Flexible Pavement Research in Virginia

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This paper reviews developments in flexible pavement design in Virginia since the report by Woodson at the 1954 meeting of the Highway Research Board. The trend away from macadam toward dense-graded aggregate and finally to "black base" in flexible base construction is traced. Several notable failures which led to the abandonment of selective grading and the substitution of select borrow foundation courses are described. Mention is made of experimental construction involving cement- and lime-treated subgrades; more recently this feature has been incorporated into a number of projects where suitable select borrow has not been available locally.

Because recent trends have in most cases tended toward drastic increases in construction costs, Virginia's Highway Research Council has persistently sought to develop means of minimizing these increases. The construction of experimental projects as part of the regular construction program in order to compare the performance of a variety of pavement designs under the same traffic conditions has been proposed, and one such project has been built. The principal purpose of this paper is to describe the program of experimental construction, the methods of evaluation used, and the application of these methods to the first experimental project.

●WOODSON (1) described in 1954 Virgina's newly adopted modified CBR method of designing flexible pavements. A year later Maner (2), who had succeeded Woodson as Assistant Engineer of Tests, carried the description a bit further in a prepared discussion of a paper by Hveem. The method is still used in essentially the same form (3), but only to provide a guide with regard to total pavement thickness required to prevent overstressing the subgrade. The composition of the pavement, the type and thickness of the various layers making up this total thickness, is not determined by test, although in theory the CBR method could be used to establish the greater portion of the design. For example, suppose that the basement soil for a given project is expected to have a CBR value of 6, from which the total structural thickness might be established as 19 in. This thickness could be made up of, first, a 6-in. layer of material with a minimum CBR value of 13, then a 5-in. layer with a minimum CBR value of 30, and, finally, 8 in. of some combination of high-type base and surfacing materials. The make-up of at least this top 8 in., and usually the entire 19 in. in actual practice, reflects the preference of the designer on the basis of his experience and judgment.

It is far from the purpose of this paper to question the judgment of the persons who make these decisions. Instead, its purpose is to describe certain research which has been undertaken to broaden their experience. But first, a brief review of the recent history of flexible pavement performance in Virginia and of the evolution of present design practices seems in order.

Ten years ago, most heavy-duty flexible pavement designs incorporated water-bound macadam as the base course, with bituminous treatments of the mixed in place or penetration types usually not totaling more than 3 in. in thickness. But this type of construction is slow and requires skilled workmen, and macadam seldom appears now in Virginia's bidding proposals.

Envisioned as the successor to macadam, the pug-mill mixed dense-graded aggregate base was introduced to Virginia in 1953. Three of the major aggregate producers promptly equipped themselves to made this material, and a number of pavement designs for projects in the vicinity of these plants included graded aggregate as the base course.

Figure 1 represents one such design for a project advertised for bids in 1955 and built the following year to carry traffic which at the latest published count included 875 trailer trucks and buses in a total daily volume of 4,294 vehicles.

The 12-in. graded aggregate base here had an asphaltic concrete surface approximately 2½ in. thick. The balance of the typical section consisted of select borrow whose only requirement was that it have a minimum laboratory soaked CBR value of 12. The practice of attempting to obtain this select material from within the limits of regular excavation by the method known as selective grading has been largely discontinued. The difficulty seemed to lie in distinguishing the select soils from those not so select, and more recently all types of select material have been obtained from sources outside the project.

Trench-type construction also has practically disappeared on projects of major importance, and some type of select material is used from ditch to ditch. For this reason, the system of computing pavement construction costs in terms of the square yard has been abandoned in favor of one which considers the cost per lineal foot. In this manner the cost of all materials above the top of the earthwork is included. In the case illustrated, the cost per lineal foot was $9.16.

Unfortunately, Virginia's luck with dense-graded aggregate pavements was not as good as that of North Carolina, from whom the idea for this type of construction was borrowed. Of the first 17 major projects built, all have developed at least minor distress, ten have cracked badly, and two required either partial or complete resurfacing within six months after completion. All told, seven projects required some resurfacing in less than three years.

Figure 2 shows the condition of one such pavement only a month after it was opened to traffic. This is the same pavement whose design was described in Figure 1. Patching of this badly cracked surface along with a complete plant mix resurfacing applied the first summer, added $2.12 per lineal foot to the investment here before the pavement was eight months old. An investigation by the Virginia Council of Highway Investigation and Research revealed that deflections on this pavement under an 18,000-lb axle load averaged 0.069 in., as measured with the Benkelman beam. Specific causes for these high deflections could not be determined.

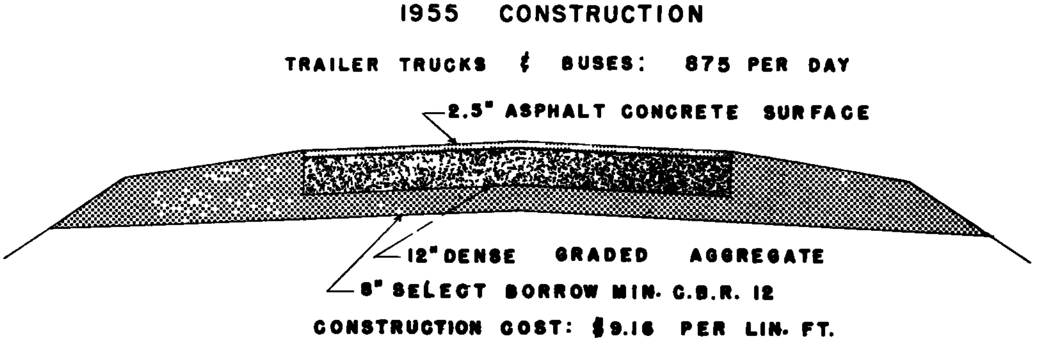


Figure 1. Typical pavement section for 1955 construction.

