FINAL REPORT

REVIEW OF THE DESIGN AND PERFORMANCE OF SANDWICHED PAVEMENTS

by

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Virginia Transportation Research Council
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SUMMARY

The studies reported herein concern one of the basic pavement design types used by the Virginia Department of Transportation for many years. Known as a sandwiched pavement design, the pavement consists of a relatively weak aggregate base layer between a strong, asphaltic concrete upper layer and a strong cement-treated stone or cement-stabilized soil lower layer.

Included in the studies were deflection analyses, performance evaluations, and the collection of aggregate base mechanical properties.

The studies show that the sandwiched pavements generally do not perform as well as conventional pavements where layers grow successively weaker from the top to the bottom of the pavement. The life expectancy for sandwiched pavements was on the average two years shorter. Studies also showed that the deflection characteristics and therefore the performance of the sandwiched pavements is strongly influenced by the amount of minus 200 material in the aggregate base layer.

A recommendation to management that the Department consider greater use of a graded aggregate base with no more than 8 percent minus 200 is included.

To Convert From	To	Multiply By
Length:		
in	cm	2.54
in	m	0.025 4
ft	m	0.304 8
yd	m	0.914 4
mi	km	1 . 609 344
Area:		
in^2	cm^2	6.451 600 E+00
ft^2	m^2	9.290 304 E-02
yd^2	m^2	8.361 274 E-01
mi^2	Hectares	2.589 988 E+02
acre (a)	Hectares	4.046 856 E-01
Volume:		
oz	m^3	2.957 353 E-05
pt	m^3	4.731 765 E-04
qt	m^3	9.463 529 E-04
gal	m^3	3.785 412 E-03
in^3	m^3	1.638 706 E-05
ft^3	m^3	2.831 685 E-02
yd^3	m^3	7.645 549 E-01
Volume per Unit Time:	NOTE: 1m^3 = 1,000 L	
ft^3/min	m^3/sec	4.719 474 E-04
ft^3/s	m^3/sec	2.831 685 E-02
in^3/min	m^3/sec	2.731 177 E-07
yd^3/min	m^3/sec	1.274 258 E-02
gal/min	m^3/sec	6.309 020 E-05
Mass:		
oz	kg	2.834 952 E-02
dwt	kg	1.555 174 E-03
lb	kg	4.535 924 E-01
ton (2000 lb)	kg	9.071 847 E+02
Mass per Unit Volume:		
lb/yd^2	kg/m^2	4.394 185 E+01
lb/in^3	kg/m^3	2.767 990 E+04
lb/ft^3	kg/m^3	1.601 846 E+01
lb/yd^3	kg/m^3	5.932 764 E-01
Velocity: (Includes Speed)		
ft/s	m/s	3.048 000 E-01
mi/h	m/s	4.470 400 E-01
knot	m/s	5.144 444 E-01
mi/h	km/h	1.609 344 E+00
Force Per Unit Area:		
lbf/in^2 or psi	Pa	6.894 757 E+03
lbf/ft^2	Pa	4.788 026 E+01
Viscosity:		
cS$_t$	m^2/s	1.000 000 E-06
P	Pa·s	1.000 000 E-01

Temperature: (°F-32)^5/9 = °C

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INTRODUCTION

For a number of years the Virginia Department of Transportation has used as one of its basic pavement designs a cross section incorporating a 6-in thick soil cement subgrade, a 4- to 8-in thick aggregate base, and 4 or more inches of bituminous concrete. Used to provide a stable pavement foundation in poor soil areas, the soil cement layer normally contains about 10 percent portland cement by volume. The aggregate base is a densely graded material designated in Virginia specifications as No. 21 or No. 21-A or a similarly graded local sand and gravel. In some instances, a layer of cement-treated aggregate is used in lieu of the soil cement-stabilized subgrade. Because of the presence of the weaker aggregate layer between the strong cement-stabilized layer and the bituminous concrete layer, both of the above cross sections have become known as "sandwiched" pavements.

While the widespread use of sandwiched pavements was encouraged, and their generally good performance was documented in earlier research studies (1), it has been noted in recent years that some such pavements have not performed up to original expectations. In the latter cases studied, Virginia strength equivalency values for the aggregate bases have been found to be significantly lower than given in design tables, sometimes to the point of having negative values (2, 3). Several studies with an apparent bearing on the poor performance of some sandwiched pavements have shown (1) that the weak sandwiched layer should be no more than 4-in thick (4); (2) that when the sandwiched layer is weak enough, the strength of the lower sandwiching layer is not mobilized (5); and (3) that aggregate base courses should have a design minus 200 fraction of no more than 7 percent if maximum strength and adequate drainage are to be achieved (7).

In none of the above studies were sufficient data collected to permit the development of strength equivalency values for the weak sandwich layers of various thicknesses and gradations. The present study was undertaken in an effort to correct some of those deficiencies.

PURPOSE AND SCOPE

The purpose of the study was to collect performance and deflection data from in-service pavements containing sandwich layers and to develop appropriate guidelines for the use of such layers. Data were collected from 26 sandwich pavements and 5 non-sandwich (control) pavements over a range of pavement ages and traffic exposures. The pavements studied are fully described in Appendix 1. Note that four of the non-sandwich pavements are constructed of asphaltic concrete on a cement-treated aggregate base underlaid by a soil cement foundation. The fifth is thick (11.5 in) asphaltic concrete directly on the soil cement.

