FINAL REPORT

DESIGN OF OVERLAYS BASED ON PAVEMENT CONDITION, ROUGHNESS, AND DEFLECTIONS

by

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SUMMARY

Changing economic conditions have led to a need for more objective means of prioritizing pavement resurfacing needs and for determining the required thickness of resurfacings. Both of these issues have been addressed in the study for which this document constitutes the final report.

Earlier reports on the study were directed at the development of a pavement maintenance rating system and a tentative method for designing the thickness of overlays. The present report is divided into two parts. The first deals with field trials and verification of the pavement rating system and the second with further development of thickness design procedures.

Among the major findings of the study are:

1. An objective rating system can be used to provide a common basis of comparison of pavements between various raters.

2. Methods for designing the thickness of overlays based on the volume of traffic and the existing pavement structure, or on a combination of the two, appear to be practical.

Material developed in the study is being used in an inventory of Virginia interstate pavements.

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INTRODUCTION

For many years in Virginia decisions as to when to provide flexible pavement overlays of bituminous concrete, slurry seals, or other materials have been based on a system wherein pavements are reviewed by at least three engineers utilizing subjective criteria.(1) While such an approach was reasonably effective from the standpoint of funds allocation, the absence of hard data meant the system provided little information concerning the true condition of Virginia's various highway systems (interstate, primary, and secondary). Further, during a study of the interstate system in 1975, the author noted large discrepancies in the levels of pavement surface maintenance between highway districts.(2) In addition, overlay thicknesses historically have been established on the basis of funds available rather than on engineering criteria. As a result of these factors, Vaswani, in 1977, initiated the present study directed at providing objective means for both the establishment of resurfacing priorities and for the design of overlay thicknesses.(3)

Earlier reports on the study established tentative methods for both pavement condition ratings and overlay thickness designs.(4,5,6) The tentative rating system was based on the analyses of results of condition surveys on numerous pavements scheduled for major maintenance during the 1975-77 time period. Among the factors considered in this system were pavement surface distress, pavement rideability, and traffic volume. The system, discussed in more detail later, assigns a rating of 100 to a perfect pavement while the factors mentioned above generate adjustments to that rating.

During 1979 the rating system was used on a trial basis in two highway districts to rate all pavements under consideration for resurfacing or other major maintenance. In 1980 the system was extended to three districts and a formal research evaluation of the findings was undertaken.(7)

Because of the extensive scope of research under the original and revised working plans,[3,7] the present report is divided into two parts. Part I deals with the pavement rating research while Part II is a restatement of the overlay thickness design work with some commentary on the applicability of the design approaches given. It is envisioned that materials discussed in both segments of the report will find use in Virginia's inchoate pavement management system.[8]

PART I

PAVEMENT MAINTENANCE RATINGS

Scope and Approach

To facilitate the implementation of pavement condition rating procedures by field personnel, a pilot study was conducted in which pavements under consideration for resurfacing during 1979 in two districts were unofficially rated and prioritized by the resident engineer, the assistant district engineer, and the state maintenance engineer. Procedures used were those outlined in reference 6. Many of these same pavements were also rated by research personnel. During the fall of 1979, the field, operations, and research engineers involved met and agreed on some modifications to the procedures given in reference 6, and a second-stage pilot study including ratings by a third district was planned.

Pavements rated in this second phase were under consideration for resurfacing during 1980 or later. Again, ratings were made by the resident engineer, the assistant district engineer, and the state maintenance engineer. As soon as rated sections of roadway were identified, they were rated by research personnel using Mays meter roughness tests in addition to the normal procedures. When projects were subsequently resurfaced, roughness tests were conducted. Before pavements in this phase were rated, a short training session was conducted by the author in the Richmond and Suffolk districts. Due to time constraints, no session was held in the Salem District.

The objective of the second-stage study was to refine the rating procedures, including the correction of any deficiencies, and to reflect the findings of the study in a final report.

Procedures

Details of the pavement rating procedure, as modified from that described in reference 6, are given in Appendix A. The modifications, a result of the 1979 first-phase pilot study and discussions with operations personnel, were as follows:

1. Longitudinal and alligator cracking, originally recognized as separate and discrete distress types, were combined. This step was taken to eliminate the confusion experienced by raters when both types of cracking occurred simultaneously. Since in Virginia both types of distress are considered to be related to fatigue, combining them appeared to be justifiable.

2. A traffic coefficient (C_T) was developed and applied to adjust the numerical ratings determined from distress evaluations. This factor, shown graphically in Appendix A, was derived from Virginia pavement design procedures.[9] The shape of the curve relates to the impact of traffic on pavement life as reflected in those procedures. The coefficient has a base of 1.0 at a traffic level equal to the average traffic reported for Virginia interstate and primary highways during 1979 (approximately 6,700 vehicles per day).[10] The coefficient utilizes average traffic counts rather than estimated 18-kip (8160-kg) equivalencies because of the expense and difficulty of determining such estimates for a broad spectrum of pavements. The traffic adjustment was developed because operations personnel perceived a need to recognize that, other factors being equal, pavements with higher traffic counts merit attention before those with lower counts. In use, the coefficient provides a downward adjustment in rating for high volume pavements and an upward adjustment for those with low volumes.

3. A ride adjustment factor (C_R) was developed and applied to the rating based on distress and traffic. This factor is given in Table 1 along with the basis for its development. The factor is based on the ratio of the estimated serviceability index (SI_e) perceived by the rater to a desirable minimum SI_e for a new pavement. As shown in Table 1, each level of C_R has a corresponding approximate roughness level (Mays meter) and approximate SI_e. In practice, the rater does not think of numerical roughness values or SI levels. Rather, he makes subjective evaluations using the verbal description given in Table 1 and assigns the corresponding C_R values. However, when roughness tests were conducted, as will be discussed later, the SI_e values were estimated from Mays meter results using the methodology reported by Walker and Hudson.[11]

The rating procedures were applied to a total of 85 pavement sections in the three districts participating in the second-phase study. All three districts incorporated pavements having a broad range of traffic volumes as indicated in Table 2.

Table 1

Ride Coefficient (C_R)

Ride Quality	C_R	Mays Roughness, in/mi	SI_e
Very Rough	0.7	170	2.8
Rough	0.8	130	3.2
Slightly Rough	0.9	95	3.6
Average	1.0	70	4.0
Smooth	1.1	<70	>4.0

Metric Conversion: 1 in/mi = 1.58 cm/km

Table 2

Pavements Evaluated

District	Number	Traffic Range, vpd
Richmond	31	1,500 - 35,000
Salem	28	500 - 20,000
Suffolk	26	900 - 20,000

In addition to the above normal rating procedures, Mays meter roughness tests were conducted on all pavements rated and on those overlaid again after the overlay.

Results

Results of the second-phase pilot study are discussed below and details of the data collected are summarized in Appendix B. In the succeeding discussion, many abbreviations are used. Those used throughout the discussion are defined below.

DMR - Distress maintenance rating. A measure of pavement distress as defined in Appendix A.

C_R - A ride coefficient as defined above.

C_T - A traffic coefficient as defined above.

MR - An overall maintenance rating defined as MR x C_R x C_T.

ME - Maintenance engineer or his staff.

DE - District engineer or his staff.

RE - Resident engineer or his staff.

R_1 - Researcher No. 1.

R_2 - Researcher No. 2.

Mean Ratings

Mean ratings, by district, of all pavements rated by each rater are summarized in Tables 3 through 5.

Table 3

Mean Ratings, Salem District
(n = 28)

Rating Factor	ME	DE	R_1	R_2
DMR	85.0	75.0	80.0	78.0
C_R	0.92	0.85	0.85	0.88
C_T	1.13	1.10	1.13	1.15
MR	88.0	70.0	77.0	79.0

Table 4

Mean Ratings, Suffolk District
(n = 26)

Rating Factor	ME	DE	RE	R_1	R_2
DMR	93.0	93.0	78.0	81.0	82.0
C_R	0.90	0.90	0.86	0.85	0.89
C_T	1.10	1.08	1.08	1.10	1.12
MR	92.0	90.0	72.0	76.0	82.0

Table 5

Mean Ratings, Richmond District
(n = 31)

Rating Factor	DE	RE	R_1	R_2
DMR	87.0	78.0	79.0	74.0
C_R	0.93	0.88	0.86	0.93
C_T	1.01	1.00	1.00	1.03
MR	82.0	69.0	68.0	71.0

Note that the values tabulated above are those for raters submitting enough data to provide statistically sound comparisons (approximately 30 values). For this reason, the resident engineers' ratings are not given for the Salem District nor are the maintenance engineer's for the Richmond District. The values given are grand averages for all pavements rated in a given district by a given rater. Thus, in the Salem District the maintenance engineer, the district engineer, and two researchers all rated the same 28 pavement sections and the average values for each rater are tabulated.

