IN SITU MATERIALS VARIABILITY

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The problem of material variability has plagued highway engineers since their first attempts to design a pavement structure. The heterogeneous composition of the materials required for the support of traffic loads has made it extremely difficult to develop rational theoretical design values. As a result, empirical formulas and tests have been devised and used in order that roads might be built with some semblance of order.

Since 1964 when the U. S. Bureau of Public Roads first emphasized the need for better definition of material characteristics, many states have conducted studies to evaluate variability. In situ measurements have been made and compared with design specifications. It has been recognized that highway materials are not "unique" and that they do follow statistical laws. The variabilities of materials, sampling, and testing are being isolated and analyzed. Initial steps, at least, are being taken by some states, other governmental agencies, and private consultants to make allowances for such variabilities. Specifications are being examined and in some cases changed because they do not fit the variability of the materials.

In this paper, only a limited amount of available information can be covered. The examples chosen were selected because they illustrate a problem and not because they were the best of such examples. They are intended to emphasize that designers do not deal with a uniform material.

It would seem that the designer should consider two major types of variabilities that can affect the performance of a pavement structure:

1. Variations between assumed design criteria and actual conditions during construction or during the life of the pavement; and
2. Normal variations in the materials used to construct the pavement structure.

The designer's task is to develop, by the most economical means, a highway structure that will survive in its environment to safely carry a stipulated amount of traffic. To accomplish his task he must have at present some method of estimating foundation strength; a knowledge of availability, strength, and durability characteristics of materials for constructing the structural section; and a knowledge of the performance of roads under similar environmental conditions. Under most design systems currently in use, each of his decisions is based on empirical data, semidocumented performance data, or personal experience. Part of the gap between the designer's assumptions and the final product will be discussed in this paper.

In the design of a pavement structure, use is generally made of a so-called soil profile. This consists of drilling holes and testing samples of removed material along the proposed highway alignment. From these tests and the position of the soil strata they represent, an estimate is made of the expected support value for the foundation soil that will be in place when the contractor completes his grading operation. Sometimes such estimates give a misleading indication of resulting support; but it is also surprising, when a close examination is made, how many foundation soils are actually within reasonable range of design values. Many illustrations could probably be developed along this line by using either the CBR test, the Texas triaxial test, or possibly the resilient modulus, but it will suffice to illustrate the comparison by using the California resistance (R) value.

It was estimated from soil survey data that basement soil along a 2-mile project that passes through low, rolling hills in San Benito County, California, would have a resistance value of 40 minimum. This was based on an average value of 55 and the assumption that, because of the low rolling terrain, there was only a small probability that all of the poorest materials would end up in a noncritical part of the fill. After the subgrade was completed, a series of tests showed the following:

Sample Source	R-value	Avg
Drilled holes in excavation areas	13, 47, 58, 65, 61, 68, 22, 62, 60, 36 67, 65, 47, 70, 48, 68, 68, 65	55
Top of foundation soil	56, 68, 69, 66, 65, 68, 68, 69, 75, 72 69, 61, 74, 73, 60, 35	65

The average resistance value of 65 is 25 points above the minimum design value but only 10 points above the average. However, if it is assumed that these data represent one population, the standard deviation, which is 9.4, would indicate that the variation in this material will be such that 95 percent of the material will be above 46 R-value or that roughly 96 percent will be above 40.

This example raises questions that have bothered some designers: Should all of the tests be above the design minimum? How much chance is there that additional testing will indicate more areas below minimum? What risk is there in accepting a few low values? Much of the designer's indecision is based on a lack of documented performance information as well as on a lack of statistical information to calculate the risks in accepting a few small areas of weak material. New methods of interpretation of in situ values of foundation materials are needed. Decisions to design or modify structural sections should not be made based on selective, individual tests but rather on as complete a statistical picture as is economical to obtain.