METHODOLOGY

Structural Evaluation

Dynaflect deflection tests were conducted on each pavement segment to determine 5-sensor deflection basin characteristics. Methods developed by Vaswani (5) utilizing the Chevron layer analysis program were employed to determine the subgrade modulus, to examine layer interactions, and to determine strength coefficient for the layers. All deflection data are tabulated in Appendix 2.

Performance Evaluation

At the time deflection tests were conducted, evaluations of the condition of the pavement were performed using methods employed by the Department in the pavement management process (6). Subsequent evaluations by pavement management personnel and the accumulated 18-kip equivalent axle loadings corresponding to each evaluation permitted the development of performance characteristics for those pavements for which full data were available. Detailed evaluation data are tabulated in Appendix 3.

Aggregate Base Evaluations

While it was originally envisioned that aggregate base samples would be collected from projects showing exceptionally good or exceptionally poor performance, it soon became evident that such sharp delineations of performance were not going to materialize. Since it was not within the scope of the study to collect and analyze samples from all projects, it was decided that project records and data base files for the sources of the materials would be used. While it was recognized that such file data would be subject to more uncertainty than would samples from the road, it is also clear that statistically it would be better to have some data from all projects. Aggregate base samples taken from sites 4 and 22, both of which were especially poor performers with around 9 percent minus 200, showed that the bases were saturated due to poor drainage characteristics and to the "bathtub" designs.

The data of primary interest, gradations and Atterberg limits, are tabulated in Appendix 4. The alphabetical source codes in that tabulation refer to commercial stone producers whose names are available in Research Council files.

The gradation design ranges for the No. 21-A aggregate base material used in virtually all sandwiched projects is given in Table 1. The Department's specifications provide for suitable tolerances (± 3%) once the producer chooses his job mix from the values given in Table 1 (8). These tolerances, then, provide an extremely wide operating range that can result in large variations in the end product from different producers. For example, the percentage passing the No. 200 sieve can range from 5 to 15 percent for individual test results.

Table 1

Design Ranges for No. 21-A Base Material

Sieve	Percentage Passing (by weight)
2"	100
1"	94-100
3/8"	63-72
No. 10	32-41
No. 40	16-24
No. 200	8-12

RESULTS AND DISCUSSION

Structural Evaluation

Detailed analysis of deflection data showed that the sandwiched pavements do not develop the design structural strength as measured by the thickness index (2, 4). The contrast between sandwiched and non-sandwiched pavements in their ability to develop the design structural strength is shown clearly in Table 2.

Table 2

Structural Strength Development

	Design Thickness Index		Effective Thickness Index		Design Efficiency	
	Sandwich	Non-Sandwich	Sandwich	Non-Sandwich	Sandwich	Non-Sandwich
Average	10.7	15.8	6.7	16.6	62.6	105.0
Std. Dev.	1.4	1.2	1.9	1.9	--	--

Note in Table 2 that, on the average, the sandwiched designs were able to develop only 62.6 percent of the design structural strength while the non-sandwiched pavements averaged 105 percent.

While a major purpose of the present study was to develop strength equivalency values for sandwiched pavement systems, attempts to do so on the sandwich pavements studied were unsuccessful. If asphaltic concrete and cement-treated layers are assumed to have developed their expected strengths, then the aggregate base course contribution to overall pavement strength was negative in nearly every case; i.e., the pavements were better off without the aggregate base layer. The computed average equivalency of the aggregate base, based on the data in Table 2 and Appendix 1, is -0.30 as opposed to a design value of 0.35. Since such a conclusion is ridiculous, it is the author's contention that the observation made by Vaswani in earlier studies is true: the strength of the lower sandwiching layer is not mobilized and does not realize its design strength (5). It is clear that no meaningful equivalency values can be developed from the data collected in the present study.

The deflection characteristics from which the effective thickness index values were developed are given in detail in Appendix 2 and are summarized in Table 3.

Table 3

Average Deflection Characteristics

Pavement Type	Maximum Deflection (in)	Spreadability	Subgrade Modulus	Pavement Thickness (in)
Sandwich	.026	55.4	9400	17.5
Non-Sandwich	.012	79.0	12000	19.5

The data clearly shows the large difference in maximum deflection and in the spreadability values between the sandwiched and non-sandwiched pavements. The deflection difference is no doubt partially due to small differences in subgrade strength, pavement thickness, and layer compositions. Statistical analysis, however, shows that aggregate base gradation, particularly the percentage passing the No. 200 sieve, is a major contributor to the total deflection. The multiple regression equation of best fit is

$$\text{DEFL} = 0.0044\,(\text{PP200}) - 1.8 \times 10^{-6}\,(\text{ES}) \qquad (1)$$

where

DEFL = deflection

PP200 = percentage of aggregate base passing the No. 200 sieve

ES = subgrade modulus.

