pavements have center-line stripes, whereas only about 90 percent of those paved with asphalt use a center-line stripe. The average life of the center-line stripe on country roads is 2.5 years. This compares with 2.2 years in 1960.

Sealing joints and cracks on PCC pavement has been somewhat controversial over the years. Of the 42 counties reporting, all of them sealed their cracks and joints. The average sealing rate is approximately every 3.5 years as compared with every 2 years 20 years ago. This rate change may be due to progress made in types of material used for that purpose or it may be evidence of today's economic situation.

The questionnaire requested information on the major problems on different types of roads. On earth roads, the major problem was that the roads were built to poor standards (62 of 85 counties). Perhaps one could even say that some had not been built to any standards at all. There was a scattering of other subjects such as narrow right-of-way and insufficient and too small drainage culverts.

The major problem on loose-surfaced roads was too much traffic for the type of surface, which is the same as it was 20 years ago. The preponderance of this problem was not quite so large as 20 years ago. The second and third problems dealt with poor granular surfacing material and poor grades. This follows closely the comments of 20 years ago.

Fifty-five of 80 counties reported that frost action was their major problem on bituminous-surfaced or paved roads. This also was the leading problem 20 years ago. The second major problem listed 20 years ago was too heavy traffic, and this is true today also. The third major problem was numerous breakups due to insufficient base, and this concurs with 20 years ago. One item mentioned frequently that was not a problem 20 years ago was insufficient funds. Surprisingly, it was the major problem today.

The counties were asked whether they had sufficient equipment to clear the snow from the highways. Ninety to ninety-five percent indicated that they did not have an equipment problem. This has improved, since 20 years ago 25 percent indicated a need for additional equipment.

Bridges were a problem 20 years ago but have certainly had the spotlight for the last 10 years or so because of the national emphasis. There was an increase in the concern that there was not an adequate bridge-replacement program. Twenty years ago, 32 counties stated that they had an adequate bridge-replacement program, whereas only 24 counties did this time. Fifty-three counties stated that they did not have adequate bridge-replacement programs 20 years ago; today, 65 counties reported that they did not.

The average number of bridges per county is 259. This figure is significant, since 20 years ago the average number of bridges per county was 390; there are 131 less bridges per county today. Approximately 13 000 bridges have been eliminated. Part of this problem may be explained by the criteria used in 1960, which were to identify bridges more than 12 ft in length. Today, one of the significant factors is the definition of a bridge by federal criteria as 20 ft in length. This can affect considerably the number of bridges reported. In addition, there has been an effort to eliminate bridges that were not needed, and many bridges have been replaced by culverts that are less than 20 ft in span. In 20 years, the number of bridges has been reduced by approximately one-third.

The major problem with bridges is a deficiency in width. Eighty-five of 90 counties reporting indicated that an average of 60 percent were deficient in width. Eighty-three counties reported that approximately 50 percent of their bridges were deficient in load capacity.

There are some significant statistics in these questionnaires. Some of the answers are not explainable. The questions that arise are the same questions as 20 years ago. Are we planning our maintenance as we should? Are we keeping records of what we are doing for an efficient maintenance operation? There appears to be little change in the problems of 20 years ago. There is still much room for improvement. The new terms for today are "maintenance management system" or "pavement management system." That sophistication is probably not applicable to the type of roads we are talking about. The process may not be one that can be readily utilized by those who operate the low-volume roads, but the concept and the needs are still there. Perhaps with time we will find and arrive at practical and simpler processes to aid in improving maintenance operations for the operators of low-volume road systems.

REFERENCE

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Simplified Design Approach to Surface Treatments for Low-Volume Roads

PRITHVI S. KANDHAL

The use of surface treatment as an economical maintenance technique to preserve the service life of the existing pavements has increased substantially in recent years. Although many surface treatment design methods have been developed in the past on a rational basis, a vast majority of highway agencies, in both developed and developing countries, still use the quantities of binder and cover aggregate determined by experience and/or precedent and this often results in surface treatments that have poor performance characteristics. This is due primarily to the fact that most of these design methods involve time-consuming or complex test procedures and/or computations. A need was felt to develop a simplified rational design method especially for the low-volume, low-cost roads that could be used at the local level by the county maintenance managers and contractors. This has been accomplished in four phases: (a) literature review of the existing design procedures, (b) construction of field research projects, (c) laboratory experiments to correlate the complex and simple test properties of the materials, and (d) analysis of field and laboratory data to develop a simple nomographic design method. This design method has been used extensively for low-volume roads by the contractors and county maintenance forces in Pennsylvania during the 1980 and 1981 construction seasons with apparent success.

The use of surface treatments to preserve the service life of existing pavements has increased substantially in recent years. Such an increase in the use of this method of construction can be attributed to the growing number of miles of low-volume roads throughout the world and to the need for providing economical maintenance techniques to retain desired pavement performance levels. However, the economic advantage of surface treatments may easily be lost through faulty design.

Although many design methods have been developed on a rational basis in the past, a vast majority of highway agencies, in both developed and developing countries, still use the quantities of binder and cover aggregate determined by experience and/or precedent and this often results in surface treatments that have poor performance characteristics. This is primarily due to the fact that most of these design methods involve time-consuming or complex test procedures and/or computations. This has discouraged their use, especially for the low-volume, low-cost roads.

A need was felt by the Pennsylvania Department of Transportation (PennDOT) to develop a simplified rational design method that could be used at the local level by the county maintenance managers and contractors on a routine basis.

The term "surface treatment" in this paper implies a single application of bituminous binder followed by a single application of cover aggregate, both placed on an existing bituminous surface. "Single" surface treatment is more widely used than "double" and "triple" surface treatments.