Now let us turn to the in situ variability of the resistance of materials to resilient deformation as measured by the Benkelman beam or other deflection devices. Again there are many examples in the literature that cannot be covered here. Figure 1 shows the effect of base layers on deflection values. On this project Benkleman beam deflections were obtained on the basement soil, at the top of the base, and on the top of the asphalt concrete. Attempts were made to obtain deflections on the subbase, but because of its sandy nature this was not practical or possible. However, the data do illustrate the variability of support that can be expected in the basement soil. Other such measurements have been made on other projects with sometimes a greater and sometimes a lesser degree of variability. The addition of a base material generally tends to lower the deflections but, as in this case, such a generalization is not always true. In any event, the asphalt concrete layer effect on deflection is quite notable and quite common in our measurements. This layer usually causes substantial reduction in deflection as

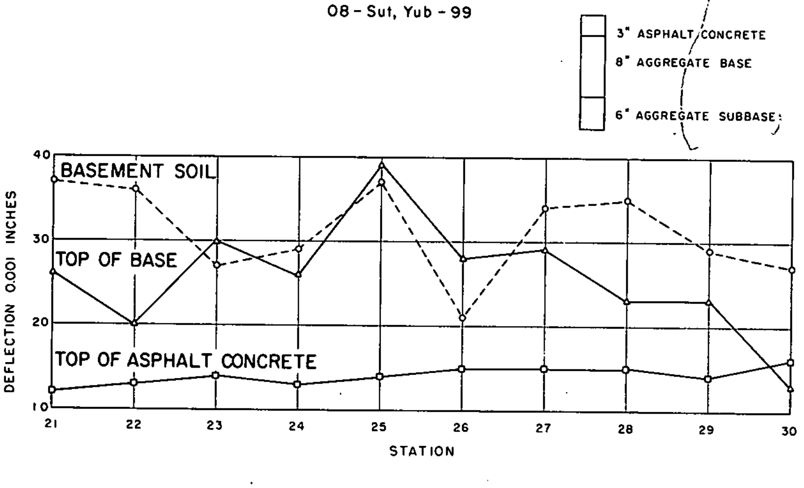

Figure 1. Effect of layers on basement soil layer.

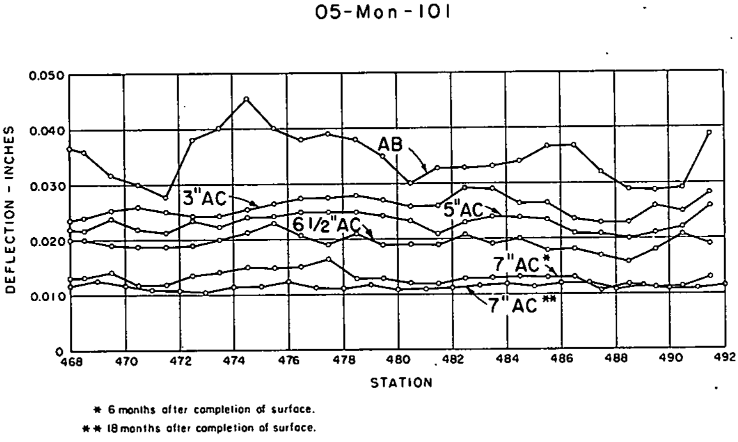

Figure 2. Effect of successive layers of asphalt concrete.

well as in the range of values. The effect of successive layers of asphalt concrete in achieving this uniform condition is shown in Figure 2.

Deflection measurements are usually fairly uniform when thicker layers of uncracked asphalt concrete are tested. As shown in Figure 3, this is not always true of older roads with thin asphalt-treated cover. In this situation the weakness of the 10-year-old double seal coat cover layer would appear to allow deflections to approach the variability of the supporting soil. A cushion course overlay consisting of 6 in. of aggregate base and 6½ in. of asphalt concrete placed on this highly resilient pavement in 1960 restored this road to a uniform tolerable deflection level and allowed for substantial increase in traffic. Deflections made in 1967 do not indicate any great change.

Although the designer assumes a certain uniformity of quality in the basement soil, he also assumes a uniformity of compaction that may or may not be present. Figure 4