Therefore, the MR of 88 listed for the maintenance engineer in the Salem District reflects all 28 pavements rated in that district by the maintenance engineers' office. Being an average, the result is made up of a wide variation in individual ratings and, therefore, is not extremely meaningful by itself. However, when all the data are viewed in this manner, some interesting trends are detected.

First, those raters most closely associated with the pavements in their day-to-day activities tend to rate the pavements more harshly. Thus, the resident engineer rates very harshly, the district engineer on an intermediate level, and the maintenance engineer on a high level. This trend may be a manifestation of the psychology involved in the current prioritizing — funds allocation process wherein the resident engineer makes initial resurfacing recommendations. Those recommendations and pavements are then reviewed by the district engineer and the maintenance engineer, both of whom will need to reduce the number of pavements scheduled for resurfacing in order to stay within the funds available. This trend seems to be true with respect to both the overall rating (MR) and the perceived ride quality (C_R). Thus, individuals who tend to rate harshly on the basis of distress also tend to perceive a rougher ride.

Secondly, there are discrepancies between raters on the traffic coefficient. Since this value is determined from a curve based on the reported traffic volume, differences between raters can be due only to differences in interpolation of the curve values.

Finally, pavements rated in the Richmond District tended to have higher traffic volumes and, therefore, lower ratings than those in the Salem and Suffolk districts.

Typical Ratings

Before further discussion of the relationships between raters and between other variables, it is helpful to consider the levels of pavement ratings encountered in the study and the meanings of typical levels.

From composite MR values of all three districts, the distribution of ratings and the description of pavements within those levels is as follows.

1. Pavements rating over 100 comprise approximately the upper 10 percentile of those rated and generally consist of pavements on roads with low traffic volumes. Such pavements were generally perceived to have good ride quality and only a small amount of distress.

2. Pavements rating over 90 comprise approximately the upper 25 percentile of pavements rated. Those rating between 90 and 100 generally do not carry heavy traffic, but are somewhat rougher and more distressed than those rating over 100.

3. Pavements rating between 65 and 90 comprise approximately the medium 50 percentile of those rated. Most of these pavements carry near-medium traffic, tend to be rather rough, and may be badly distressed. On the other hand, some may carry very heavy traffic yet ride well and have very little distress.

4. Pavements rating less than 65 comprise approximately the lower 25 percentile of those rated. In general,

such pavements carry medium to heavy traffic, have a moderately rough ride, and are badly distressed, particularly with alligator cracking and patching.

5. Pavements rating less than 55 comprise the lower 10 percentile of those rated. These pavements tend to be very rough, carry medium to heavy traffic volumes, and be very badly distressed. Most pavements in this category probably have been in service for several years beyond the time it would have been desirable to overlay them.

Clearly, the above descriptions apply only to typical or "average" pavements. The rating system is such that for a given pavement it is possible to have a great deal of distress and a rough ride, yet have such a low traffic volume that the pavement will have a high overall rating. The converse can be true for a pavement with a very high traffic volume. Finally, it should be noted that the above ratings apply only to the pavements actually rated and are in no way indicative of overall pavement conditions in any of the three districts or in the state as a whole.

Correlations Between Raters

For each district, MR values for each rater were correlated with the values for all other raters such that at least 25 degrees of freedom were provided for each correlation. The resulting correlation coefficients are provided in matrix form in Tables 6, 7, and 8.

Perhaps the most striking aspect of the correlation data is the poor correlations observed for the Salem District as opposed to relatively good values in the Suffolk and Richmond districts. The writer is of the opinion that this somewhat strange result is related to the fact that the short training sessions were held in Richmond and Suffolk but not in Salem. It is important to note, however, that even in the Salem District all correlations are significant at the 95% confidence level. Thus, the relationship between raters is real rather than a result of chance, although there apparently were some unidentified factors operating in the Salem ratings. These factors may be related to differences in the interpretation of instructions or in the identification or weighing of distress types.

In both the Suffolk and Richmond districts all correlations are significant at the 99% level; so it may be safely said that in those districts all raters view the same pavements in a similar manner and rate those pavements on similar relative scales. Again, personal biases result in pavements being rated at different levels by different individuals, as discussed earlier.

Table 6

Correlation Coefficients — MR Values, Salem District (n = 28)

Rater	DE	R_1	R_2
ME	0.49	0.67	0.75
DE	-	0.52	0.65
R_1	0.52	-	0.74

Table 7

Correlation Coefficients — MR Values, Suffolk District (n = 26)

Rater	DE	RE	R_1	R_2
ME	0.90	0.90	0.90	0.85
DE	-	0.80	0.80	0.85
RE	0.80	-	0.83	0.87
R_1	0.80	0.83	-	0.79

Table 8

Correlation Coefficients — MR Values, Richmond District (n = 31)

Rater	RE	R_1	R_2
DE	0.82	0.80	0.83
RE	-	0.66	0.67
R_1	0.66	-	0.76

Prioritizing By Raters

Although the use of the pavement maintenance rating procedure in establishing maintenance priorities was not a direct goal of the pilot studies, it is of some interest to examine the possible priorities one could establish using that rating. In Tables 9, 10, and 11 the ten pavement sections having the lowest MR values for each district are listed according to increasing MR values.

Table 9

Rankings by MR, Salem District

Section	Overall	ME	D	R_1	R_2	SI_e	Avg. MR
2-31	1	1	5	1	4	3.27	54
2-10	2	3	2	5	1	3.09	57
2-32	3	2	6	10	6	3.21	62
2-21	4	11	7	2	2	3.34	62
2-30	5	10	1	9	9	3.28	64
2-29	6	9	4	7	14	3.23	66
2-33	7	4	13	3	3	3.54	66
2-28	8	7	3	6	7	3.57	66
2-3	9	8	11	13	5	3.19	71
2-2	10	12	9	17	17	2.73	74

Percentage time raters agree on top ten = 80.

Table 10

Rankings by MR, Suffolk District

Section	Overall	ME	D	RE	R_1	R_2	SI_e	Avg. MR
5-22	1	1	1	1	1	1	3.22	54
5-21	2	2	5	2	2	2	3.21	58
5-23	3	5	3	3	3	3	3.33	58
5-20	4	6	4	4	5	6	2.79	61
5-15	5	7	2	10	8	4	2.83	63
5-13	6	3	7	8	4	9	3.04	63
5-5	7	9	6	7	6	5	3.18	64
5-18	8	8	13	6	10	7	2.51	68
5-25	9	4	12	9	7	14	2.51	71
5-19	10	10	16	5	12	8	3.33	73

Percentage time all raters agree on top ten = 90.

Table 11

Rankings by MR, Richmond District

Section	Overall	D	RE	R_1	R_2	SI_e	Avg. MR
4-7	1	2	1	9	2	2.69	48
4-24	2	4	4	1	1	3.68	49
4-2	3	5	3	2	4	3.40	50
4-1	4	3	2	5	9	3.18	51
4-20	5	1	6	11	8	3.60	54
4-3	6	10	5	8	5	3.40	57
4-13	7	6	15	3	7	3.60	58
4-11	8	7	7	6	12	2.92	59
4-25	9	8	14	4	6	3.41	59
4-12	10	11	11	10	3	3.23	61

Percentage time all raters agree on top ten = 85.

Pavements having equal MR values were ranked according to the estimated serviceability index (SI_e). The pavements are prioritized from 1 through 10 for each rater involved. Note that in the Salem District section 2-31 was assigned a priority of 1 based on the overall average MR value. It was also scored 1 by the maintenance engineer and by one of the researchers. The district engineer placed the section number 5 on his list, while the second researcher placed it number 4. Thus, while there is agreement among the raters that section 2-31 is in poor condition, there is not very good agreement concerning its exact priority. When all ten sections for the Salem District are considered, the data show that the ten pavements receiving the highest priorities based on their overall scores are rated in the top ten on 80% of the individual rating sheets. Due to the greater consistency between raters in the Suffolk and Richmond districts, there is better agreement on the top ten priority pavements in those districts. There is agreement in 90% and 85% of the cases in those two districts, respectively.

Significance of Roughness Tests

Mays meter roughness tests were used to examine the reliability and significance of the perceived ride quality evaluation made by raters at the time of the distress surveys. In Appendix B, serviceability indexes estimated from both the perceived and the measured roughnesses are listed for the three districts evaluated.

