greatest in the wearing course. Tests on the asphalt recovered from the roadway cores, Figure 20, showed low penetration in the wearing course, although very reasonable penetration in both the leveling and base courses. The 45 F (7.2 C) ductilities, Figure 21, on the same roadway cores were low in the wearing course and very high in the leveling and base courses.

Figure 7. Roadway performance curve, West Ellensburg to Bull Road.

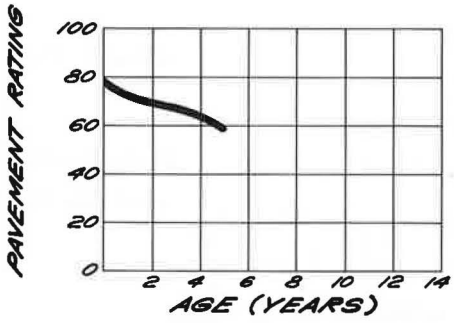


Figure 8. Percent air voids, as constructed and after service, West Ellensburg to Bull Road.

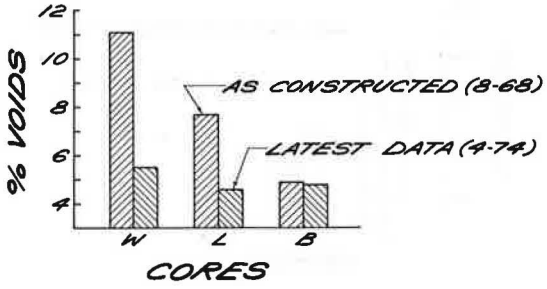


Figure 9. Penetration, original and after service, West Ellensburg to Bull Road.

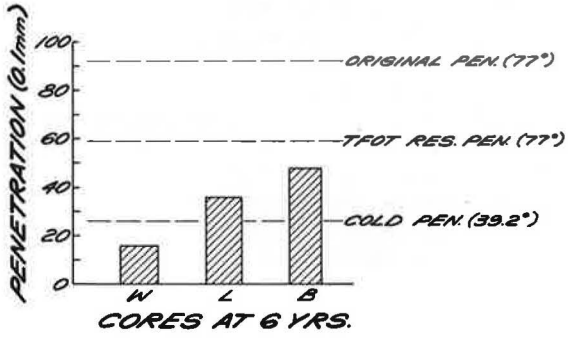


Figure 10. Ductility of recovered asphalt after service, West Ellensburg to Bull Road.

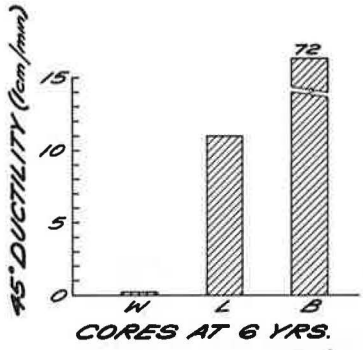


Figure 11. Roadway: traffic, climatic, and structural section, Renslow to Ryegrass.

COMPLETED: JULY 1967

1973 ADT = 8100
TRUCKS = 14 %

AVG. RAINFALL
11.0"

TEMP. RANGE
-7° TO 101°

ACP	0.35'
ACB	0.45'
CSTC	0.25'
SOIL	FRAG. ROCK

Figure 12. Roadway performance curve, Renslow to Ryegrass.

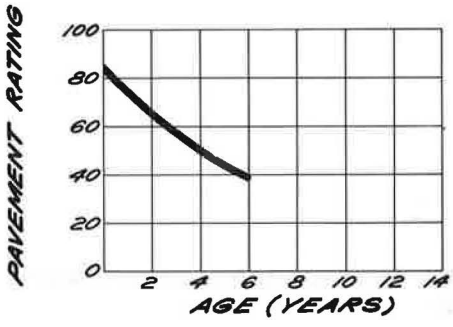


Figure 13. Percent air voids, as constructed and after service, Renslow to Ryegrass.

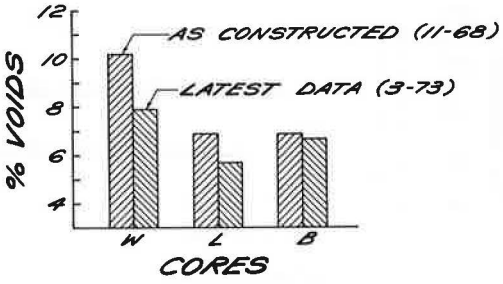


Figure 14. Penetration, original and after service, Renslow to Ryegrass.