The coefficient of determination (R^2) of equation (1) is 0.879, indicating a significant relationship at a 99 percent confidence level for the 26 sandwich pavements studied. A similar analysis for the 5 non-sandwiched pavements shows that the deflection is a function only of the subgrade modulus and the pavement thickness.

The deflections predicted from equation (1) are plotted in Figure 1 as a function of the measured deflections. While the figure shows a strong overall relationship between measured and predicted deflection, it is also evident that due to variation unexplained by the equation, the equation would not be a good deflection predictor for an individual project.

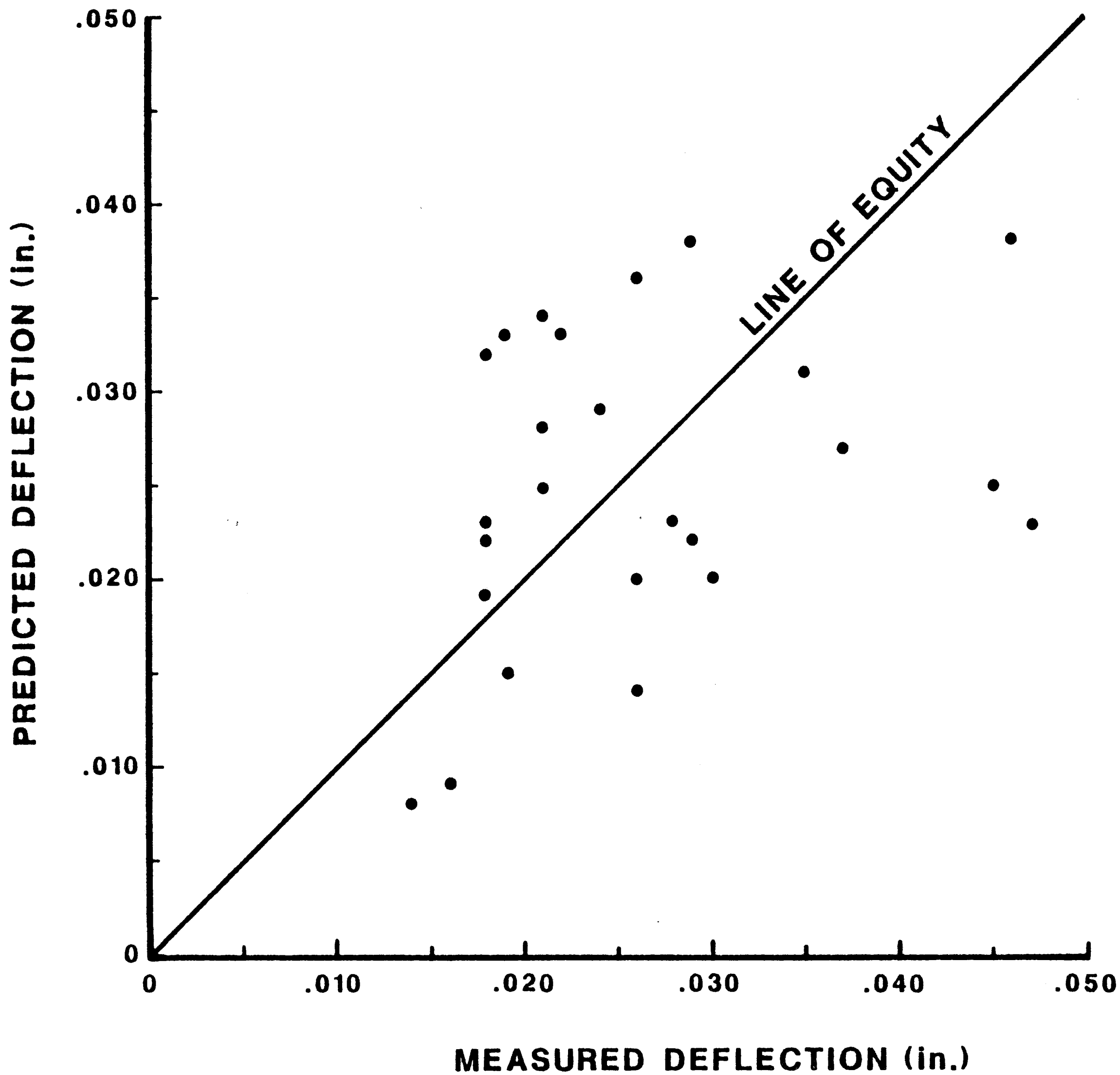

Figure 1. Predicted vs. measured deflection for sandwich projects.

Performance Evaluation

Performance equations for both the sandwich and non-sandwich pavements were developed through methods described by the author in an earlier study (6). In that study, the general form of a pavement deterioration equation was:

$$DMR = 100 - A\ (ESAL)^{B} \qquad (2)$$

DMR is the distress maintenance rating, Virginia's measure of pavement surface condition computed through a deduct system comprised principally of cracking and rutting distresses. The coefficient (A) and exponent (B) are load and design variables for a particular pavement, and ESAL is the cumulative 18-kip axle loading sustained by the surface at the time the DMR is determined.