Linear regression analyses of these values yielded the results given in Table 12. As the tabulated results show, there were very poor relationships between the two estimates of serviceability index. Raters generally tended to seriously underestimate the true roughnesses, thus overestimating the serviceability index. For reasons unknown to the author, the perceived and measured roughnesses were fairly close in the Richmond District. In all cases, however, the correlation coefficients (r^2) were poor and the standard errors of estimate (SE's) too high to permit reliable estimates of measured values from those perceived. From another point of view, the correlations showed that in both the Salem and Richmond districts the correlations were highly significant. That is, there was a relationship between measured and perceived values. Yet, the relationship was too obscure to be of practical value.

Table 12

Regression Analyses of Perceived vs. Measured Serviceability Indexes

District	N	Average SI_e Measured	Perceived	r^2	SE
Salem	28	3.07	3.52	0.45	0.28
Suffolk	26	3.03	3.53	0.10	0.24
Richmond	31	3.48	3.53	0.56	0.20

In view of the above discussion, it is apparent that if roughness is to be a significant factor in pavement management, actual tests rather than a subjective evaluation must be conducted.

Analysis of Resurfaced Pavements

Subsequent to the field ratings and tests discussed earlier, 29 of the 85 study pavements were resurfaced with approximately 1½ in. (3.8 cm) thick asphaltic concrete mats. Tables 13, 14, and 15 summarize the rating information on pavements resurfaced and those not resurfaced for each district.

The tabulations provide comparisons of each variable for pavements not resurfaced to those resurfaced. Also given is the statistical level of significance to the resurfacing decision for each variable in each district.

Table 13

Comparison of Resurfaced and Not Resurfaced Pavements, Salem District

	N		DMR	C_T	C_R	MR	SI_e
Pavements Not Resurfaced	17	Avg.	83	1.11	0.90	83	3.12
		Std. Dev.	5.7	0.25	0.04	13	0.26
Pavements Resurfaced	11	Avg.	76	1.14	0.85	73	2.98
		Std. Dev.	6.1	0.31	0.08	14	0.50
		Level of Sign.	99%	N.S.	95%	95%	N.S.

Table 14

Comparison of Resurfaced and Not Resurfaced Pavements, Suffolk District

	N		DMR	C_T	C_R	MR	SI_e
Pavements Not Resurfaced	20	Avg.	86	1.11	0.89	86	3.09
		Std. Dev.	5.1	0.20	0.05	19	0.21
Pavements Resurfaced	6	Avg.	81	1.04	0.86	72	2.84
		Std. Dev.	5.7	0.12	0.05	11	0.30
		Level of Sign.	95%	N.S.	N.S.	95%	95%

Table 15

Comparison of Resurfaced and Not Resurfaced Pavements, Richmond District

	N		DMR	C_T	C_R	MR	SI_e
Pavements Not Resurfaced	19	Avg.	83	1.00	0.91	76	3.61
		Std. Dev.	4.1	0.19	0.06	16	0.22
Pavements Resurfaced	12	Avg.	78	1.02	0.84	72	3.28
		Std. Dev.	3.9	0.17	0.07	15	0.42
		Level of Sign.	99%	N.S.	99%	N.S.	99%

Since, as discussed in earlier reports, the whole rating system was built upon the distress rating (DMR), it is not surprising that this variable was significant at the 95% confidence level in each district. On the other hand, the traffic and ride coefficients were fabrications of the researchers in an effort to provide some quantification of these variables in pavement ratings. In the decision to resurface, the traffic coefficient apparently was not a significant variable in any district. The ride coefficient was significant in the Salem and Richmond districts and not significant in the Suffolk District. Strangely, the estimated serviceability based on measured roughness was a significant variable in Suffolk and Richmond, but not in Salem.

The consideration of roughness and serviceability values for pavements resurfaced is of some interest. The values are summarized for all three districts in Table 16, which gives the roughness and serviceability data taken both prior to and after resurfacing. Pavement section numbers with prefixes of 2, 4, and 5 are located in the Salem, Richmond, and Suffolk districts, respectively. Note that those pavements programmed for resurfacing had an average SI_e of 3.08, with a range of 1.93 to 3.76. As shown earlier, ride quality alone is not a sufficient criterion for a decision regarding resurfacing. After resurfacing, the average SI_e was 3.63, with a range of 3.15 to 4.16. The mean increase in SI_e was 0.55 units, or 18%. Mean increases in SI_e were 19%, 12%, and 29% for the Salem, Richmond, and Suffolk districts, respectively. The reader is cautioned that these percentages say nothing about the relative quality of work in the three districts, because of differences in the original quality of pavements programmed by those districts. For example, pavements resurfaced in both Salem and Suffolk tended to be much lower classed roads than those done in Richmond.

Table 16

Roughness and Serviceability
Before and After Resurface

	Before Overlay		After Overlay	
Section	RR (in/mi)	SI_e	RR (in/mi)	SI_e
2-2	174	2.73	132	3.15
2-4	239	2.20	117	3.33
2-8	282	1.93	95	3.61
2-10	138	3.09	97	3.58
2-11	146	3.00	96	3.60
2-13	119	3.30	92	3.65
2-28	98	3.57	77	3.87
2-29	125	3.23	98	3.57
2-30	121	3.28	102	3.52
2-31	122	3.27	89	3.69
2-32	127	3.21	97	3.58
4-3	111	3.40	72	3.94
4-6	107	3.45	86	3.74
4-7	178	2.69	106	3.47
4-9	101	3.53	95	3.61
4-10	209	2.43	112	3.39
4-11	154	2.92	110	3.41
4-12	136	3.23	114	3.36
4-18	135	3.12	81	3.81
4-20	96	3.60	84	3.76
4-24	90	3.68	90	3.68
4-27	100	3.54	82	3.79
4-28	84	3.76	58	4.16
5-5	130	3.18	76	3.88
5-14	160	2.86	91	3.67
5-15	163	2.83	115	3.35
5-18	198	2.51	108	3.44
5-25	198	2.51	71	3.96
5-26	130	3.18	89	3.69
Averages	143	3.08	94	3.63

Metric Conversion: 1 in/mi = 1.58 cm/km.

Conclusions

From the above data and discussions, the following conclusions appear warranted for Part I of this report.

1. An objective rating system can be used to provide a common basis for comparing pavements on the basis of ratings by various individuals. This rating system tends to give more consideration to visual distress on the pavement surface than to traffic volume or perceived ride quality.

2. Raters charged with day-to-day responsibility for the maintenance of pavements rate those pavements more harshly than do raters who see the pavements only for the purpose of rating. While different raters rate pavements at different levels, there are excellent correlations between the raters.

3. To achieve consistent and well-correlated results between raters, a training session to ensure commonality of language and procedures is highly desirable.

4. From a large population of pavements, different raters will place the same pavements within a top ten maintenance priority some 80% to 90% of the time.

5. A perceived or subjective evaluation of ride quality is a poor estimate of the true or measured ride quality. For this reason, the perceived ride quality is not consistently a significant factor in prioritizing pavement sections.

6. Traffic volume was not a significant factor in prioritizing pavements considered in this study.

7. A significant increase in present pavement serviceability is provided by an overlay of about 1½ in. (3.8 cm). This increase ranges between about 10% and 30%.

PART 2

OVERLAY THICKNESS DESIGNS

Introduction

Vaswani, in the original working plan for the present study, spelled out four possible approaches to designing the thickness of overlays. These were as follows:

1. A recommendation of Department engineers based on experience. This practice may not always result in the pavement being restored to its original condition; however, this judgment cannot be ignored.

2. A method presently used in Virginia to determine the on-site structural strength of a pavement by means of deflection data. Deflections taken before the overlay will give the loss in the thickness index since construction. Deflections taken after the overlay will provide the increase in the thickness index due to the provision of the overlay. For example, the provision of a 2½-in. (6.4 cm) thick overlay would give a theoretical increase in thickness index of 2½ in. (6.4 cm.) while the actual gain indicated by deflection data could be more or less.

3. Development of a method that would correlate the loss in the structural strength of a pavement with increased traffic for a given thickness index of the pavement and the soil support value. This chart could be similar to the design chart presently used in Virginia,(9) except that the thickness index lines would be replaced by overlay thickness lines.

4. A correlation of DMR and accumulated traffic with the overlay thickness.

Vaswani pointed out that methods 1 and 2 have been used for sometime and that both have strong disadvantages. The first is so subjective and difficult to quantify that no uniformity between areas of the state is possible; nor is it possible to assess the efficiency of the method in optimizing expenditures. The second method is based on physical testing and is highly quantified. However, the tests are expensive and time consuming to the point that the method is impractical for widespread use in a large highway system.

For the above reasons, most of Vaswani's effort was directed at using the last two approaches to thickness design as discussed below.