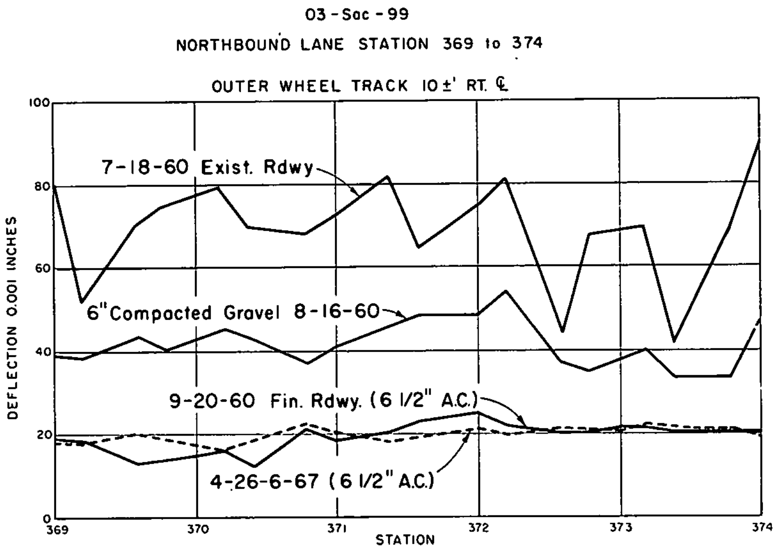

Figure 3. Effect of age on deflections.

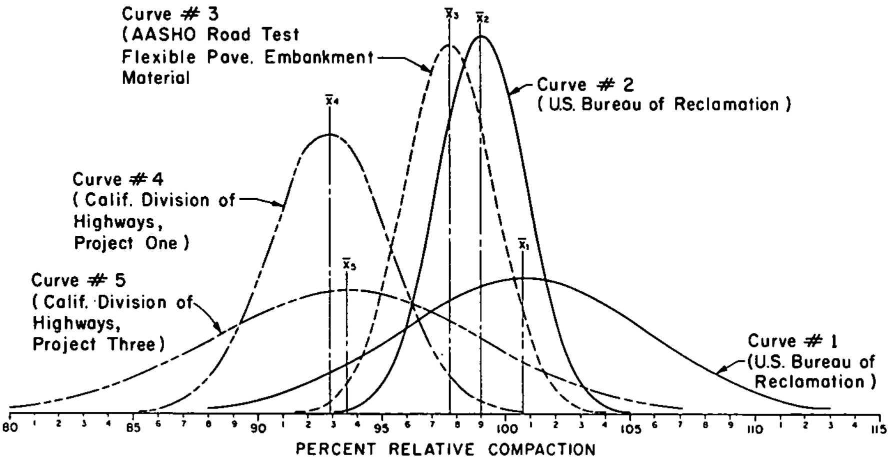

Figure 4. Comparison of uniformity of compaction.

shows the variability that has been encountered when compaction tests were made after the completion of a grade. As might be expected, the more uniform the material is, the narrower the range of values that are encountered on a particular project is. The more heterogeneous the material is, the greater the standard deviation and the spread of test results are. It is also noteworthy that the standard deviation bears no real relationship to the average value. The curves shown in the figure were obtained from data published by the U. S. Bureau of Reclamation, the AASHO Road Test, and the California Division of Highways (1). Resemblance of curves between two agencies such as the California Division of Highways and the Bureau of Reclamation indicates that these data are not uncommon and probably span the compaction variation of many subgrades. The specifications and compaction test methods used to obtain the data for these curves are given in Table 1.

There are many other variations between assumed design criteria and in situ conditions. Some of these are the variations between the material that the designer thought might be used and the material that the contractor actually used. This is particularly true of pit run material such as imported borrow or subbase. There are generally variations between the assumed moisture and density conditions that might develop in the road and those that actually occur. Such variations are affected not only by assumed and actual environmental conditions but also by improper compaction, poor drainage, or materials of inferior durability. The determination of moisture and density of test

TABLE 1

DATA FOR COMPACTION CURVES SHOWN IN FIGURE 4

Agency	Material	Compaction Test Method	Average Compaction	Standard Deviation	Approximate Percent Less Than Minimum Specified Limit
U.S. Bureau of Reclamation	Heterogeneous	Proctor E-11	100.7	5.0	29.5[a]
U.S. Bureau of Reclamation	Uniform	Proctor E-11	99.0	1.8	28.9[a]
AASHO Road Test	Flexible pavement embankment	AASHO T-99	97.7	1.9	7.8[b]
California Division of Highways	Uniform	Calif. 216	92.9	2.4	11.3[c]
California Division of Highways	Heterogeneous	Calif. 216	93.6	5.5	25.6[c]

[a]98 percent minimum relative compaction limit.
[b]95 percent minimum relative compaction limit.
[c]90 percent minimum relative compaction limit.

specimens to obtain support values for design is one of the difficult areas yet to be investigated to a positive conclusion. Currently there is a strong leaning toward the use of soil suction values as developed by the Great Britain Road Research Laboratory, but this system remains to be proved as a positive tool for the operating highway soils laboratory. There are also variations in the prediction of weight and amount of traffic as well as many other factors that the designer must assume at the time of designing the road. It is not the purpose of this paper to explore in detail all such variations but rather to emphasize those relating to construction. Nevertheless, it is necessary that such variations be considered in any evaluation of the effectiveness of a structural design system.