A weighted averaging of the detailed performance data given in Appendix 3 yielded equations (3) and (4) for the sandwich and non-sandwich pavements, respectively.

$$\text{Sandwich pavements:} \quad DMR = 100 - 56.7\ (ESAL)^{1.21} \qquad (3)$$

$$\text{Non-sandwich pavements:} \quad DMR = 100 - 3.47\ (ESAL)^{2.33} \qquad (4)$$

Note that these equations reflect average values and do not purport to represent particular pavements. They do, however, show significant differences in the performance behavior of the two pavement types. Their difference is shown graphically in Figure 2 in which both deterioration curves are plotted using the initial average daily 18,000 lb equivalent axle loads (ESAL-18) for each pavement type. These initial values were 105 and 393 for the sandwich and non-sandwich, respectively. A 5 percent annual traffic growth rate was used, and the curves depict DMR as a function of age to normalize the data and demonstrate the performance difference between the two pavement types. According to Virginia's design procedure both pavement types should perform similarly with traffic differences accommodated by structural differences. However, when judged on the basis of time required to reach the terminal DMR of 78 (9) used in Virginia for rehabilitation of primary system flexible pavements, it can be seen that there is an average two-year difference in performance in favor of the non-sandwich pavements.

An examination of the age-to-terminal DMR data in Appendix 3 shows that the life expectancy of the sandwich pavements is much more variable than the non-sandwich, but that the two year average difference is statistically significant at over the 90 percent confidence level. Given the large differences in pavement response to loads measured by deflection, the differences in performance are not surprising.

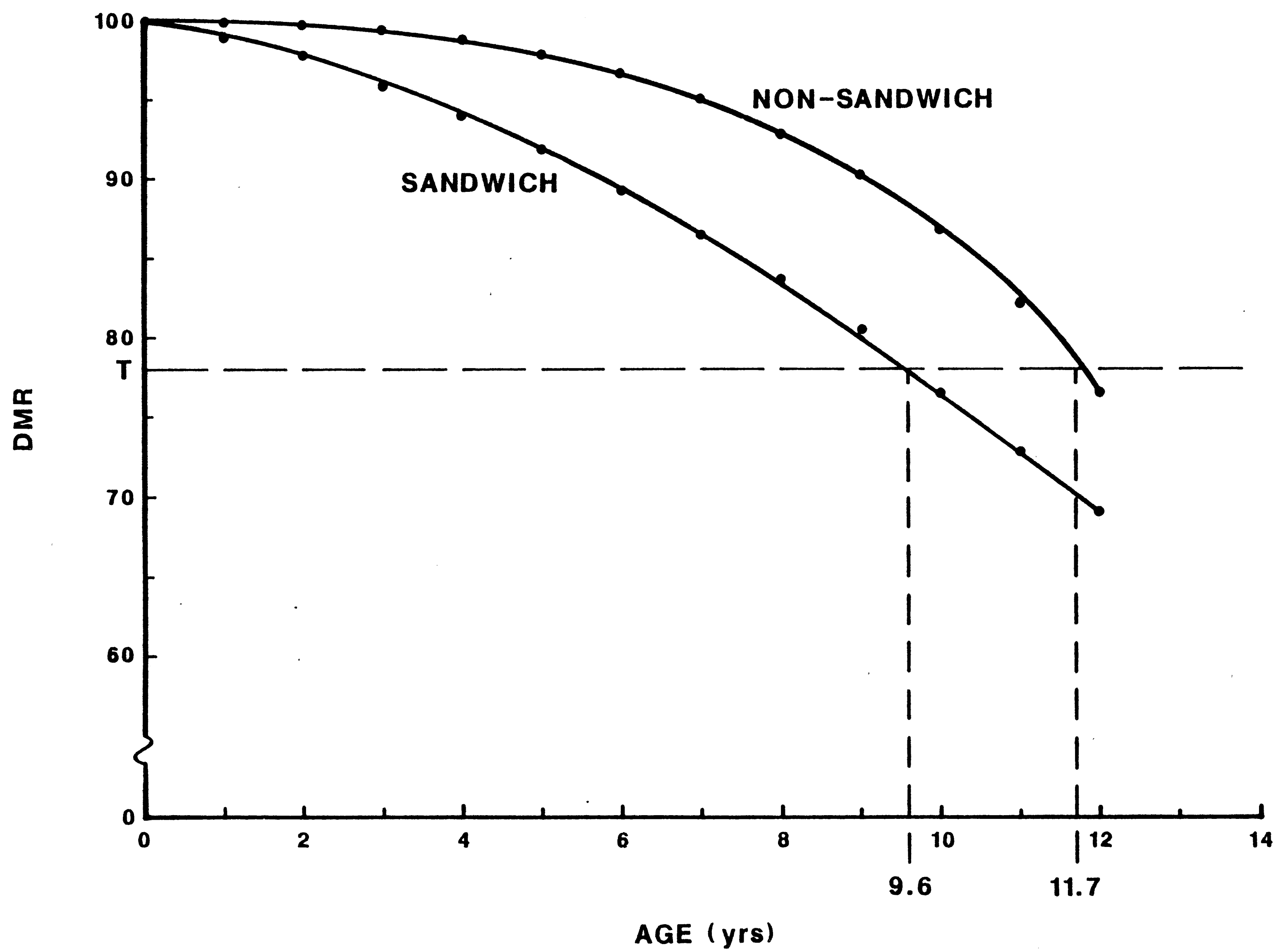

Figure 2. Pavement deterioration curves.