Although one-size cover aggregate is ideal for surface treatment, it is costly to produce. Normally, a graded cover aggregate is used in North America for low-volume, low-cost roads. The data in this paper are based on a graded aggregate--3/8 in to No. 8 size (equivalent AASHTO M43 or ASTM D448, Size 8)--unless otherwise stated.

REVIEW OF EXISTING DESIGN PROCEDURES

An excellent review of surface treatment design procedures existing prior to 1968 was made by Herrin, Marek, and Majidzadeh (1). These include the following design methods: (a) Hanson, (b) California, (c) Nevitt, (d) modified Kearby, (e) Lovering spread-modulus, (f) European, (g) McLeod, (h) Mackintosh, (i) American Bitumul, and (j) Asphalt Institute.

Subsequently, Marek and Herrin (2) presented a surface treatment design procedure based on the measured voids content of an aggregate layer and the depth of embedment of the aggregate into the underlying surface. McLeod presented an excellent paper (3) in 1969 for designing surface treatments by using one-size and graded cover aggregates. His 1974 paper (4) is basically similar except that it is adapted to the use of asphalt emulsions.

The design procedures considered for developing a simplified approach for low-volume roads will be described briefly.

California Design Method

Hveem, Lovering, and Sherman (5) developed nomographs to estimate the quantity of the graded cover aggregate and the binder for surface treatment. Although no data are available on the development of the nomographs, this approach is attractive because it avoids complex computations. The following information is required to use the nomographs:

1. Effective maximum size of the aggregate (theoretical sieve size in inches that would allow 90 percent of the aggregate to pass through the sieve openings),
2. Loose unit weight of the aggregate,
3. Condition of underlying surface, and
4. Porosity of the aggregate [centrifuge kerosene equivalent test is used to determine the surface factor (K_C)].

Lovering Spread-Modulus Design Method

Lovering (6) determined that the direct measurement of individual aggregate particles for determining the average mat thickness was not feasible for a graded cover aggregate. He determined that there was a satisfactory correlation between the loose volume of cover aggregate required to produce a layer one stone thick and the "mean particle diameter," defined as the weighted average of the mean size of the largest 20 percent of the aggregate, the middle 60 percent, and the smallest 20 percent. The mean particle diameter was called by Lovering the spread modulus. He concluded that a factor of 0.85-0.95 times the spread modulus would provide the proper quantity of aggregate in cubic feet per square yard with a reasonable allowance for both compaction and whip-off. Accordingly, he suggested the following formula:

$$S = 0.9M \tag{1}$$

where S is the cubic feet of aggregate per square yard and M is the spread modulus.

The required amount of bituminous material can be calculated from the following formula:

$$A = 0.4M + V \tag{2}$$

where A is the required amount of bituminous material in gallons per square yard and V is the quantity of bituminous material in gallons per square yard to allow for absorption by the underlying surface.

American Bitumul Design Method

Kari, Coyne, and McCoy (7) derived the following formulas theoretically:

$$B = 0.708D + P_c \tag{3}$$

$$W_A = 80D \tag{4}$$

where

B = binder quantity to be applied (gal/yd²);
D = average stone diameter (in) as obtained from gradation analysis; it appears that the value used is the 50 percent passing size;
P_C = pavement rating factor (existing pavement surface condition) (gal/yd²); and
W_A = weight of applied stone cover (lb/yd²).

The following assumptions were made in arriving at these formulas:

1. An aggregate specific gravity of 2.65 was used, and
2. The aggregate particles were assumed to be of a spherical shape.

The weight of the aggregate required to form a layer one stone in thickness uniformly distributed over a specified area was also determined in laboratory experiments. The linear relationship between average stone size (D) and aggregate cover spread (W_A) was determined to be W_A = 68D. To allow for spreading inequalities and losses, the experimenters increased the aggregate quantity by 15 percent; therefore the formula $W_A = 80D$.

Asphalt Institute Design Method

For graded aggregate, the Asphalt Institute (8) recommends the following formulas:

$$S = 0.80MW \quad (5)$$

$$A = 1.122MT + V \quad (6)$$

where

S = aggregate spread (lb/yd²),
W = loose unit weight of aggregate (lb/ft³), and
T = traffic factor (8, Table C-2).

McLeod's Design Method

McLeod (3) has recommended the following formulas for designing surface treatment with graded cover aggregate:

$$C = 46.8(1 - 0.4V)HGE \quad (7)$$

$$B = (2.244HTV + S + A)/R \quad (8)$$

where

C = aggregate spread (lb/yd²),
V = fraction of voids in aggregate in loose condition,
= 1 - (W/62.4G),
G = bulk specific gravity of aggregate,
H = average least dimension (ALD) of aggregate determined from median size and flakiness index,
E = wastage factor for aggregate loss due to whip-off by traffic and uneven spread,
S = surface texture correction for existing surface (gal/yd²), and
R = fraction of residual asphalt in bituminous binder.

FIELD EXPERIMENTS

Research Project 71-11 was planned in 1971 to attempt a rational design approach for constructing surface treatments. Two projects were constructed, one in 1972 and one in 1973.

1972 Field Project

The 1972 project, consisting of three sections (total, 1.8 miles), was constructed in August 1972 on Pennsylvania Route 346 east of Derrick City in McKean County. The road consists of two lanes and had an average daily traffic (ADT) that ranged from 900 to 1610 vehicles in 1972. It was decided to use McLeod's design method to compute the application rates. Although Pennsylvania No. 1B aggregate is used in the state for surface treatments, experimental use of No. 1NS aggregate was also included. Two sections, A and B, that used 1NS and 1B aggregates, respectively, were designed by McLeod's method. Section C was constructed based on local maintenance practice for comparison. Road tar (RT-9) was used as binder.