The contractor who builds a highway project has the obligation of furnishing and placing materials that comply with the specifications and plans. In so doing, he is required to place the materials in the structural section to certain specified thicknesses. Thickness would appear to be a noncontroversial, easy to obtain, and easy to measure specification. Yet, any inspector will confirm that this is not true. Materials paid for by the "square yard in place" tend to encourage keeping the thickness to a minimum. Materials paid for by the ton deposited on the grade have a reverse effect. On projects involving federal funds, layer thicknesses must be verified by cutting cores and digging holes in the completed structure. Table 2 gives summaries of thicknesses measured for various layers in the structural section in California from 1962 through 1969.

The ability of the contractor to accurately lay and place a layer is dependent somewhat on the accuracy required in placing the layer below. Therefore, the data given in Table 3 reveal an increasing degree of accuracy in the layer thicknesses from subbase through base to surface. This is fortunate because the lower materials are generally the cheaper materials. The figures on the asphalt concrete represent measurements that include surface, leveling, and base courses as one measurement. This includes all projects having between 0.2- and 0.6-in. total thickness. Although thicker

TABLE 2

THICKNESS MEASUREMENT VARIATIONS

Year	Material	Mean Deviation From Planned Thickness (ft)	Standard Deviation	Number of Measurements
1962	Asphalt concrete	+0.02	0.03	823
	Cement-treated base	+0.02	0.06	934
	Aggregate base	0.00	0.07	1,149
	Aggregate subbase	0.00	0.08	1,037
1963	Asphalt concrete	+0.01	0.03	1,327
	Cement-treated base	+0.02	0.06	1,173
	Aggregate base	0.00	0.06	1,310
	Aggregate subbase	0.00	0.09	1,183
1964-1965	Asphalt concrete	+0.02	0.03	1,760
	Cement-treated base	+0.02	0.05	2,187
	Aggregate base	0.00	0.06	1,285
	Aggregate subbase	+0.02	0.10	1,922
1966	Asphalt concrete	+0.02	0.04	1,569
	Cement-treated base	0.00	0.06	1,569
	Aggregate base	0.00	0.07	1,272
	Aggregate subbase	+0.03	0.12	1,833
1967	Asphalt concrete	+0.01	0.03	1,838
	Cement-treated base	0.00	0.06	1,412
	Aggregate base	+0.01	0.07	1,134
	Aggregate subbase	+0.03	0.11	1,887
1968	Asphalt concrete	+0.02	0.04	1,135
	Cement-treated base	+0.01	0.05	1,156
	Aggregate base	+0.01	0.06	828
	Aggregate subbase	+0.01	0.10	1,526
1969	Asphalt concrete	+0.02	0.04	1,323
	Cement-treated base	+0.01	0.06	1,318
	Aggregate base	+0.02	0.07	1,075
	Aggregate subbase	+0.02	0.11	1,370