Relationship Between Pavement Performance and Aggregate Base Properties

In an effort to identify possible relationships between performance of the sandwich pavements and the properties of the aggregate base materials, a multiple regression analysis of the data given in Appendices 1 through 4 was performed. Specifically, the analysis was directed at determining statistically significant relationships between the performance equation parameters A and B in equation (2) and traffic, design, and materials variables. The significant relationships identified are given in equations (5) and (6) where LNA is the natural logarithm of the coefficient (A), ACT is the thickness of the asphaltic concrete portion of the pavement, PP200 is the percentage of the aggregate base passing the No. 200 sieve, and LNB is the natural logarithm of the exponent (B).

$$\mathrm{LNA} = 0.949\ (\mathrm{PP200}) - 0.839\ (\mathrm{ACT}) \quad (5)$$

$$\mathrm{LNB} = 0.208\ (\mathrm{LNA}) - 0.645 \quad (6)$$

Equations (5) and (6) have coefficients of determination (R^2) of 0.894 and 0.274, respectively. While both are highly significant with 24 degrees of freedom, it is clear that the interaction of variables is much better explained by equation (5) than by equation (6). That is, there is a strong relationship among the performance curve parameter (A), the interaction of asphaltic concrete thickness, and the percentage of the aggregate base passing the No. 200 sieve. The relationship between A and B, while statistically significant, could not be used with confidence in predicting pavement performance.

In addition to the low R^2 for equation (6), equations (5) and (6) have another serious limitation in that the pavements studied had aggregate bases with a relatively narrow range (7.2 to 12.2) of values for the percentage of aggregate passing the No. 200 sieve. Thus, the equations are not representative of the gradation of aggregate bases having less than 7 percent of the aggregate passing the No. 200 sieve, where one would expect the best pavement performance (based on other studies where both strength and drainage were shown to be much improved) (7). Nevertheless, an analysis of equations (5) and (6), which were used to develop performance equations for 8 percent and 10 percent passing the No. 200 sieve with other factors being equal, shows that performance would be much improved if bases with fewer fines were provided. This is shown graphically in Figure 3 where the average asphaltic concrete thickness (6 1/4 in) and average prevailing daily 18-kip equivalent axle loads (288) for the 26 sandwich projects were used.

The reader is cautioned that the dramatic difference in projected pavement performance indicated in Figure 3 is based on statistically determined relationships and for that reason is subject to question. However, it is clear from that figure and from the strong adverse effect on deflections of high percentages of minus 200 base material that better pavement performance could be expected if the minus 200 fraction was restricted to the levels indicated in earlier studies (7).