Overlay Thickness as a Function of Thickness Index

Relation of Maintenance Rating, Traffic, and Structural Strength

The rate and amount of pavement deterioration as measured by the DMR is a function of the pavement strength and accumulated traffic in terms of 18-kip equivalents. Vaswani determined that the following model equation could be used to correlate these three variables.[5]

$$\text{Log 18-kip} = A + B\ (\text{thickness index}), \tag{1}$$

where A = f (DMR) is a function of the maintenance rating and a constant for a given DMR value, and B = a constant for any given DMR value.

The daily 18-kip (8160-kg) (ESAL-18) equivalent can be determined from a traffic count by means of the chart given in Figure 1. The yearly traffic counts are prepared by the Traffic and Safety Division of the Virginia Department of Highways and Transportation.[10]

The thickness index shows the strength of the pavement without the subgrade support. It is a nondimensional quantity and is obtained by the model equation

$$D = a_1h_1 + a_2h_2 + a_3h_3 + \ldots . \tag{2}$$

In this equation h_1, h_2, and h_3 are the thicknesses of the asphaltic concrete surface layer, the base layer, and the subbase layer, respectively. The terms a_1, a_2, and a_3 are the thickness equivalencies for the respective layers h_1, h_2, and h_3. The values of a_1, a_2, a_3, ... are given in Table 17.

Because no maintenance rating data for pavements in Virginia were available for evaluation, raw data from AASHTO road test pavements were used in this investigation. The AASHTO road test results give raw data on 270 projects comprising different pavement cross sections. On each of the 270 projects, traffic in terms of 18-kip (8160-kg) equivalents is given for DMR values of 83, 71, 60, 48, and 36. The thickness index on each project was obtained by use of the thickness equivalency values given in Table 17 as

$$D = (1.0 \times h_1 + 0.35\ h_2 + 0.2\ h_3). \tag{3}$$

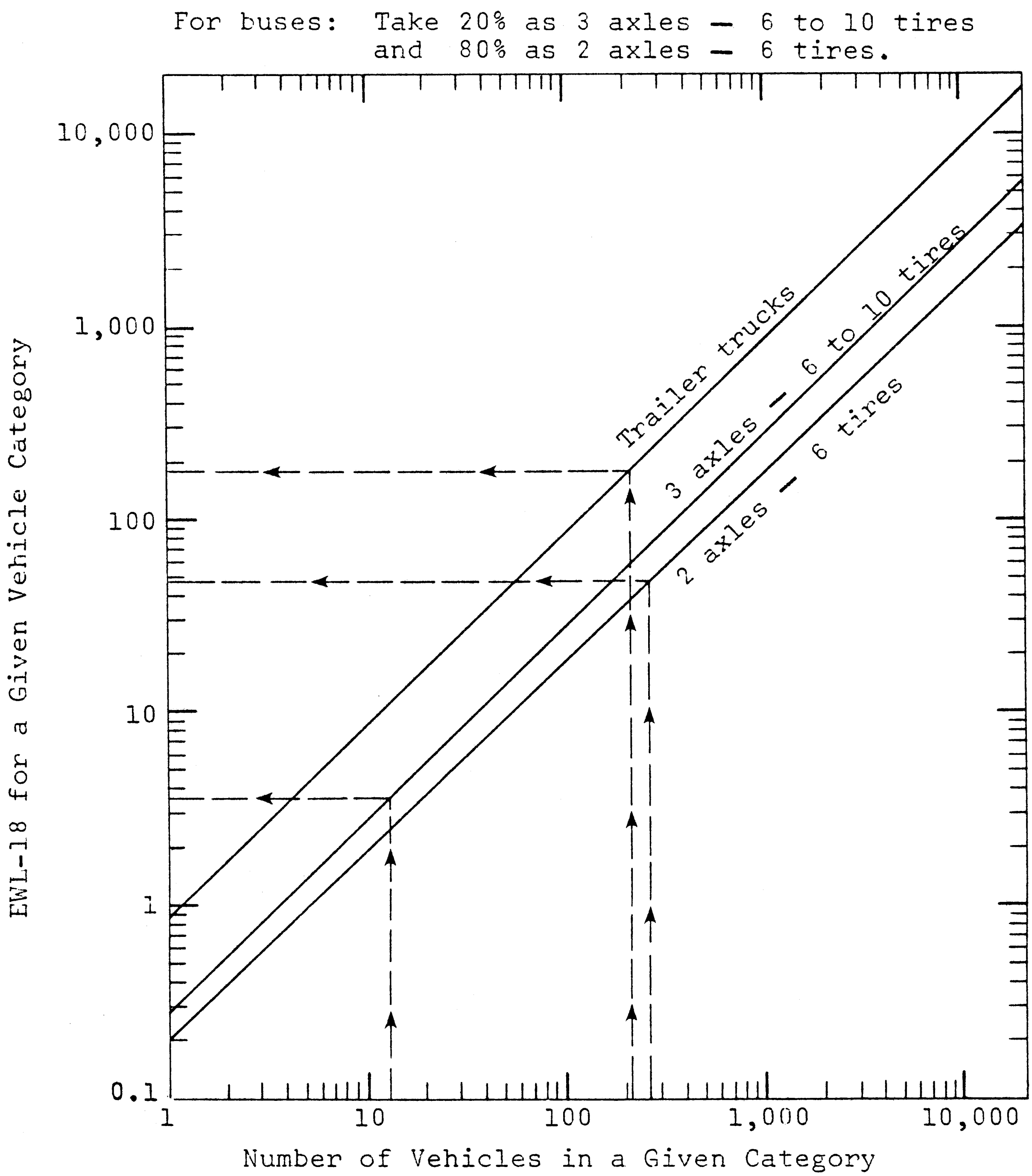

Figure 1. Determination of daily 18-kip equivalent from traffic count. (From reference 12) Metric Conversion: 18-kip = 8160 kg.

Table 17

Thickness Equivalencies of Materials in Virginia For Interstate, Arterial, and Primary Roads

Location	Material	Notation	Thickness Equiv.
Surface	Asphalt concrete.	AC	1.0
Base	(a) Asphaltic concrete.	AC	1.0
	(b) Cement treated aggregate base material over untreated aggregate base or soil cement or soil lime and under AC mat.	CTA	1.0
	(c) Untreated aggregate base material crushed or uncrushed. Spec. No. 20, 21, and 22.	Agg.	0.35
	(d) Select material I directly under AC mat and over a subbase of a good quality (a < 0.2).	Agg.	0.35
Subbase	(a) Select materials types I, II, III.	Sel. Mat.	
	1. In Piedmont area.		0.0
	2. In Valley and Ridge area and Coastal Plain.		0.2
	(b) Soil cement or soil lime.	SC	0.4
	(c) Cement treated aggregate base directly over subgrade.		

(From reference 9.)

Equations based on model equation (1) were developed for DMR values of 83, 71, 60, 48, and 36. These are as follows:

For DMR = 83 (270 data points)

$$\text{Log (18-kip)} = 1.14 + 0.511\ D \quad (4)$$

(Cor. Coeff. = 0.87).

For DMR = 71 (258 data points)

$$\text{Log (18-kip)} = 1.70 + 0.480\ D \qquad (5)$$

(Cor. Coeff. R = 0.92).

For DMR = 60 (239 data points)

$$\text{Log (18-kip)} = 1.82 + 0.488\ D \qquad (6)$$

(Cor. Coeff. = 0.94).

For DMR = 48 (230 data points)

$$\text{Log (18-kip)} = 1.83 + 0.499\ D \qquad (7)$$

(Cor. Coeff. = 0.94).

For DMR = 36 (216 data points)

$$\text{Log (18-kip)} = 1.85 + 0.500\ D \qquad (8)$$

(Cor. Coeff. = 0.94).

As can be seen, the values of B in model equation (1) for the five maintenance ratings as shown by equations 4 through 8 are very similar. The maximum value is 0.511, the minimum is 0.480, and the average is 0.50. The value of the constant B was, therefore, taken as 0.5 and the values of A redetermined through multiple solutions of equation (1). The general equation so determined and the values of A obtained are

$$\text{Log 18-kip} = A + 0.5\ (\text{thickness index}), \qquad (9)$$

and

A = 1.213 for DMR = 83 (R=0.87; S.E.=0.71)
A = 1.582 for DMR = 71 (R=0.92; S.E.=0.49)
A = 1.742 for DMR = 60 (R-0.94; S.E.=0.41)
A = 1.823 for DMR = 48 (R=0.94; S.E.=0.39)
A = 1.871 for DMR = 36 (R=0.94; S.E.=0.39).