TABLE 3

VARIANCE DATA FOR BASE AND SUBBASE MATERIALS

Test	Arithmetic Mean	Material Variance	Sampling Variance	Testing and Splitting Variance	Overall Variance	Overall Standard Deviation
Base Materials[a]						
Project B-1						
R-value	81.9	0.081	0.160	1.480	1.721	1.31
Sand equivalent	42.9	10.685	0.875	4.225	15.785	3.97
Percent passing No. 4	50.9	9.246	0.270	0.335	9.851	3.14
Percent passing No. 30	23.8	4.478	0.235	1.525	6.238	2.50
Percent passing No. 200	6.0	0.231	0.035	0.180	0.446	0.67
Project B-2						
R-value	79.9	1.133	0.0	4.695	5.828	2.42
Sand equivalent	30.6	35.171	0.525	1.325	37.021	6.08
Percent passing No. 4	58.1	5.603	0.700	1.710	8.013	2.83
Percent passing No. 30	27.3	4.402	0.400	0.580	5.382	2.32
Percent passing No. 200	7.9	0.952	0.075	0.215	1.242	1.11
Project B-3						
R-value	79.7	0.242	0.210	1.770	2.222	1.49
Sand equivalent	59.2	11.121	0.0	4.670	15.791	3.97
Percent passing No. 4	52.7	21.382	6.885	3.970	32.237	5.68
Percent passing No. 30	23.4	5.178	1.595	1.720	8.493	2.91
Percent passing No. 200	4.6	0.353	0.0	0.540	0.893	0.94
Subbase Materials[a]						
Project S-1						
R-value	68.8	14.612	0.0	25.855	40.467	6.36
Sand equivalent	30.2	3.502	0.0	12.835	16.337	4.04
Percent passing No. 4	49.5	14.378	0.265	3.685	18.328	4.28
Percent passing No. 200	7.8	0.474	0.190	1.100	1.764	1.33
Project S-2						
R-value	77.2	4.456	0.059	5.277	9.792	3.13
Sand equivalent	36.2	60.623	2.356	9.362	72.341	8.50
Percent passing No. 4	72.6	36.737	0.048	5.910	42.695	6.53
Percent passing No. 200	10.0	2.448	0.0	0.755	3.203	1.79
Project S-3						
R-value	70.9	54.038	0.0	25.250	79.288	8.9
Sand equivalent	29.2	5.519	0.0	1.915	7.434	2.73
Percent passing No. 4	45.0	34.253	3.275	5.990	43.518	6.60
Percent passing No. 200	8.6	2.245	0.110	0.450	2.805	1.68

[a]Slight negative variances were equated to zero.

layers have been placed on relatively few full-depth projects, a significant number have not yet been cored and measured. However, because the tolerance for making subgrade on foundation materials has not changed, it is anticipated that the standard deviations of full-depth asphalt pavements placed directly on the subgrade will increase and may approach those now recorded for the base materials.

In general, it would appear that variations in thickness of the asphalt concrete are more significant from a load-carrying viewpoint in thin layers than they are or will be in the thicker layers.

The strength of the various layers in the structural section is affected by compaction; and, in general, the higher the relative compaction is, the greater the strength is. Because of the difficulty in time required to make normal density and relative compaction tests, many states have adopted prescription types of compaction operations for the granular base and cement-treated base layers. Whereas this has some contractual advantages, it does not always result in the best possible compaction because granular materials can vary in their resistance to compaction. Sand, for instance, is usually difficult to compact, and graded aggregates are very easy. There are also differences between crushed and uncrushed aggregates. Several states are exploring and some specifying the use of statistical parameters to determine relative compaction.

The ability of the base materials to prevent lateral deformation within themselves and within the supporting basement soil has been determined by different agencies by using such tests as the CBR, R-value, Texas triaxial, and others. All of these tests are more or less empirical. However, each has proved its usefulness over the years. The principal disadvantage of such tests lies in the time required to perform them. At the speed of today's construction, the contractor can be far down the road before he learns that the material he placed a week ago does not comply with the CBR or R-value test.

Other tests in more or less general use as criteria of quality are gradation, the Los Angeles rattler, plastic index, sand equivalent, compressive strength for cement-treated base, percentage of crushed material, and petrological examination or durability test that will prevent the use of materials that disintegrate under adverse environments.

Since 1964 when the Bureau of Public Roads first encouraged research in measuring the variability of materials, there have been many reports on this subject. A good general coverage can be found in the six reports published during 1969 (2, 3, 4, 5, 6, 7). An attempt was made in these studies to separate material, sampling, and testing variances. Although the method used led to some doubts about its precise validity, it produced the first such information available to the highway engineer. Table 3 gives the results obtained on three projects in California (8). Generally, the data show small but significant variance due to sampling and larger variances in the material and testing areas. For these projects subbase material was obtained unprocessed from pits, and the base material was a crushed product processed by the contractor. As expected, the material variance for the subbase is high. However, the disturbing factor is that the testing variance increased in proportion to the material variance. When comparing these results to the base tests, one would conclude that the variance in the R-value test, for example, is not a uniform variance but is dependent on the type of material being tested. Such does not seem to be the case with the sand equivalent test, inasmuch as the results for base materials show overall standard deviations in the same ranges as those found for subbase materials.