The test data on the materials and the design factors used are given in Table 1. The data indicate that the 1NS aggregate was coarser (1/2-in sieve) than specified, and also its flakiness index of 35 was marginal.

The design (desired) application rates and the actual application rates (measured in the field) of the binder and cover aggregates are given in Table 2. The rates of application based on the nomograph method developed subsequently are also included for comparison and will be discussed later.

The weather at the time of construction was sunny and the ambient temperature ranged from 48 to 72°F. Rolling was accomplished by a steel-wheel tandem roller closely followed by a pneumatic-tired roller. The performance of these sections was evaluated and is given in Table 2. The extent of bleeding was moderate on section B and none to slight on sections A and C. Estimated loss of the cover aggregate ranged from 5 to 10 percent (Table 2).

1973 Field Project

The 1973 project consisted of six sections (total, 6 miles) and was constructed in September 1973 on Route 346 in McKean County between Duke Center (east end) and the intersection with Route 446. The two-lane highway had an ADT of 1610 vehicles in 1973. The existing surface was badly pocked, porous, and oxidized.

Three bituminous materials (250-300 penetration-

Table 1. Design data, 1972 field project.

	Aggregate	
Item	1NS	1B
Aggregate type	Gravel	Gravel
Gradation (% passing)		
1 in	100 (100)[a]	-
3/4 in	90	-
1/2 in	49 (90-100)	100 (100)
3/8 in	10	87 (75-100)
No. 4	3 (0-15)	25 (10-30)
No. 8	-	3 (0-10)
Loose unit weight W (lb/ft³)	89.9	90.5
Specific gravity G	2.63	2.63
Flakiness index (%)	35.0	19.2
Median size (in)	0.50	0.24
Average least dimension H (in)	0.32	0.185
Wastage factor E	1.05	1.05
Traffic factor T	0.65	0.65
Correction for surface texture S (gal/yd²)	0.03	0.03
Aggregate absorption correction A (gal/yd²)	0.05	0.05
Bituminous material type	Road tar (RT-9)	Road tar (RT-9)
Fraction for residual binder R	0.9	0.9

[a]Data in parentheses are specified range.

Table 2. Application rates, 1972 field project.

	Section		
Item	A	B	C[a]
Design method	McLeod	McLeod	Maintenance practice
Aggregate type (Pennsylvania no.)	1NS	1B	1B
Binder type	RT-9	RT-9	RT-9
Rate of application			
Desired			
Binder (gal/yd²) at 60°F	0.32	0.23	0.30
Aggregate (lb/yd²)	34	20	23.4
Actual			
Binder (gal/yd²) at 60°F	0.33	0.25	0.26
Aggregate (lb/yd²)	32	21.4	23
Nomograph method			
Binder (gal/yd²) at 60°F	0.39	0.205	0.22
Aggregate (lb/yd²)	37	18	18
Performance evaluation			
Estimated loss of aggregate (%)	10	10	5
Bleeding	None to slight	Moderate	None to slight

[a] Average daily traffic on section C is 900 compared with 1610 on sections A and B.

grade asphalt cement, RC-800 cutback asphalt, and RS-2 emulsified asphalt), each with two aggregate types (1NS and 1B), resulted in six experimental sections. All sections were designed by using the California design method.

As the work continued, the weather turned progressively cooler. Problems were experienced in the first two sections, which used 250-300 penetration asphalt cement, because the damp aggregate would not adhere sufficiently to the straight-run asphalt cement in cool weather. When RS-2 emulsified asphalt was used, the maximum temperature recorded was 55°F in the day and the humidity was high. In north-central Pennsylvania, temperatures during the night approach freezing. This resulted in severe loss of cover aggregate in asphalt-cement and emulsified-asphalt sections. This problem was not encountered in RC-800 sections and therefore for design evaluation only the two RC-800 sections (D and E) will be considered here. The ambient temperature ranged from 50 to 65°F when RC-800 was used. Rolling procedure was similar to that for the 1972 project.

The test data on the materials and the design factors used are given in Table 3. The design (desired) application rates and the actual application rates of the binder and cover aggregates are given in Table 4. The rates of application based on the nomograph method developed subsequently are also included for comparison and will be discussed later.

Table 3. Design data, 1973 field project.

Item	Aggregate	
	1NS	1B
Aggregate type	Gravel	Gravel
Gradation (% passing)		
1 in	100 (100)[a]	-
3/4 in	100	-
1/2 in	99 (90-100)	100 (100)
3/8 in	74	82 (75-100)
No. 4	4	17 (10-30)
No. 8	0.6	2.5 (0-10)
Loose unit weight (lb/ft^3)	96.9	96.7
Flakiness index (%)	15.0	18.8
Effective maximum size (in)	0.450	0.433
Median size (in)	0.303	0.275
Whip-off loss (%)	0	0
Existing surface condition	Old, dry, porous	Old, dry, porous
K_c	1.4	1.4
Bituminous material type	RC-800	RC-800

[a]Data in parentheses are specified range.

Table 4. Application rates, 1973 field project.

Item	Section	
	D	E
Aggregate type (Pennsylvania no.)	1NS	1B
Binder type	RC-800	RC-800
Rate of application		
California design method		
Binder (gal/yd^2)	0.32	0.31
Aggregate (lb/yd^2)	23.5	22
Actual		
Binder (gal/yd^2)	0.37	0.35
Aggregate (lb/yd^2)	24.8	23.2
Nomograph method		
Binder (gal/yd^2) at 60°F	0.37	0.34
Aggregate (lb/yd^2) at 0 percent whip-off	23	21
Performance evaluation		
Estimated loss of aggregate (%)	10	10
Bleeding	None	None
Surface texture	Excellent	Good

The performance of these two sections was evaluated and is also given in Table 4. No bleeding was observed on these sections. The estimated loss of cover aggregate was 10 percent. The surface texture ranged from good (1B aggregate) to excellent (1NS aggregate). Both aggregates had a low flakiness index.