The correlation coefficient values and the standard error for the DMR values are also given. The former show that there is an excellent relationship for DMR, traffic, and structural strength.

Overlay Design

While Vaswani showed that the above "A" values can be determined for any maintenance rating,[4] it is evident from Part I of the present report that pavements are subject to some attention by maintenance personnel by the time the DMR reaches the low 80's. For this reason, the subsequent discussion is based upon the assumption that some action will be considered for a pavement when its DMR reaches 83.

The design of the overlay thickness as a function of the pavement thickness index, then, will incorporate a fixed DMR = 83 and A = 1.213. A 12-year overlay design life and an overlay bituminous concrete thickness equivalency of 0.5 have been assumed for reasons given by Vaswani.[4]

It should be noted that since the AASHO road test showed that the deterioration function for an overlaid pavement is similar to that for a new pavement, a pavement with a given DMR prior to resurfacing may be assumed to have the same DMR at the end of the life of the overlay.[5]

With the above in mind, it is possible to write equation (9) as

$$\text{Log 18-kip} = 1.213 + 0.5\ (\text{T.I.}), \tag{10}$$

where T.I. is the thickness index of the pavement after the overlay and log 18-kip (8160-kg) is the accumulated 18-kip (8160-kg) axle loads the overlay will carry during its life. Further, T.I. = D + 0.5T, where D is the thickness index of the existing pavement and T is the overlay thickness. Equation (10) can then be written as

$$\text{Log 18-kip} = 1.213 + 0.5\ (D + 0.5T). \tag{11}$$

Finally, for a 12-year overlay design life equation (11) can be transformed to

$$\text{Log (ESAL-18)} = 0.5D + 0.25T - 2.43. \tag{12}$$

Equation (12) can be rewritten to an overlay thickness design equation

$$T = 4 \log (\text{ESAL-18}) + 9.72 - 2D, \tag{13}$$

where T is the overlay thickness in inches and ESAL-18 is the daily 18-kip (8160 kg) equivalent single axle loads carried by the pavement as determined from Figure 1.

Solutions to equation (13) for D values from 1 to 10 are indicated in Figure 2.

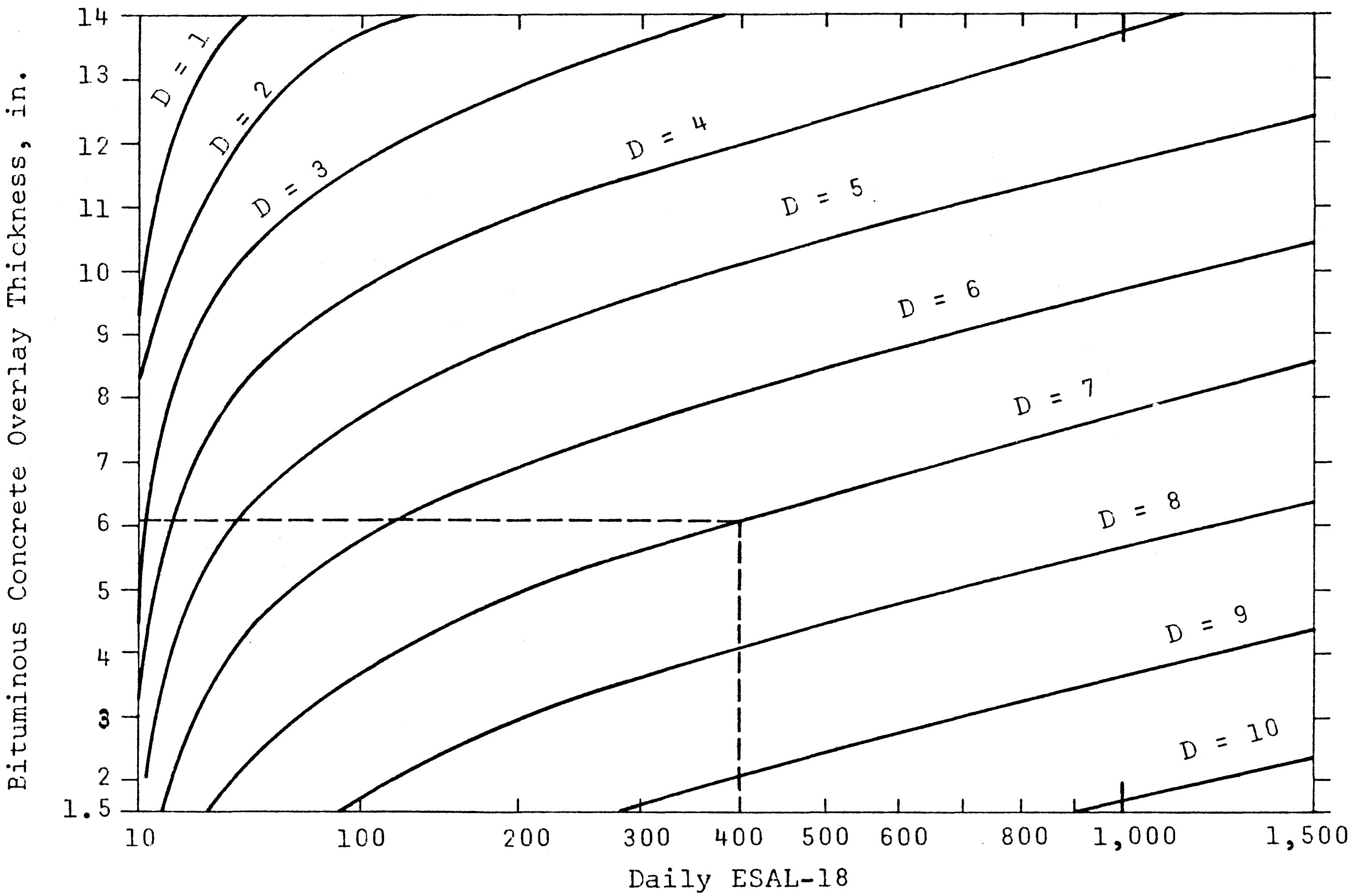

Figure 2. Overlay thickness vs. daily 18-kip equivalency for thickness index (D) of existing pavement.
Metric conversion: 18 kip = 8160 kg; 1 in. = 2.5 cm.

Determination of Overlay Thickness

The overlay designer would first determine the daily 18-kip (8160 kg) equivalent axle loads from Figure 1 and the thickness index of the existing pavement from construction records and Table 17. Then, the required overlay thickness is determined directly from Figure 2. The example shown on Figure 2 is for an ESAL-18 of 400 and an existing D of 7. The required overlay thickness is approximately 6 in. (15 cm).

Discussion

It should be noted in Figure 2 that a minimum overlay thickness of 1.5 in. (3.8 cm) has been provided. This limitation was placed on the chart in the belief that it is difficult to properly construct thinner overlays and that little structural value is received from such overlays. Also, note that pavements with thickness indexes greater than 10 would normally receive the minimum 1.5-in. (2.8 cm) overlay. Thus, nearly all interstate and high type primary roads would receive only the minimum overlay unless special studies show that the design thickness index is not being realized. In practice, then, the procedure would normally be used on lower class primary highways where, unless drastic changes have taken place in the traffic volume, the ESAL-18 values would be below 200 to 300 daily.

Finally, while the design chart in Figure 2 provides results consistent with recommendations resulting from deflection analysis, those results have not been satisfactorily verified in performance studies. For that reason, the chart must at present be considered a tentative approach to overlay design. Additional charts can be developed from the basic equation (1) for different levels of DMR or for design overlay lives other than 12 years.

Overlay Thickness Based on Increased Traffic Volume

For pavements where there is little information concerning the structural capability, Vaswani developed a tentative overlay thickness design method based on the projected increase in traffic volume.[4] This method is summarized below.

Approach

Based on equation (9) the traffic carried by an overlaid pavement could be obtained as

$$\text{Traffic} = \text{Antilog}\ (Aa + 0.5\ Da) - \text{Antilog}\ (Ab + 0.5\ Db), \quad (14)$$

where Ab and Aa are the constants for the maintenance rating before the overlay and at the end of the overlay service life, and Da and Db are the thickness indexes of the pavement before and after the overlay.