Not one of the projects studied was in 100 percent compliance with the specifications, and yet the construction seemed to be normal, the materials were normal, and all the processes used were good. This led to conclusions, confirmed by other job tests, that judgment factors were being applied by field forces. This, in effect, made acceptance dependent on the amount of experience, background, and judgment of the inspector in charge of a particular operation. The final result of these studies has been the revision of specifications to include the opportunity for reasonable variability based on historical data of completed projects.

Specification tolerances and controls for the asphalt concrete layer of the pavement structure are generally tighter than those for the base and subbase layers. Nevertheless, the materials and workmanship that go into this layer are also subject to variations of significant magnitude. Granley (5) very capably describes variations in asphalt concrete construction. Table 4 gives a summary of 22 projects that were studied under the Bureau of Public Roads research program and reported by Granley. The data are similar to those given in Table 3 for the base and subbase materials, although they are more representative because they cover a wider geographical area. The data given in the table again confirm that variance is not a constant and that the sampling and testing errors are of significance.

Data reported for these projects also showed variations in asphalt content with a standard deviation of 0.28 percent for 23 surface course projects. The testing variance represented 40 percent, sampling variance 10 percent, and material variance 50 percent of the total variance. On the six base or binder course projects, the standard deviation was 0.35. The testing variance was 43 percent, the sampling variance 23 percent, and the materials variance 34 percent of the total. These variances indicate a need for more accurate testing and sampling procedures if the variations of the material are to be accurately determined.

Assuming proper mix design and satisfactory, available aggregates, we are still lacking a third and very important ingredient in asphalt concrete that affects the durability of the layer. This, of course, is the compaction of the hot mixture. Many articles have been written about the compaction of asphalt concrete, and all seem to emphasize that the mixture must be spread and compacted at the proper temperature to achieve maximum

TABLE 4

AVERAGES OF AGGREGATE GRADATION DATA FROM EXTRACTION TESTS

Sieve Size	Average Standard Deviation of Percent Passing	Shift of Mean From Job Mix Target	Average Variance as Percent of Total			Computed Average Compliance of Job Mix Tolerances
			Testing	Sampling	Material	
Surface Mixes (22 Projects)						
3/4 or 1/2 in.	1.43	1.70	72	4	24	99
3/8 in.	2.49	1.73	29	31	40	93
No. 4	3.51	2.95	12	18	70	78
No. 8 or 10	2.81	2.45	10	15	75	77
No. 20 or 30	1.74	2.10	13	18	69	87
No. 40 or 50	1.37	1.72	18	15	67	87
No. 80 or 100	1.00	1.44	17	11	72	82
No. 200	0.94	1.43	21	14	65	74
Average	1.91	1.94	24	16	60	85
Base or Binder Mixes (6 Projects)						
3/4 or 1/2 in.	4.33	1.66	65	13	22	83
3/8 in.	4.93	5.88	55	30	15	60
No. 4	3.92	2.03	46	17	37	76
No. 8 or 10	2.53	1.81	19	13	68	50
No. 20 or 30	2.17	2.22	25	28	47	81
No. 40 or 50	1.67	1.63	23	31	46	84
No. 80 or 100	1.15	1.23	30	30	40	97
No. 200	0.88	1.02	21	14	65	74
Average	2.70	2.19	36	21	43	76

density. Kilpatrick and McQuate (9) concluded that break-down rolling, either steel or pneumatic, should be concluded before the pavement temperature drops below 220 F. Experience in California tends to confirm this conclusion. For adequate compaction, the rolling pattern and the number of rollers required are dependent on the air temperature at which the mixture is placed, the thickness of layer, and the production of the contractor. Generally speaking, the thicker the layer is, the longer the allowable time cycle between the beginning of compaction and the end of compaction is. Stated another way, the proper compaction is achieved more easily and more readily in thicker lifts than in thinner lifts simply because the cooling of the mass is greatly retarded in thicker lifts.

It is not possible in this paper to explore all of the variations present in asphalt concrete mixtures. References previously quoted give a more complete picture. The discussion would not be complete, however, without covering the variability of in situ test data from tests on surfaces that have been used for a period of several years.

An article recently published by Welborn (10) discusses the durability factors of a group of projects constructed from 1954 to 1956. Table 5, taken from Welborn's report, gives the test variability after 10 to 12 years of service for six pavements all constructed with the same asphalt. Although there is a large spread in standard deviation. Welborn relates these variations very effectively to the void content of the mixtures. These are factors, however, that might be present in many asphalt concrete layers. How is the designer to anticipate the hardening that takes place in varying degrees? How can theory take this into account? What factors of safety are needed to prevent premature distress of the road surface? All of these questions and many more are in need of answers if a design method is to be adequately evaluated.