LABORATORY EXPERIMENTS

It was necessary to compare complex and simple test properties of the graded cover aggregate so that the latter could reasonably be used for designing surface treatments on low-volume roads.

Pennsylvania 1B aggregate (equivalent AASHTO or ASTM size 8), which is 3/8 in to No. 8 sieve size, was evaluated in this study. The gradation of 425 samples (stone, gravel, and slag) of this aggregate tested in 1974 was analyzed statistically as follows:

Sieve Size	Percent Passing: Specified Range	Mean	SD	Skewness
1/2 in	100	99.96	0.254	-10.477
3/8 in	70-100	91.03	5.077	-0.646
No. 4	10-30	17.79	6.529	+0.260
No. 8	0-10	2.63	1.905	+2.061

Seventy representative samples of aggregates of different mineralogical compositions were obtained from various quarries in Pennsylvania for further evaluation.

Aggregate Shape

Cubical particle shape is preferred for the cover aggregate, and thin particles are to be avoided. The flakiness index represents the percentage by weight of flat particles that have a least dimension smaller than 60 percent of the mean size of a sieve fraction. The lower the flakiness index, the more nearly the aggregate particles approximate cubical shape.

PennDOT does not specify the flakiness index of the aggregate. Like those of some other states, the specifications require that the thin, elongated pieces should not exceed 15 percent. The particle is defined as thin and elongated if the ratio between the maximum and minimum dimensions of a circumscribing rectangular prism exceeds 5:1.

The flakiness index and the percentage of thin, elongated particles were determined for the 70 samples and the correlation is shown in Figure 1. The correlation appears good. It is quite interesting to note that 15 percent thin and elongated corresponds to a flakiness index of 31.5. The National Association of Australian State Road Authorities specifies 35 as the maximum permissible flakiness index for surface treatment (3). Therefore, an aggregate of tolerable shape can be obtained for low-volume roads if the specification requirements on thin, elongated particles are enforced.

Median Size Versus Spread Modulus

The Lovering and the Asphalt Institute methods use the spread modulus, which is computed, as described earlier, from three mean sizes. The median size is that theoretical sieve size in inches through which 50 percent of the aggregate will pass. It is easily read from the gradation chart and no computation is required.

These two parameters were determined for the 70 samples and plotted in Figure 2. The correlation is very good and indicates that the median size can reasonably be used in lieu of spread modulus for the aggregate type tested in this study.

Median Size Versus Aggregate Spread Rate

Out of 70 aggregate samples, 40 samples had a flakiness index of less than 35 percent. These samples were selected for further experiments. Asphalt cement at the rate of 0.20 gal/yd² was applied at 325°F to aluminum pans (10.5x6.5 in), immediately followed by the aggregate spread. A small hand roller was used to embed the aggregate in the hot asphalt. The experimental quantity of aggregate used for each source was determined prior to this operation by spreading the aggregate one layer deep in the pan without asphalt and then adding 20 percent to the application rate. The pans were cooled over-

Figure 1. Flakiness index versus percentage of thin and elongated particles.

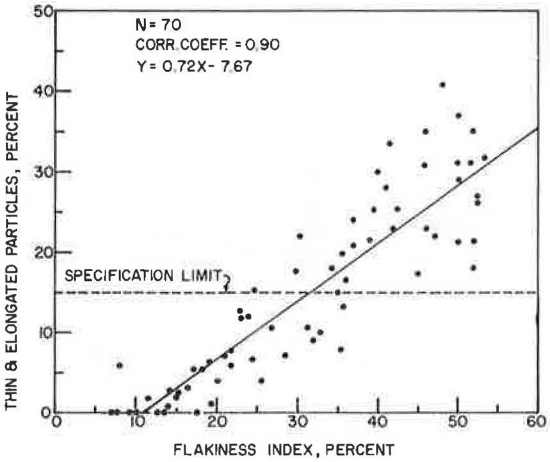


Figure 2. Median size versus spread modulus.