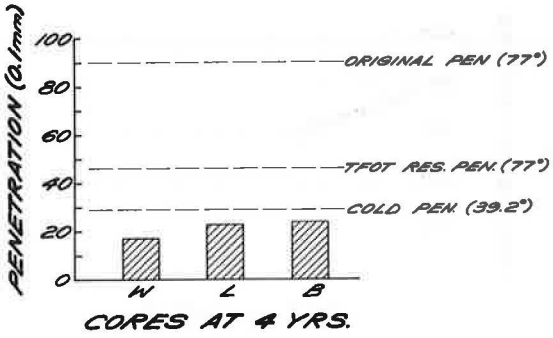


Figure 15. Ductility of recovered asphalt after service, Renslow to Ryegrass.

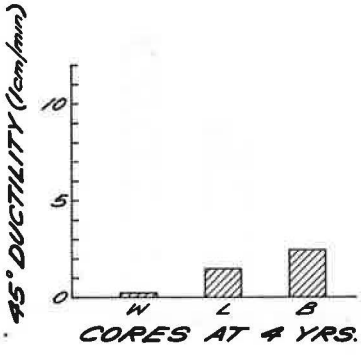


Figure 16. Original asphalt, Renslow to Ryegrass.

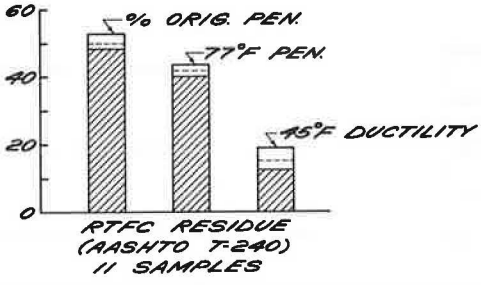


Figure 17. Roadway: traffic, climatic, and structural section, Ryegrass to Vantage.

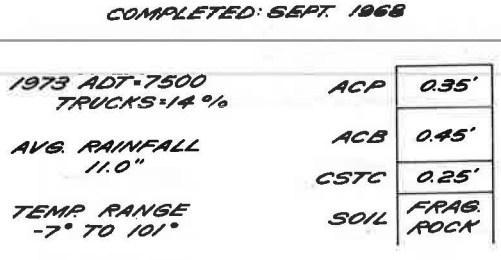


Figure 18. Roadway performance curve, Ryegrass to Vantage.

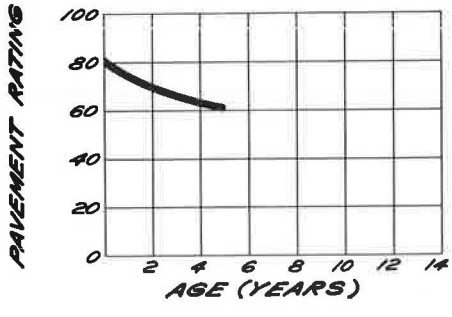


Figure 19. Percent air voids, as constructed and after service, Ryegrass to Vantage.

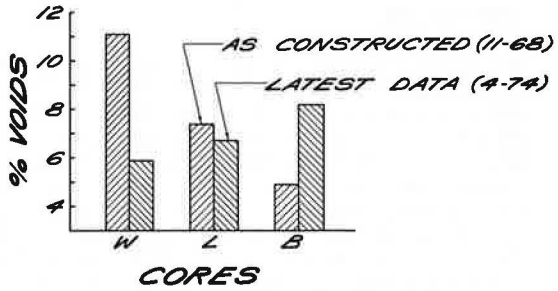


Figure 20. Penetration, original and after service, Ryegrass to Vantage.