As stated above, for a given highway type the DMR values before the overlay and at the end of the overlay service life are the same; that is, Aa - Ab. In such a case equation (14) reduces to

$$\text{Traffic after the overlay} = \text{Traffic before the overlay} \times [\text{Antilog}\ (0.5 \times \text{overlay thickness} \times \text{thickness equivalency of overlay}) - 1], \text{ or} \quad (15)$$

$$\frac{\text{Traffic after the overlay}}{\text{Traffic before the overlay}} = [\text{Antilog}\ (0.25 \times \text{overlay thickness}) - 1], \text{ or} \quad (16)$$

$$\text{Percentage increase in traffic after the overlay} = [\text{Antilog}\ (0.25 \times \text{overlay thickness}) - 1] \times 100. \quad (17)$$

Based on equation (17), Figure 3 has been drawn. It shows the percentage increase in the 18-kip (8160 kg) equivalent versus the overlay thickness and can be used in determining the required thickness of an overlay. This figure shows that the traffic capacities for overlay thicknesses of 1, 2, and 3 in. (2.5, 5.1, and 7.6 cm) are respectively 80%, 220%, and 460% of the traffic before the overlay.

If these percentage increases in traffic are examined carefully, it is seen that the percentage increase would be the same if the overlay were applied in several thin layers rather than in one thick layer.

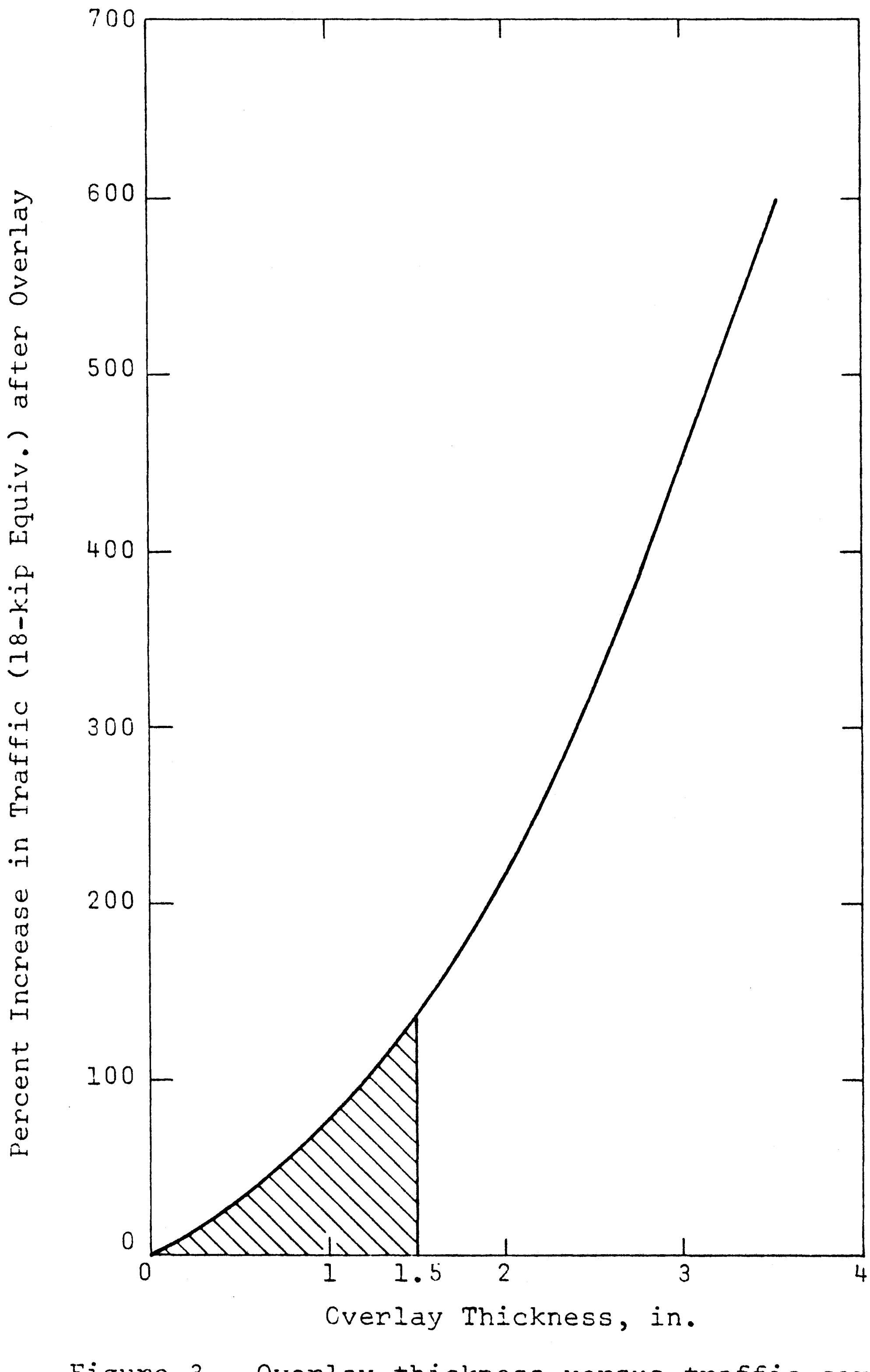

Figure 3. Overlay thickness versus traffic-carrying capability.(Note minimum recommended overlay is 1.5 in.)
Metric conversion: 18-kip = 8160 kg;
1 in. = 2.5 cm.

Deflection studies in Virginia carried out before and after the application of asphaltic concrete overlays have shown that overlay thicknesses of 1.5 in. (3.8 cm) and above contribute to an increase in the structural strength of the pavement. This, as mentioned earlier, is the minimum overlay suggested when structural strengthening is needed.

Thickness of Overlay

The required thickness of an overlay is dependent on the durability of the asphaltic concrete mix as affected by the age, hardening, and stripping of asphalt. An overlay made from a well-designed mix and properly constructed could perform satisfactorily for 10 to 15 years without surface rejuvenation. For determining the thickness of the overlay, a service life of 12 years is recommended for use. The procedure is as follows.

1. Determine the accumulated traffic in terms of the 18-kip (8160-kg) equivalents that the pavement has carried from the date of construction to the date of the proposed overlay, irrespective of any previous overlay. Use Figure 1 to convert the traffic count into 18-kip (8160-kg) equivalents.

2. Determine the accumulated traffic in terms of the 18-kip (8160-kg) equivalents the pavement will carry in the 12 years after the overlay.

3. The percentage ratio of the traffic after the overlay to the traffic before the overlay is

$$\frac{\text{18-kip after the overlay}}{\text{18-kip before the overlay}} \times 100.$$

4. From Figure 3, determine the thickness of the overlay for the percentage ratio of the estimated traffic after the overlay to the traffic before the overlay.

Example

For an interstate highway pavement that was built in 1967 and had a maintenance rating of 76.5 in 1977, it has been decided that an overlay is justified. Having determined the need for an overlay, the thickness of the overlay could be calculated as outlined below.

1. Determination of the daily traffic in 18-kip (8160-kg) equivalents — From the average daily traffic volume records (the average daily traffic volumes on interstate, arterial and primary routes are published by the Department for each year), the ADT values obtained for 1976 are given in Table 18. Figure 1 is used to convert the traffic count to 18-kip (8160-kg) equivalents.

Table 18

ADT Counts and 18-kip Equivalents

Vehicle Type*	ADT	18-kip (8160-kg) Equivalents (From Figure 1)
2-axle — 6 tire	320	58
3-axle — 10 tire	50	14
Trailer Trucks	2,850	2,500
Buses (Assume 20% of 3-axle and 80% of 2-axle vehicles)	40	6
Total		2,578

For four-lane highway
Design Traffic = 2,578 x 0.5 x 0.8 = 1,031 18-kip (8160-kg).**

*Cars and 2-axle — 4-tire vehicles are not considered, because their damaging effect on the pavement is almost negligible.

**The Traffic and Safety Division traffic counts include both directions of travel. One-half the reported traffic is assumed to travel in each direction and 80% of the truck traffic is assumed to use the outside (design) lane.

2. Determination of the accumulated traffic before the overlay — This could be determined from the traffic record or it can be estimated on the assumption that the traffic has increased at the rate of 5% a year (the national standard). Table 19 has been developed to show [a] the growth rate for each year for a 20-year period (the ADT after 9 years = 1.47 x ADT during the first year) and [b] the accumulated traffic for each year for a 20-year period (the accumulated traffic after 9 years = 4,016 x ADT during the first year).