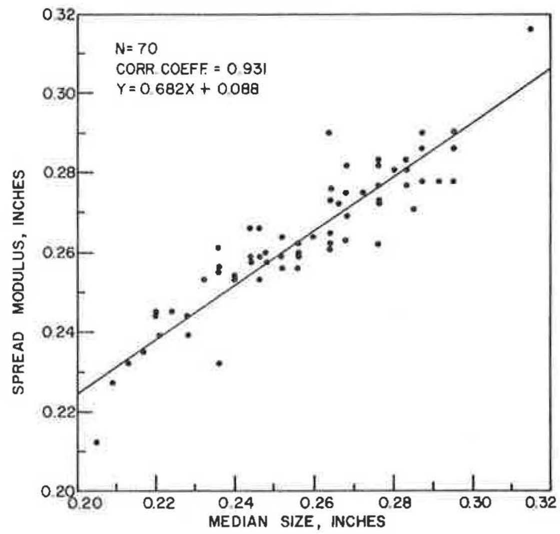
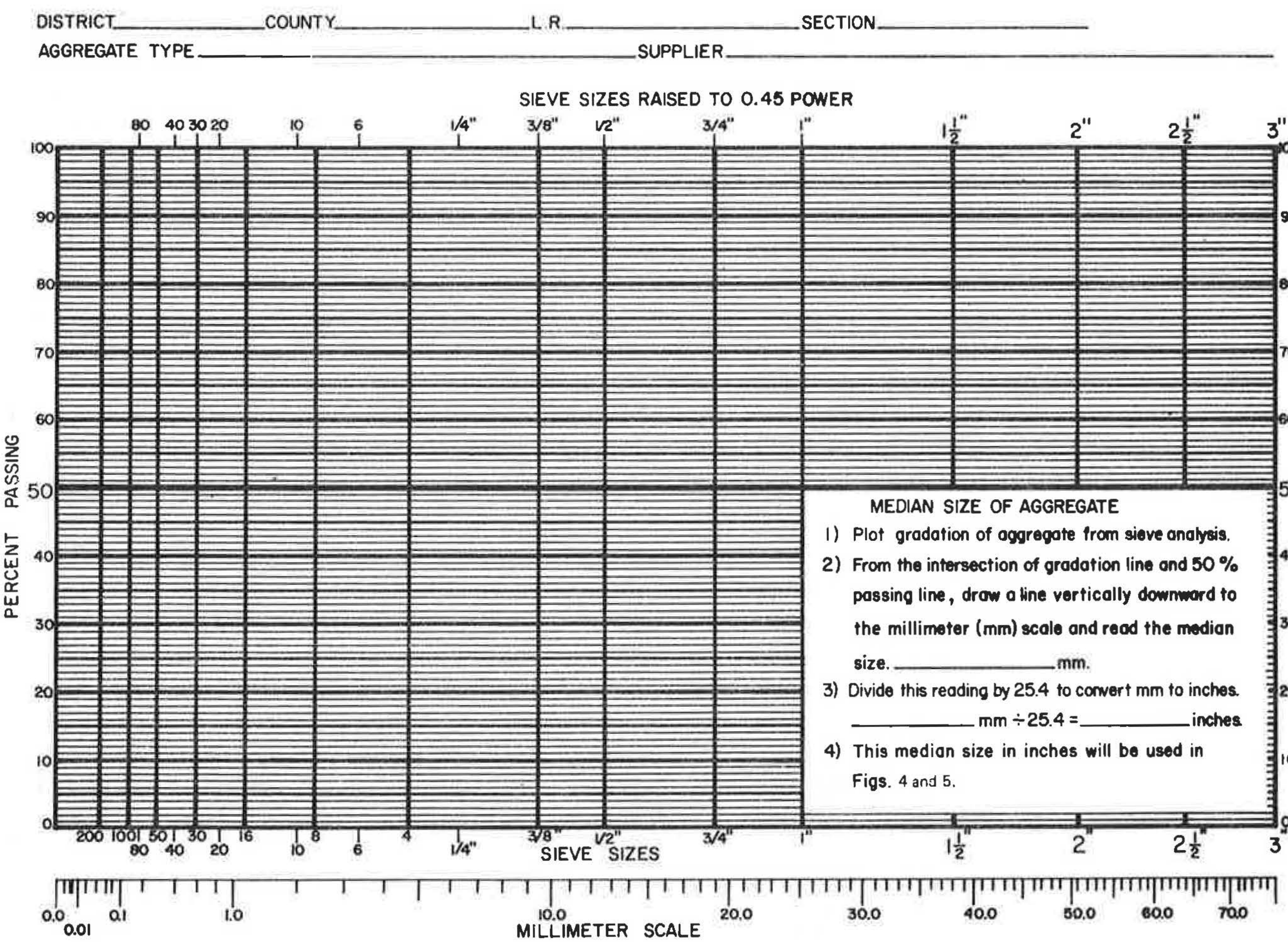


Figure 3. Gradation chart.



night. A stiff brush was then used to remove the loose aggregate particles and the aggregate spread rate in cubic feet per square yard was determined.

A fair correlation (correlation coefficient of 0.76) was obtained between the median size (in inches) and the aggregate spread rate (in cubic feet per square yard):

$$\text{Spread rate} = 0.53\ (\text{median size}) + 0.02 \qquad (9)$$

Since the action of aggregate spreader, roller, and traffic in the field cannot be simulated closely in the laboratory, the correlation is fair but appears reasonable for use on low-volume roads.