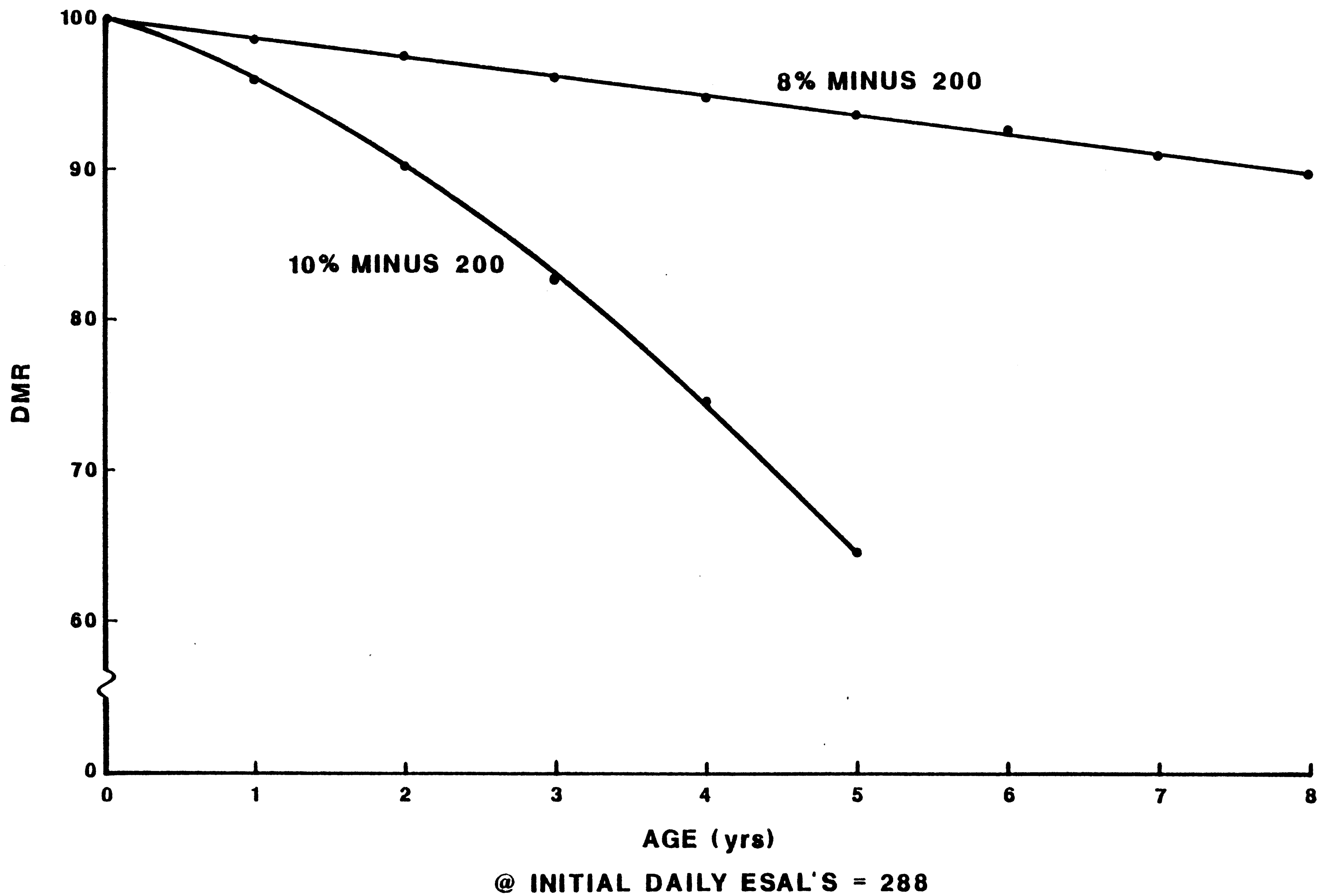

Figure 3. Pavement deterioration as effected by percentage passing the no. 200 sieve for AC thickness = 6 1/4 inches.

CONCLUSIONS

Taking account of the limitations of the data available for analysis in this study, the following conclusions appear to be justified.

1. The life expectancy of pavements constructed with sandwiched layers is significantly less than for those constructed in a more conventional manner where the strength of each layer is no greater than that of its overlying layer.

2. The average difference in life expectancy between sandwich and non-sandwich pavements is approximately two years in favor of the non-sandwiched.

3. The performance characteristics of sandwich pavements are significantly influenced by the percentage of aggregate base material passing the No. 200 sieve. Indications are that when the sandwich bases have no more than about 8 percent minus 200, performance will be similar to that for non-sandwich pavements. Higher percentages of minus 200 have a dramatic negative impact on pavement performance.

RECOMMENDATIONS

In view of the conclusions enumerated above, it is evident that serious consideration should be given to more widespread use of aggregate bases containing less than 8 percent minus 200. Such an aggregate base was developed in earlier studies based on an optimization of drainability and strength (7). The base material, designated No. 21B, appears in the latest specifications of the Department and has a design minus 200 range of 6 percent to 8 percent (9). As was noted at the time that specification was developed, widespread use of the No. 21B material would affect construction because it is slightly more difficult to compact and it would also affect some producers who already have excessive fines.

Nevertheless, when the sandwich type of design is indicated for other reasons, the author recommends that Department managers and pavement designers seriously consider the use of the 21B aggregate base as the standard. There appears to be no discernable problem with the 21A material for other types of design, although better drainage would be expected with the 21B in all cases.

GUIDELINES FOR USE OF SANDWICHED PAVEMENT DESIGNS

Results of the study were such as to severely limit the anticipated development of guidelines for the use of sandwiched pavements. As a consequence, the author offers only the advice that the Department should attempt to restrict the minus 200 fraction of aggregate bases used as sandwiched layers to no more than 8 percent.