Table 19

Growth Rate and Accumulated Traffic
Assuming 5 Percent Growth

Period of Traffic in Years	Growth Rate	Accumulated Traffic Rate
1	1	365
2	1.05	748
3	1.10	1,149
4	1.16	1,572
5	1.22	2,017
6	1.27	2,480
7	1.34	2,969
8	1.40	3,480
9	1.47	4,016
10	1.54	4,578
11	1.62	5,169
12	1.70	5,789
13	1.78	6,438
14	1.87	7,120
15	1.97	7,839
16	2.07	8,595
17	2.17	9,387
18	2.28	10,219
19	2.39	11,091
20	2.51	12,007

In the above example the accumulated traffic on the road in 1977 at the end of 11 years of service

$$= \frac{\text{Design daily traffic in 1977 x accumulated traffic rate}}{\text{Growth Rate}}$$

$$= \frac{1{,}031 \times 5{,}169}{1.62} = \text{3.29 million 18-kip (8160-kg).}$$

342

3. Determination of the estimated traffic for the life of the overlay — Assuming that the life of the bituminous mix in the overlay will be 12 years, the projected traffic during this 12-year period would

$$= \text{Design daily traffic in 1977} \times \text{accumulated traffic rate for 12 years,}$$

$$= 1{,}031 \times 5{,}\ 789$$

$$= 5.97 \text{ million 18-kip (8160-kg)}$$

4. Design of overlay thickness — The ratio of two traffics

$$= \frac{\text{Accumulated 18-kip (8160-kg) after the overlay}}{\text{Accumulated 18-kip (8160-kg) before the overlay}} \times 100$$

$$= \frac{5.97}{3.29} \times 100 = 180\%.$$

From Figure 2, the design thickness of an overlay for this ratio is 1.75 in. (4.4 cm).

Discussion

As mentioned earlier, the latter approach to the design of overlay thickness presumes that the designer does not know the thickness index of the existing pavement. He will, however, always have access to traffic counts for primary highways, and for this reason, the latter method would find more general use than the first on all lower class roads for which little information is available.

Nevertheless, the method does not accommodate unexpected changes in the structural integrity of a pavement and should be used only when structural information is not available.

Finally, it should be noted that the traditional 5% per annum increase in traffic has not held true over the past few years of high energy costs. For this reason, the designer may wish to make traffic projections on some other basis, although modified long-range factors have not been identified to the author's knowledge.

Conclusions

The data and discussions presented in Part II of this study support the following conclusions.

1. Overlay thickness design methods based on traffic volume and existing pavement structure, or on a combination of the two, appear to be practical.

2. While the methods developed herein are tentative, they are consistent with present new pavement designs and current overlay practice, and they should prove beneficial to maintenance personnel.

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3. Vaswani, N. K., "Working Plan, Design of Overlays Based on Pavement Condition, Roughness, and Deflections", VHTRC 77-WP20, April 1977.

4. _____, "Design of Overlays Based on Pavement Condition, Roughness, and Deflections, Part 1, Tentative Method for Overlay Design Based on Visual Pavement Distress", VHTRC 78-R24, January 1978.

5. _____, "Design of Overlays for Flexible Pavements Based on AASHO Road Test Data', VHTRC 78-R37, February 1978.

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7. McGhee, K. H., "Addendum to Working Plan, Design of Overlays Based on Pavement Condition, Roughness, and Deflections", VHTRC 77-WP20, June 1980.

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9. Vaswani, N. K., "Recommended Design Method for Flexible Pavements in Virginia", VHRC 71-R26, March 1969.

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11. Walker, Roger S., and W. Ronald Hudson, "Method for Measuring Serviceability Index with the Mays Road Meter", Transportation Research Board, Special Report 133, 1973.

12. Vaswani, N. K., "Estimation of 18-kip Equivalent on Primary and Interstate Road Systems in Virginia", Highway Research Record 466, 1973.

APPENDIX A

PAVEMENT MAINTENANCE RATING PROCEDURE

1. Distress Types are identified in "Bituminous Surface Maintenance", MT-5-70.

2. If shoulder width permits, drive slowly on the shoulder over the length of the section to be rated. If insufficient shoulder, stop at three randomly located spots per mile of pavement to be considered.

3. Placing emphasis on the traffic lane, make an overall evaluation of the pavement section by:

   (a) Estimating the frequency of occurrence of each major distress type and indicating it on the rating worksheet in column (2).

   (b) Estimating the severity of each distress type and indicating it on the rating worksheet in column (3).

   (c) For the combination of frequency and severity, select a rating factor for each distress type and record on rating worksheet in column (4).

   (d) Multiply column (4) by column (5) and write the result in column (6).

   (e) Obtain the sum column (6).

4. Compute the distress maintenance rating (DMR) by subtracting the sum of column (6) from 100 as given on the worksheet.

5. The final maintenance rating (MR) is the product of the DMR, a ride coefficient (C_R) obtained from the bottom of the worksheet, and a traffic coefficient (C_T) obtained from the attached graph. Ride quality is judged subjectively by the rater using one of the descriptive terms given to determine the coefficient. The traffic volume used is the latest total traffic reported by the Traffic and Safety Division for the roadway section. The user enters the graph at the appropriate daily traffic, moves vertically to the curve, and reads the traffic coefficient directly from the vertical scale.

PAVEMENT MAINTENANCE RATING

Definitions

Frequency of Occurrence	Percentage of Length Affected
None	0
Rarely Observed	Less than 10%
Occasionally Observed	10% - 40%
Frequently Observed	More than 40%

Severity

Longitudinal Cracking (1-6)*
or Alligator Cracking (1-8)

- Not severe — Cracks not readily apparent.
- Severe — Well-defined cracks.
- Very severe— Well-defined cracks with spalling.

Rutting (1-38)

- Not severe — Not readily apparent.
- Severe — Apparent to naked eye.
- Very severe— Capable of serious ponding.

Pushing (1-34)

- Not severe — Not readily apparent.
- Severe — Apparent but not rough.
- Very severe— Apparent and rough.

Ravelling (1-32)

- Not severe — Not readily apparent.
- Severe — Apparent.
- Very severe— Apparent and rough.

Patching

Rated only on basis of frequency of occurrence.

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*Numbers in parentheses refer to page numbers in Training Guide MT-5-70.

PAVEMENT MAINTENANCE RATING

Worksheet

Date ____________

ounty ____________ Route ____________ Section ____________

rom: ____________ M.P.: ____________

o: ____________ M.P.: ____________

ength: ____________ Traffic Count: ____________

(1)	(2)	(3)	(4)	(5)	(6)
istress Type	Frequency (Circle One)	Severity (Circle One)	Rating Factor (0 to 9)		
ongitudinal Cracking (LC) r Alligator Cracking (AC)	N R O F	NS S VS	____	x 2.4 =	____
utting (Ru)	N R O F	NS S VS	____	x 1.0 =	____
ushing (Pu)	N R O F	NS S VS	____	x 1.0 =	____
avelling (Ra)	N R O F	NS S VS	____	x 0.9 =	____
atching (Pa)	N R O F	NS	____	x 2.3 =	____
				Sum =	____

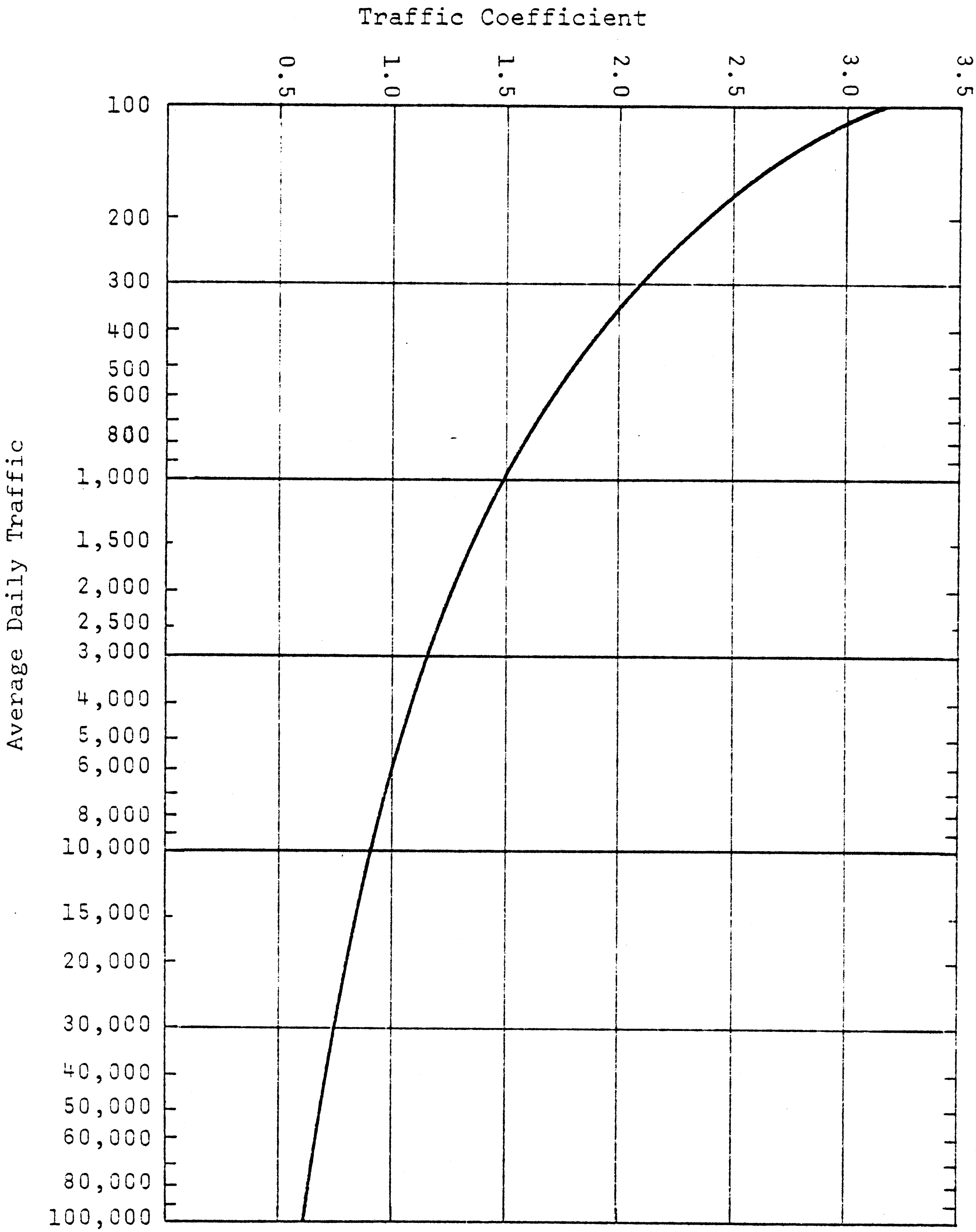
DMR = 100 - sum of column 6 = 100 - ____ =