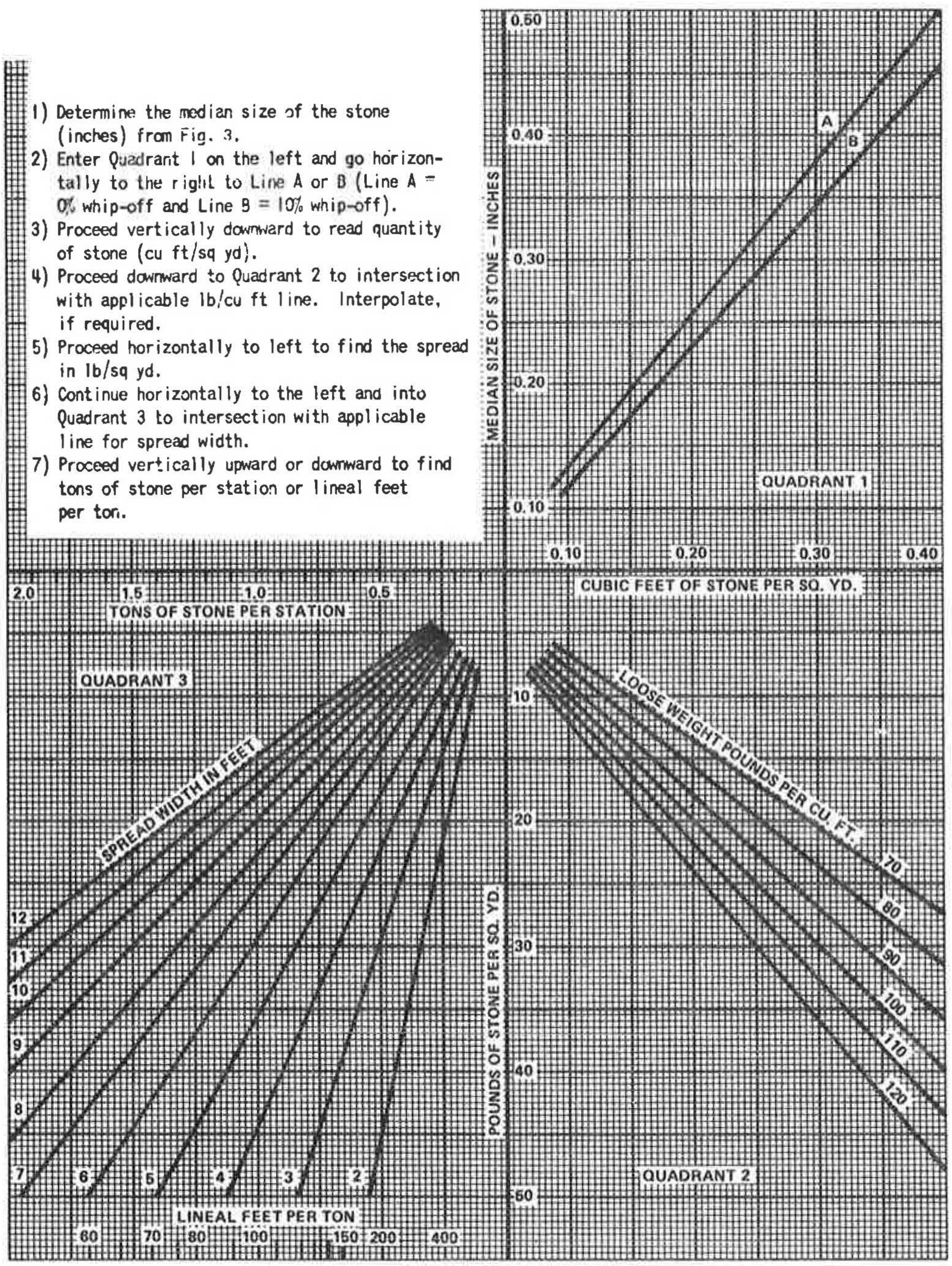
Median Size Versus Effective Maximum Size

As mentioned earlier, the effective maximum size (90 percent passing size) of the aggregate is used in the California method (5). The correlation between the median size and effective maximum size was attempted for 70 samples. The correlation coefficient of 0.65 was not considered satisfactory. It appears that the effective maximum size is not suitable for graded cover aggregates. This probably led Lovering to develop the spread modulus (6) in lieu of this parameter.

DEVELOPMENT OF NOMOGRAPH METHOD

The simplified nomograph design method for low-volume roads was developed in 1975 based on (a) synthesis of the five design procedures described earlier, (b) data from the laboratory experiments, and (c) verification on the 1972-1973 field experiments. It should be reiterated that for high-volume

Figure 4. Quantity of stone required.



roads and/or exceptional circumstances (such as flaky aggregate), additional data and a more rational design approach (such as that of McLeod) are recommended.

The following data are needed to use this design method:

1. ADT;
2. Condition of existing surface (five categories);
3. Type of bitumen to be used (asphalt cement, tar, cutback, emulsion);
4. Type of aggregate (limestone, gravel, slag);
5. Loose unit weight of cover aggregate in pounds per cubic foot; and
6. Gradation of cover aggregate.

The method consists of four figures, discussed below.

Figure 3

Figure 3 is a gradation chart to determine the median size (D) in inches at the 50 percent passing level as explained in the figure.

Figure 4

Figure 4 is used to determine the aggregate application rate in pounds per square yard and is based on the following equation:

$$S = 0.8DW \tag{10}$$

It should be noted that the median size has been substituted for spread modulus (M) based on the data from laboratory experiments and the American Bitumul design method. Testing and computation of the ALD

Figure 5. Quantity of bitumen required.