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APPENDIX 1

SITE DESCRIPTIONS

Site No.	Route	County	Project Number	Direction	MP From	MP To	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Subbase Type	Subbase Thickness (in.)	Design Thickness Index	Completion Date	First Overlay Type	First Overlay Date
1	23	Scott	0023-084-108, C501	SBL	7.88	11.52	5.2	6.0	SC	6.0	9.7	8-24-71	I-2	1980-82
2	23	Scott	0023-084-108, C501	NBL	7.88	11.52	5.2	6.0	SC	6.0	9.7	8-24-71	I-2	1980-82
3	23	Lee	6023-052-102, C501	SBL	0.00	4.44	4.5	4.0	CTB	6.0	9.5	3-26-75	I-2	1982
4	23	Lee	6023-052-102, C501	NBL	0.00	4.44	4.5	4.0	CTB	6.0	9.5	3-26-75	I-2	1982
5	460	Bedford	6297-009-103, C503	EBL	9.55	11.88	7.0	4.0	CTB	6.0	12.0	12-07-69	S-5	1982
6	460	Bedford	6297-009-103, C501	EBL	0.00	9.55	7.0	4.0	CTB	6.0	12.0	11-13-70	S-5	1982-83
7	460	Bedford	7460-009-101, C501	EBL	14.06	15.38	7.0	4.0	CTB	6.0	12.0	07-03-70	SS	1979
8	460	Pr. Edward	6460-073-109, C501	WBL	0.00	3.48	4.5	6.0	SC	6.0	9.0	07-01-74	SS	1982
9	460	Pr. Edward	0460-073-105, C501	WBL	16.00	23.00	5.5	6.0	SC	6.0	10.0	07-01-74	I-2	1983
10	15	Pr. Edward	0460-073-101, C501	SBL	1.75	6.88	5.5	6.0	SC	6.0	10.0	05-12-76	S-5	1976
11	29	Campbell	7029-015-101, C501	SBL	2.41	5.22	7.5	6.0	SC	6.0	12.0	09-01-73	SS	1983
12	29	Campbell	7029-015-101, C501	NBL	0.00	2.41	7.5	6.0	CTB	4.0	12.0	09-01-73	SS	1983
13	29	Pittsylvania	7029-071-111, C501	NBL	41.20	43.36	7.5	6.0	SC	6.0	12.0	09-01-73	SS	1983
14	29	Pittsylvania	7029-071-111, C501	SBL	43.36	45.52	7.5	6.0	CTB	4.0	12.0	09-01-73	SS	1983
15	29	Pittsylvania	7029-071-101, C501	NBL	31.09	35.84	7.5	6.0	SC	6.0	12.0	11-01-74	SS	1984
16	29	Pittsylvania	7029-071-101, C501	SBL	31.09	35.84	7.5	6.0	SC	6.0	12.0	11-01-74	SS	1984
17	60	Powhatan	0060-072-007, C501	WBL	4.29	8.90	4.5	6.0	SC	6.0	9.0	08-25-71	SS	1980
18	60	Powhatan	0060-072-101, C501	WBL	9.02	11.06	4.5	6.0	SC	6.0	9.0	05-28-74	S-5	1983
19	60	Powhatan	0060-072-003, C501	WBL	0.00	4.29	4.5	6.0	SC	6.0	9.0	08-25-71	SS	1980
20	58	Mecklenburg	0058-058-111, C501	WBL	34.81	36.38	5.5	6.0	SC	6.0	10.0	05-19-71	S-5	1985
21	3	Stafford	0003-089-102, C502	EBL	4.54	7.20	4.5	4.0	SC	6.0	8.3	09-02-74	None	
22	207	Caroline	0207-016-102, C501	NBL	0.99	5.26	8.0	6.0	SC	6.0	12.4	11-02-71	S-5	1977
23	29	Madison	6029-056-107, C501	SBL	9.43	14.17	8.0	6.0	SC	6.0	12.4	11-06-75	S-5	1980
24	17	Fauquier	6017-030-103, C501	SBL	43.76	49.42	7.5	6.0	SC	6.0	12.0	07-01-74	None	
25	29	Albemarle	6029-002-112, C501	SBL	10.29	16.08	4.5	6.0	SC	6.0	9.0	11-06-75	None	
26	29	Albemarle	6029-002-103, C501	SBL	0.00	4.73	8.0	4.0	SC	6.0	11.7	08-08-74	S-5	1984
NS1	58	Suffolk	6058-061-105, C501	WBL	8.49	11.89	7.5	6.0(CTB)	SC	6.0	15.9	09-27-74	None	
NS2	58	Suffolk	6058-061-105, C501	EBL	8.49	11.89	7.5	6.0(CTB)	SC	6.0	15.9	09-27-74	None	
NS3	58	Suffolk	6058-061-106, C501	WBL	6.22	8.57	8.5	6.0(CTB)	SC	6.0	16.8	01-24-74	None	
NS4	58	Suffolk	6058-061-106, C501	EBL	6.22	8.57	8.5	6.0(CTB)	SC	6.0	16.8	01-24-74	None	
NS5	207	Caroline	0207-016-102, C501	SBL	5.26	10.15	11.5	-	SC	6.0	13.8	07-01-73	I-2	1984

APPENDIX 2

DEFLECTION DATA

Site No.	Date Deflection Tests	Temperature	No. Tests	Maximum Deflection (in.)	Spreadability	Effective Thickness Index	Percent Design Thickness Index	Subgrade Modulus
1	June 1981	108	34	0.014	46	5.9	61	18000
2	June 1981	79	36	0.016	48	6.0	62	17000
3	June 1981	96	25	0.019	49	5.9	62	14000
4	June 1981	96	23	0.026	50	5.2	55	11000
5	May 1981	86	34	0.018	66	10.4	87	9000
6	May 1981	79	72	0.021	60	8.4	70	6000
7	May 1981	79	52	0.021	66	9.8	82	8000
8	April 1981	75	66	0.029	53	5.5	61	8000
9	April 1981	75	105	0.037	60	6.0	60	5000
10	March 1983	70	72	0.045	51	3.8	38	6000
11	March 1983	60	30	0.024	51	5.7	48	11000
12	March 1983	62	30	0.022	59	7.9	66	9000
13	March 1983	60	30	0.019	64	9.8	82	9000
14	March 1983	62	30	0.026	63	8.2	68	7000
15	May 1981	90	44	0.029	53	5.9	49	8000
16	May 1981	72	50	0.046	42	2.2	18	8000
17	Sept. 1982	100	10	0.025	51	5.6	62	11000
18	Sept. 1982	100	24	0.047	52	3.8	42	5000
19	Sept. 1982	90	46	0.030	56	6.1	68	8000
20	Oct. 1982	72	20	0.035	63	6.9	69	5000
21	June 1981	102	45	0.020	61	8.8	106	9000
22	June 1981	108	51	0.018	54	7.4	60	12000
23	May 1981	73	65	0.018	60	9.0	73	10000
24	June 1981	89	72	0.028	54	5.9	49	8000
25	May 1981	90	60	0.018	54	7.4	82	17000
26	May 1981	87	36	0.021	53	6.6	56	11000
NS1	Oct. 1982	106	28	0.010	84	19.3	121	20000
NS2	Oct. 1982	106	26	0.010	82	18.6	117	20000
NS3	Oct. 1982	60	25	0.016	78	14.5	86	5000
NS4	Oct. 1982	82	22	0.015	81	15.7	93	5000
NS5	June 1981	102	51	0.009	70	15.1	109	10000
Avg. (Sand)				0.026	55.4	6.70	62.9	9400
			Std. Dev. =	0.009	6.2	1.93	17.1	3300
Avg. (NS)				0.012	79.0	16.64	105.2	12000
			Std. Dev. =	0.003	4.9	1.94	13.6	7600