MR = DMR x C_R x C_T = ____ x ____ x ____ =

requency of Distress		Rating Factor	
	Not Severe (NS)	Severe (S)	Very Severe (VS)
one (N)	0	0	0
are (R) less than 10%	1	2	3
ccasional (O) 10% - 40%	2	4	6
requent (F) over 40%	3	6	9

Ride Quality	Ride Coefficient (C_R)
Very Rough	0.7
Rough	0.8
Slightly Rough	0.9
Average	1.0
Smooth	1.1

346



Traffic Coefficient (C_T) vs. Average Daily Traffic

APPENDIX B

PAVEMENT MAINTENANCE RATING

Salem District

Project	DMR	C_T	C_R	MR	Roughness (in./mi.)	$SI_{\bar{e}}$ Perceived	$SI_{\bar{e}}$ Measured
2-1	75	1.40	0.88	93	174	3.52	2.72
2-2*	76	1.15	0.85	74	173	3.40	2.73
2-3	91	0.80	0.98	71	129	3.92	3.19
2-4*	70	1.72	0.72	87	239	2.88	2.20
2-5	79	1.15	0.88	80	145	3.52	3.01
2-6	80	1.15	0.88	81	172	3.52	2.74
2-7	78	1.78	0.85	118	162	3.40	2.84
2-8*	77	1.62	0.75	94	282	3.00	1.93
2-9	85	1.14	0.85	82	140	3.40	3.06
2-10*	71	1.00	0.80	57	138	3.20	3.09
2-11*	75	1.35	0.80	81	146	3.20	3.00
2-12	91	0.90	0.92	75	101	3.68	3.53
2-13*	78	1.30	0.92	93	119	3.68	3.30
2-14	87	1.13	0.90	88	105	3.60	3.48
2-15	79	1.40	0.92	102	164	3.68	2.82
2-16	80	1.14	0.88	80	126	3.52	3.22
2-17	78	1.16	0.88	80	153	3.52	2.93
2-18	78	1.13	0.88	78	139	3.52	3.07
2-19	87	1.12	0.92	90	145	3.68	3.01
2-20	92	0.84	0.98	76	120	3.92	3.29
2-21	82	0.82	0.92	62	116	3.68	3.34
2-27	92	1.02	0.90	84	125	3.60	3.23
2-28*	90	0.81	0.90	66	98	3.60	3.57
2-29*	83	0.81	0.98	66	125	3.92	3.23
2-30*	79	0.92	0.88	64	121	3.52	3.28
2-31*	70	0.94	0.82	54	122	3.28	3.27
2-32*	72	0.98	0.88	62	127	3.52	3.21
2-33	83	0.84	0.95	66	100	3.80	3.54

*Projects resurfaced

Metric Conversion: 1 in/mi = 1.58 cm/km.

PAVEMENT MAINTENANCE RATING

Suffolk District

Project	DMR	C_T	C_R	MR	Roughness (in./mi.)	SI_e Perceived	SI_e Measured
5-1	89	1.14	0.94	93	117	3.76	3.32
5-2	84	1.24	0.86	90	155	3.44	2.91
5-3	91	1.23	0.86	96	173	3.44	2.73
5-4	97	1.15	0.88	98	151	3.52	2.95
5-5*	85	0.87	0.87	64	130	3.48	3.18
5-6	88	1.28	1.00	113	121	4.00	3.28
5-7	87	1.29	0.96	108	116	3.84	3.34
5-8	88	1.26	0.94	104	152	3.76	2.93
5-9	90	1.26	0.92	104	135	3.68	3.12
5-10	87	1.25	0.86	93	159	3.44	2.87
5-11	88	1.32	0.90	104	110	3.60	3.41
5-12	84	0.92	0.98	76	132	3.92	3.15
5-13	88	0.81	0.88	63	142	3.52	3.04
5-14*	87	1.20	0.88	92	160	3.52	2.86
5-15*	71	1.14	0.78	63	163	3.12	2.83
5-16	77	1.10	0.94	80	166	3.76	2.80
5-17	82	1.50	0.88	108	144	3.52	3.02
5-18*	79	1.08	0.80	68	198	3.20	2.51
5-19	77	1.08	0.88	73	117	3.52	3.33
5-20	89	0.82	0.84	61	167	3.36	2.79
5-21	86	0.82	0.82	58	127	3.28	3.21
5-22	80	0.82	0.82	54	126	3.28	3.22
5-23	86	0.82	0.82	58	117	3.28	3.33
5-24	77	1.20	0.84	78	149	3.36	2.97
5-25*	80	0.98	0.90	71	198	3.60	2.51
5-26*	83	0.99	0.90	74	130	3.60	3.18

*Projects resurfaced

Metric Conversion: 1 in/mi = 1.58 cm/km.

PAVEMENT MAINTENANCE RATINGS

Richmond District

Project	DMR	C_T	C_R	MR	Roughness (in./mi.)	SI_e Perceived	SI_e Measured
4-1	83	0.77	0.80	51	130	3.20	3.18
4-2	81	0.77	0.80	50	111	3.20	3.40
4-3*	73	1.00	0.78	57	111	3.12	3.40
4-4	84	0.96	0.82	66	131	3.28	3.16
4-5	79	1.23	0.82	80	82	3.28	3.79
4-6*	81	1.10	0.92	82	107	3.68	3.45
4-7*	81	0.76	0.78	48	178	3.12	2.69
4-8	76	1.30	0.95	94	79	3.80	3.84
4-9*	73	1.20	0.85	74	101	3.40	3.53
4-10*	80	1.30	0.75	78	209	3.00	2.43
4-11*	82	1.00	0.72	59	154	2.88	2.92
4-12*	80	0.93	0.82	61	125	3.28	3.23
4-13	86	0.73	0.92	58	96	3.68	3.60
4-14	86	0.88	0.92	70	91	3.68	3.67
4-15	80	1.01	0.88	71	119	3.52	3.30
4-16	80	1.10	0.90	79	94	3.60	3.62
4-17	83	0.94	0.92	72	90	3.68	3.68
4-18*	75	1.16	0.92	80	135	3.68	3.12
4-19	81	1.33	0.95	102	91	3.80	3.67
4-20*	72	0.83	0.90	54	96	3.60	3.60
4-21	80	0.92	1.00	74	82	4.00	3.79
4-22	85	0.98	0.95	79	93	3.80	3.64
4-23	81	0.90	0.95	69	88	3.80	3.71
4-24*	74	0.81	0.82	49	90	3.28	3.68
4-25	76	0.86	0.90	59	110	3.60	3.41
4-26	92	0.87	1.00	80	81	4.00	3.81
4-27*	81	1.08	0.85	74	100	3.40	3.54
4-28*	81	1.12	0.95	86	84	3.80	3.76
4-29	88	1.11	1.00	98	70	4.00	3.97
4-30	87	1.30	0.95	107	89	3.60	3.69
4-31	85	1.10	0.92	86	89	3.68	3.69

*Projects resurfaced

Metric Conversion: 1 in/mi = 1.58 cm/km.