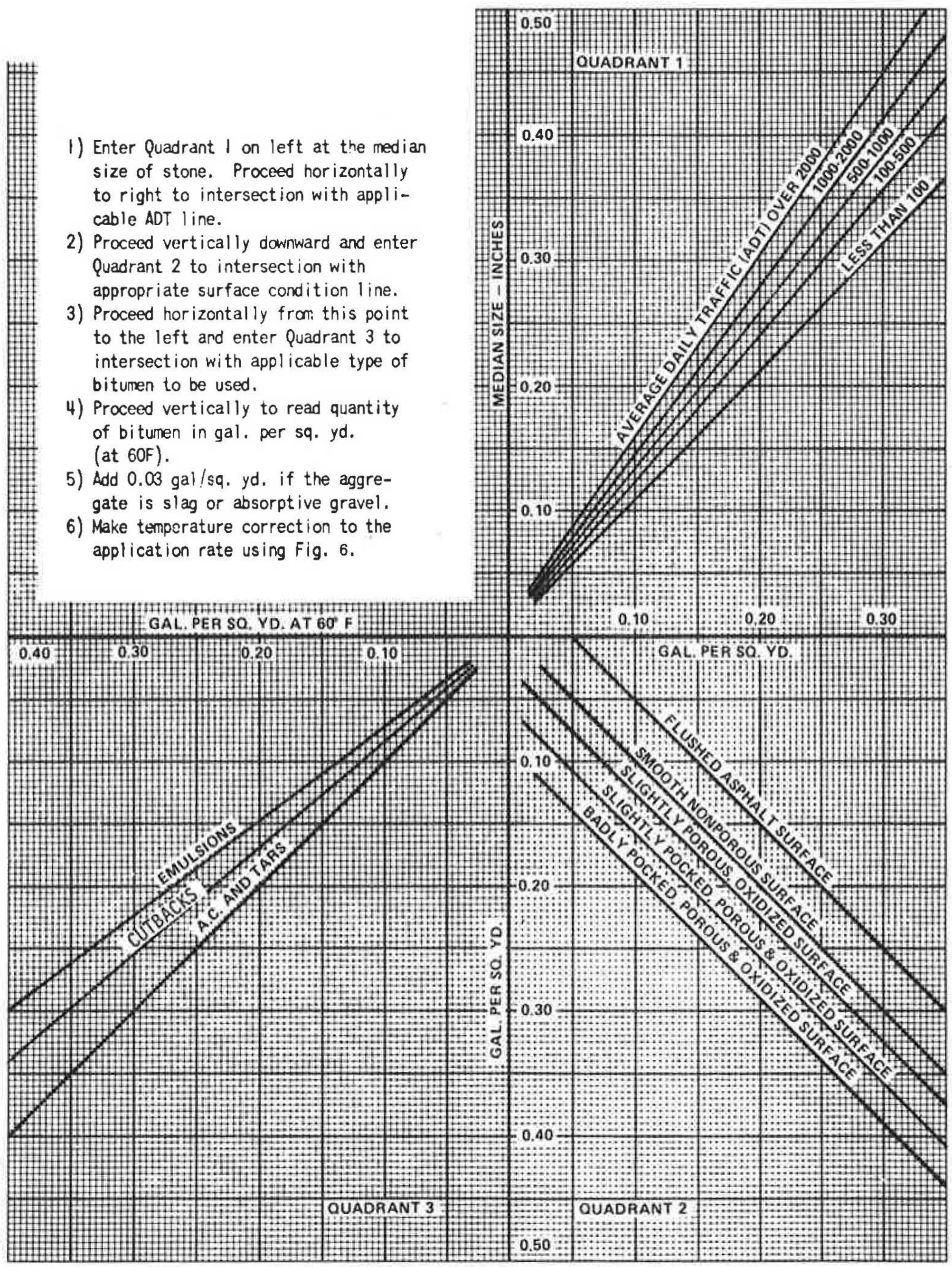
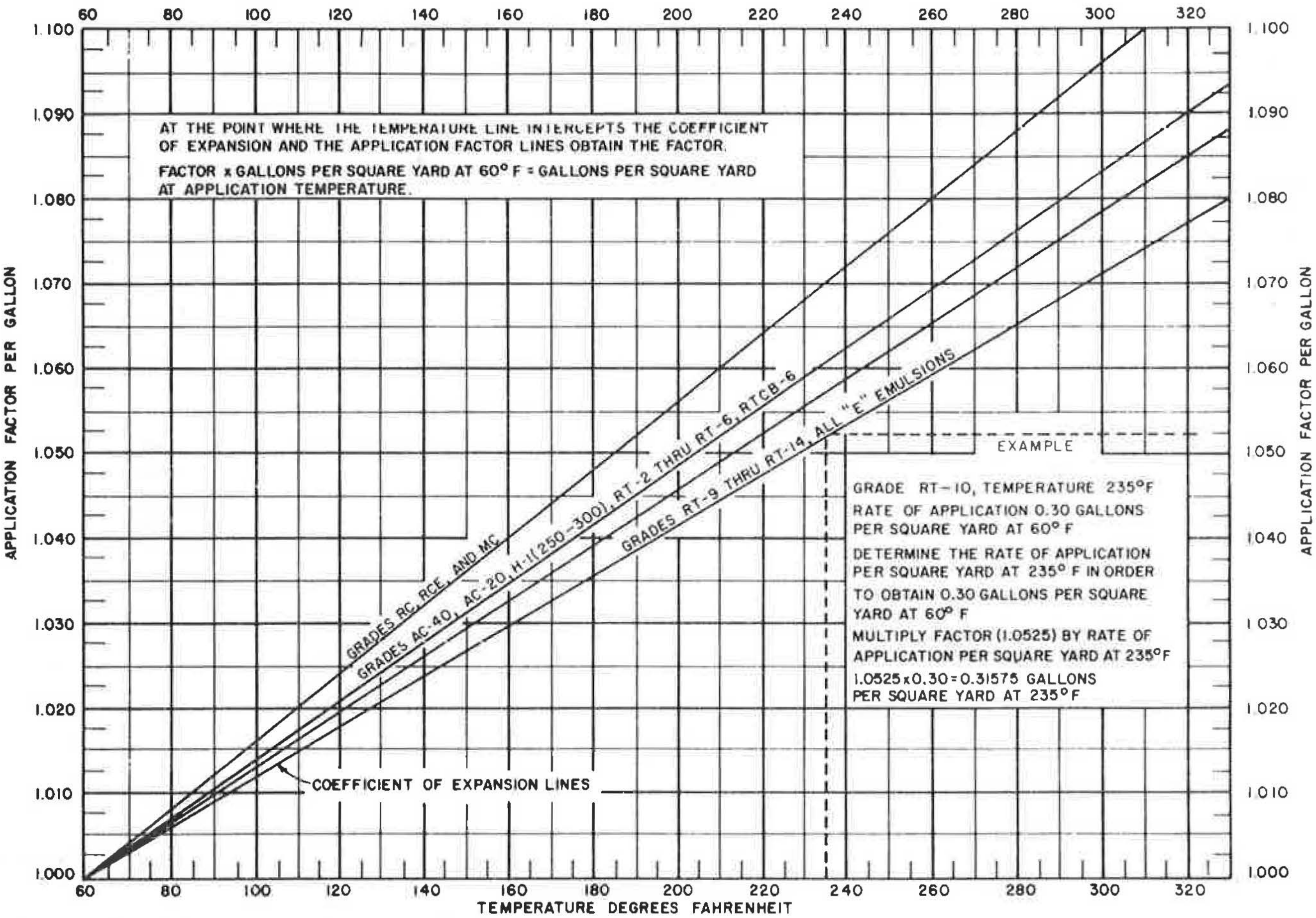