APPENDIX 3

PAVEMENT PERFORMANCE DATA

Site No.	1985 Daily ESAL-18	Rating 1 Mo - Yr	Rating 1 DMR	Rating 2 Mo - Yr	Rating 2 DMR	Constant A	Constant B	Surface Rated	Years to Term. DMR
1	150	2 - 72	100	6 - 81	74	96.5	1.21	Original	8.6
2	150	2 - 72	100	6 - 81	83	53.2	1.05	Original	12.2
3	92	9 - 75	100	6 - 81	82	446.0	1.67	Original	6.8
4	92	9 - 75	100	6 - 81	82	446.0	1.67	Original	6.8
5	405	6 - 70	100	5 - 81	83	16.8	0.83	Original	15.1
6	405	5 - 71	100	5 - 81	87	13.0	0.82	Original	18.9
7	426	5 - 81	92	11 - 83	82	30.1	0.91	Original	5.4
8	276	1 - 75	100	4 - 81	75	81.7	1.56	Original	4.3
9	299	1 - 75	100	4 - 81	87	29.7	1.24	Original	7.1
10	327	11 - 76	100	3 - 83	80	41.6	1.51	Original	7.2
11	431	3 - 74	100	3 - 83	76	22.7	1.15	Original	8.9
12	431	3 - 74	100	3 - 83	79	19.9	1.10	Original	9.8
13	409	3 - 74	100	3 - 83	82	12.0	0.90	Original	17.4
14	409	3 - 83	88	2 - 85	84	20.0	0.22	Slurry	10.7
15	467	3 - 83	100	2 - 85	94	22.3	1.75	Original	6.2
16	467	3 - 83	99	2 - 85	89	63.9	2.35	Original	4.1
17	83	12 - 82	95	2 - 85	90	85.6	1.02	Slurry	9.4
18	83	11 - 74	100	12 - 82	88	76.9	1.10	Original	14.2
19	83	12 - 82	98	2 - 85	89	2163.0	2.51	Slurry	6.0
20	106	1 - 83	84	2 - 85	78	91.1	1.45	Original	13.7
21	109	11 - 82	93	12 - 84	87	172.0	2.20	Original	12.5
22	282	10 - 82	93	12 - 84	91	12.2	0.63	Overlay	5.6
23	408	10 - 83	94	1 - 85	87	41.6	2.21	Overlay	5.7
24	321	11 - 82	86	2 - 85	82	18.9	0.85	Original	12.9
25	343	6 - 76	100	5 - 81	91	17.5	0.93	Original	13.6
26	335	5 - 81	81	12 - 82	63	84.7	2.65	Original	7.1
NS1	586	12 - 82	95	2 - 85	86	2.2	3.55	Original	11.7
NS2	586	12 - 82	95	2 - 85	86	2.2	3.55	Original	11.7
NS3	830	12 - 82	88	2 - 85	83	5.3	1.27	Original	12.8
NS4	830	12 - 82	93	2 - 85	81	0.7	3.65	Original	11.2
NS5	282	6 - 74	100	6 - 81	86	28.0	1.17	Original	11.0
							Average (Sandwich)		9.6
S	284						Std. Dev.		4.0
NS	623								
							Average (Non-Sandwich)		11.7
							Std. Dev.		0.6

APPENDIX 4

AGGREGATE BASE DATA

Site No.	Base Designation	Source Code	Percent Passing #200	Plasticity Index
1	21A	A	9.0	NP
2	21A	A	9.0	NP
3	21A	A	9.0	NP
4	21A	A	9.0	NP
5	21A	B	8.9	NP
6	21A	B	8.9	NP
7	21A	B	8.9	NP
8	21A	C	8.1	NP
9	21A	C	8.1	NP
10	21A	C	8.1	NP
11	21A	D	11.1	NP
12	21A	D	11.1	NP
13	21A	D	11.1	NP
14	21A	D	11.1	NP
15	21A	E	11.9	NP
16	21A	E	11.9	NP
17	21A	F	7.8	NP
18	21A	G	7.2	NP
19	21A	F	7.8	NP
20	21A	H	9.2	NP
21	SM-GR1	I	N/A	N/A
22	21A	J	9.3	NP
23	21A	K	9.0	NP
24	21A	L	8.4	NP
25	21A	M	12.2	NP
26	21A	M	12.2	NP