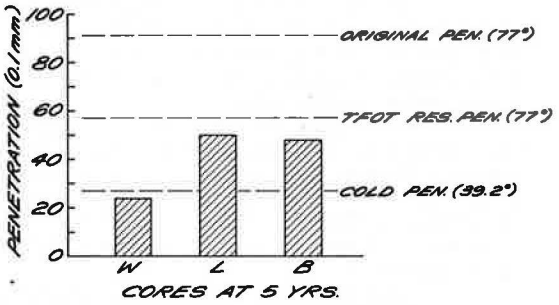
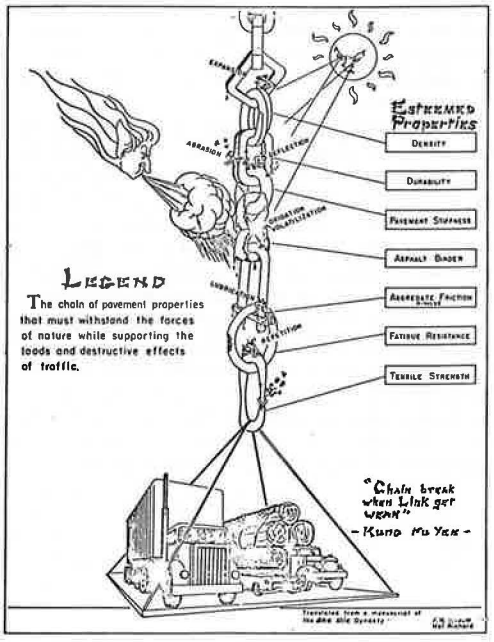


Figure 21. Ductility of recovered asphalt after service, Ryegrass to Vantage.

Figure 22. Variables that affect the performance of a pavement.



CONCLUSIONS

The serviceability of an asphalt concrete depends on many elements of design and construction. Singly and in combinations, these elements have certain sensitivity factors which determine how deviations from optimum will affect performance. Maximum service will be obtained when mineral aggregate quality and grading are correct, asphalt quality high, mix design optimum, asphalt content correct, field mixing and placing procedures proper, and compaction sufficient. Other more specific factors or details can and may be involved. Failure or deviation from optimum in any one of these operations or features, and the final product suffers in quality, the extent of which is cumulative since all are interrelated. This was well illustrated in a novel fashion some years ago by the sketch shown in Figure 22, taken from a report by Francis Hveem (2). We believe that the two most important factors, at least in these projects studied, are asphalt quality and compaction.

Over one of these -- compaction -- we engineers have a large degree of control. Just how much control we have over the asphalt quality is a moot point. While it is true that we can upgrade our asphalt specifications, whether we can then find sufficient material to meet these specifications becomes problematical, particularly in this present energy crisis.

With regard to asphalts, our data are leading us to believe that quality is best judged through tests on residue after aging in the Rolling Thin Film Oven. We also are finding some evidence that consistency measurements alone will not identify or predict durability properties at typical pavement service temperatures, particularly those well below 60 F (15.6 C). As so many have said before, what appears to be needed is a means of measuring that property which might be loosely described as durability or "toughness."

A modified ductility test, performed at 45 F (7.2 C) on aged residue, has shown some promise in identifying asphalts which have given us problems on early hardening. While not especially scientific, this test modified to extend application of release agent to the first inner surfaces of the mold end pieces, has given improved repeatability as well as providing a suitable range of values for rating paving asphalts available to us.

A report (3) presented at the 1975 Annual Meeting of the Transportation Research Board by Kandhal and Wenger of the Pennsylvania Department of Transportation indicated low temperature ductilities (39.2 F (4 C)) showed correlation with pavement performance. This appears to be another indication that low temperature ductilities are a factor in measuring durable asphalts.

As engineers concerned with and/or responsible for design, construction, and performance of asphalt concrete, we should continue to concentrate our efforts on obtaining better compaction and better asphalts while not neglecting the other factors. With a marginal quality asphalt, the better the compaction must be to obtain reasonable performance of the asphalt concrete, all other things being equal. A high quality asphalt is more "tolerant" and you might realize average performance with less than peak compaction; but with marginal asphalt it is not only desirable but almost mandatory that the optimum in compaction be obtained for acceptable pavement performance.

Asphalt seems to have somewhat of a captive market as far as flexible pavements are concerned; nonetheless, reasonable service must be provided or the costs of construction and maintenance will make things even more uncomfortable for highway funding than they are already. In this period of energy crisis, material shortages, and rising prices, highway engineers must concentrate more than ever on getting the most for their money. In this effort, two factors which cannot be slighted are asphalt quality and compaction of the asphalt concrete mix.

The future use of asphalt concrete bases will depend in a large measure on their serviceability. Certainly administrators will be reluctant to use Full-Depth asphalt concrete pavements if costs are not commensurate with service. However, with proper attention to compaction, properly designed asphalt concretes utilizing quality asphalts will provide the serviceability desired.

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2. Hveem, F. N., Designing Bases for Roads and Streets. Public Works, Jan. 1958.