Figure 6. Temperature gallonage conversion chart for bituminous surface treatment applications.



of the aggregate from the flakiness index were considered too complex for use on low-volume roads. It is assumed here that the aggregate is reasonably cubical and the percentage of thin and elongated particles is less than 15.

Figure 5

The application rate of bituminous binder is determined from Figure 5. It is based on the following equation:

$$A = (1.122DT + V)/R \qquad (11)$$

where the traffic factor is based on ADT as follows (4):

ADT	Traffic Factor
<100	0.85
100-500	0.75
500-1000	0.70
1000-2000	0.65
>2000	0.60

V is a variable in gallons per square yard to cover absorption by the existing surface:

Pavement Condition	V
Flushed asphalt surface	-0.03
Smooth nonporous surface	0.00
Slightly porous, oxidized surface	0.03
Slightly pocked, porous, and oxidized surface	0.06
Badly pocked, porous, and oxidized surface	0.09

R is the fraction of effective residual asphalt:

Bitumen Type	R
Asphalt cement and road tar	1.00
Cutback asphalt	0.85
Emulsified asphalt	0.75

It should be noted again that the median size (D) has been substituted for spread modulus (M) as explained earlier. The traffic factor (T) is based on McLeod's method. The description of the variable V is based on the Asphalt Institute method, whereas the values are from McLeod.

The fraction (R) of asphalt residue for cutback asphalts has been assumed to be 0.85, which is slightly higher than the normal residual asphalt obtained by distillation to 680°F because it is believed that the cutbacks do not cure completely in their service life due to formation of skin at the surface. The fraction (R) for emulsified asphalt is a compromise between two schools of thought. Kari (9) believes that the emulsion should be applied at the same rate as that for penetration-grade asphalt. When the emulsion sets, there is a 30-35 percent volume reduction due to evaporation of water. The film collapses due to this volume change and forms a saddle; that is, it remains high on the stone and low in the spaces between the aggregate. This ensures a high surface contact area between the asphalt and the stone to prevent aggregate whip-off by traffic. Also, the amount of asphalt in the spaces between the stones is kept low to prevent bleeding. The Asphalt Institute design method (8) also takes this approach. However, McLeod and others recommend use of the actual asphalt residual fraction of the emulsion, which is usually in the neighborhood of 0.65. The nomograph method uses 0.75 as the value of the fraction (R) for emulsions.

Figure 6

Since the design application rate of the binder is obtained at 60°F from Figure 5 and the actual application temperature of the binder is higher, Figure 6 can be used to obtain the modified application rate at the desired application temperature.

Verification

Although the 1972 and 1973 field projects were designed by the McLeod and California methods, respectively, the rates of application were determined later by this nomograph method and are given in Tables 2 and 4 for comparison. Considering the performance (percent loss of cover aggregate and extent of bleeding) of these pavements and comparing the nomograph and actual application rates, it would appear that this method is reasonably suitable for low-volume roads. As expected, if the cover aggregate is flaky (Table 2, section A--aggregate 1NS has a flakiness index of 35), the nomograph method gives slightly higher application rates for binder and aggregate.

This simplified design method was used extensively for low-volume roads by the contractors and county maintenance forces in Pennsylvania during the 1980 and 1981 construction season with apparent success. It was also used with success in August 1980 on I-81 (four lanes) between the Ravine and Route 209 interchanges (2.5 miles) in Schuylkill County (10). This road carries an ADT of 11 000 vehicles (15 percent trucks), and the design speed is 55 mph.

SUMMARY AND CONCLUSIONS

Although many design methods have been developed on a rational basis in the past, most involve time-consuming or complex test procedures and/or computations. This has discouraged their use, especially for low-volume, low-cost roads. A need was felt to develop a simplified rational design method for this purpose. This was accomplished in four phases:

1. A literature review of the existing design procedures,
2. Use of two design methods on field projects,
3. Laboratory experiments to determine the relative significance of design parameters and correlations between complex and simple test properties, and
4. Development of a nomograph method and its verification in the field.

Surface treatment is still considered more an art than a science because many judgment factors are involved in the design and construction phases. However, it is believed that the suggested simplified method is suitable as a guide for low-volume, low-cost roads.

Additional research is being conducted in the Bituminous Laboratory of the Pennsylvania Department of Transportation to refine this method further while maintaining the simplified approach.

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Graded Gravel Seal (Otta Surfacing)

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A substantial number of the roads in Norway carry low traffic volumes. Most of these roads were unpaved 15-20 years ago. Today the gravel surface is partly replaced by low-cost surface dressings and premixes after only minor structural strengthening of the roads. The economic and technical requirements for the replacement surfacings are (a) the investment should be earned back in a few years through reduced maintenance costs and (b) the road user should find the riding quality comparable to that of ordinary hard-top surfacings. One of the most common and successful types of replacement surfacings, a gravel seal that has been named "Otta surfacing", is described. Otta surfacing is a 10- to 30-mm-thick bituminous surface dressing for which