It is obviously impossible in a short paper such as this to discuss in any great depth all of the variations and the many ramifications found in variance of materials used in the highway structure. At best, only a few examples could be documented and reported.

It is extremely encouraging that a considerable amount of research effort has been expended in the past few years in measuring the variances of materials and construction operations. There is still much to be done. The designer needs better information on the anticipated in situ characteristics of foundation soils. He needs to have some method of determining the overall effect of variations in thicknesses in the various layers enter-

TABLE 5

VARIABILITY OF TEST DATA FOR PROPERTIES OF PAVEMENT SAMPLES

Property	Project Number	Variability			
		Minimum	Maximum	Mean	Standard Deviation
Penetration at 77 F	17	22	50	31.5	9.6
	18	31	70	47.6	14.4
	26	18	24	21.3	2.3
	27	23	74	46.7	17.6
	40	29	39	35.0	4.3
	41	17	38	23.8	7.3
Viscosity at 140 F, kilopoises	17	3.9	36.7	19.0	12.7
	18	2.5	17.0	9.1	5.6
	26	35.5	60.0	46.8	7.9
	27	2.1	32.0	11.6	11.2
	40	7.0	12.7	9.4	2.4
	41	13.4	315.2	119.0	104.0
Air void content, percent	17	0.1	4.9	2.5	1.5
	18	1.0	2.9	1.8	0.8
	26	4.4	7.2	5.7	0.9
	27	1.0	5.3	2.3	1.5
	40	2.1	2.9	2.4	0.4
	41	2.7	12.1	6.9	3.0
Voids filled with asphalt, percent	17	75.4	99.5	86.8	7.5
	18	84.1	94.0	89.4	4.3
	26	64.3	75.7	70.0	3.7
	27	73.4	93.3	87.4	6.8
	40	82.5	87.5	85.6	2.2
	41	47.2	85.0	67.5	12.1

ing in the structural section. He needs to have some assurance that construction methods are adequate and will result in a product equal to or better than the design requirements. He needs methods for estimating the detrimental effects of environment, which lead to changed properties of materials. Finally, he needs better methods of evaluating the product he designs because performance is the final measure of a good design.

REFERENCES

1. Sherman, G. B., Watkins, R. O., and Prysock, R. A Statistical Analysis of Embankment Compaction. California Div. of Highways, May 1966.
2. McMahon, T. F., and Halstead, W. J. Quality Assurance in Highway Construction: Part 1—Introduction and Concepts. Public Roads, Vol. 35, No. 6, Feb. 1969, pp. 129-134.
3. McMahon, T. F. Quality Assurance in Highway Construction: Part 2—Quality Assurance of Embankments and Base Courses. Public Roads, Vol. 35, No. 7, April 1969, pp. 166-176.
4. Baker, W. M., and McMahon, T. F. Quality Assurance in Highway Construction: Part 3—Quality Assurance of Portland Cement Concrete. Public Roads, Vol. 35, No. 8, June 1969, pp. 184-189.
5. Granley, E. C. Quality Assurance in Highway Construction: Part 4—Variation of Bituminous Construction. Public Roads, Vol. 35, No. 9, Aug. 1969, pp. 201-211.
6. Kelley, J. A. Quality Assurance in Highway Construction: Part 5—Summary of Research for Quality Assurance of Aggregate. Public Roads, Vol. 35, No. 10, Oct. 1969, pp. 230-237.
7. Granley, E. C. Quality Assurance in Highway Construction: Part 6—Control Charts. Public Roads, Vol. 35, No. 11, Dec. 1969, pp. 257-260.
8. Sherman, G. B., and Watkins, R. O. A Statistical Analysis of Untreated Base and Subbase Materials. California Div. of Highways, Res. Rept. M&R 631133-6, March 1967.
9. Kilpatrick, M. J., and McQuate, R. C. Bituminous Pavement Construction. U.S. Dept. of Transportation, Federal Highway Administration, Bureau of Public Roads, June 1967, 42 pp.
10. Welborn, J. Y. Asphalt Hardening—Fact and Fallacy. Feb. 1970.