Figure 2. Cracking of pavement described in Figure 1.

It was plain that something had to be done to prevent repetitions of this failure. Because there was no definite evidence that construction had not met specifications, it was decided that future designs should be made stouter.

Figure 3 shows the extent to which the design of a pavement to carry traffic similar to that carried by the previously mentioned one was "beefed up" only two years later. (Actually this project does not carry as many heavy vehicles as the first one.) The CBR requirement for select borrow on this project is 20 and the total thickness of asphaltic concrete is 9½ in. instead of the corresponding 12 CBR and 2½-in. asphaltic concrete thickness for the earlier project. These and other minor changes brought the total cost to $17.33 per lineal foot, a jump of 89 percent over the cost of the lighter design.

The greatest single factor influencing this cost rise was the adoption of asphaltic concrete as the base material. Black base, to use the colloquial term, was introduced in Virginia about 10 years ago in an area where it could be produced relatively cheaply from local pit-run aggregates, an area also where crushed stone was non-existent.

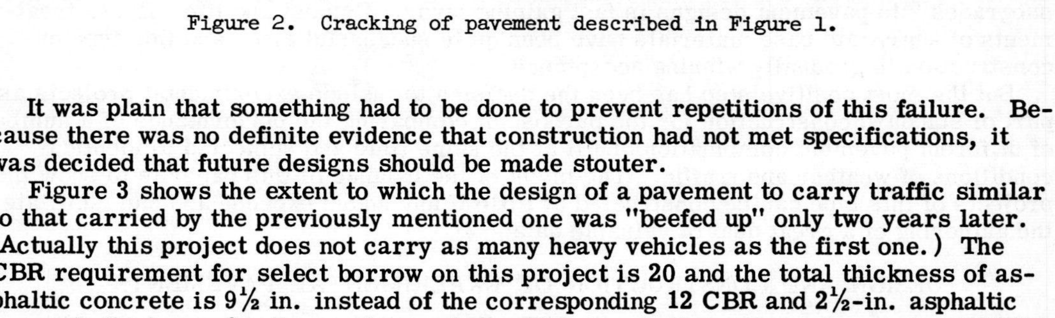


Figure 3. Typical pavement section for 1957 construction.

Its success was readily apparent, and soon black base found its way into designs for certain selected pavements in all parts of the state, though often at quite a considerable increase in cost.

The next most costly change was the requirement of increased CBR value along with certain gradation and plasticity limits for the select borrow. These requirements have largely ruled out roadside borrow pits, and on some still more recent projects have necessitated the use of commercially crushed aggregates. The unit cost of the CBR 20 select borrow in Figure 3 was almost four times that of the CBR 12 material in Figure 1.

The Virginia Council of Highway Investigation and Research when formed in 1948, had established as its first objective the carrying out of "research programs for the purpose of facilitating the economic design, construction, and maintenance of highways." In light of this stated objective, it became evident that special effort should be exerted to develop, through research, methods of minimizing such increases in construction costs as have just been described.

One of the first projects proposed toward this end involved studies of the benefits which might be realized through stabilization of subgrades and bases, as practiced in a number of western states. Soil-cement bases, with relatively high percentages of cement being added to soils occurring naturally in the roadway, had been built for many years in Virginia, but the idea of adding lower percentages of cement or even of hydrated lime to subgrade soils beneath heavy-duty pavements was relatively new. From the studies at the Research Council laboratories and from several successful experimental installations in the field (4), the incorporation of cement or lime treatments of subgrades into pavement designs is fast gaining favor. Cement and lime-flyash treatments of aggregate base materials have been quite successful also, and this type of construction is gradually winning acceptance.

But the most positive step has been the decision to include experimental projects as part of standard construction for the purpose of comparing the performance of a number of different pavement combinations built at the same time and subjected to identical conditions of weather and traffic. The intent of the balance of this paper is to show that projects of this type can be constructed at little if any added expense and can facilitate the gathering of a great deal of valuable data.

DESIGN AND CONSTRUCTION OF EXPERIMENTAL PAVEMENTS

Following the recommendation of T.E. Shelburne, Director of Research, the Virginia Department of Highways in 1957 created a four-man Research Advisory Subcommittee to assist the Research Council in outlining a program of experimental construction. At one of its earliest meetings, this subcommittee formulated certain basic principles to be considered in designing such experimental projects. These were:

1. That projects should be constructed to standard specifications with conventional equipment.
2. That the number of design variables to be incorporated into a given project should be held to a minimum (in the neighborhood of four).
3. That replication of the various designs should be provided within projects as well as from project to project.
4. That test projects should be built on roads carrying a considerable volume of heavy truck traffic.

Principles 1 and 2 were designed to minimize complexities in construction while 3 and 4 were to give some assurance that the findings of the experiment would be reliable and useful. These findings, it was hoped, would assist in interpreting certain of the findings from the WASHO and AASHO road test projects as they might apply to the conditions of traffic, climate, and soil existing in the area of this much smaller scale experiment.

Figure 4 shows the typical pavement section used in the first experimental project, located in the westbound lane of US 58 immediately west of the town of South Boston. In this section of Virginia, United States Weather Bureau normal temperatures range

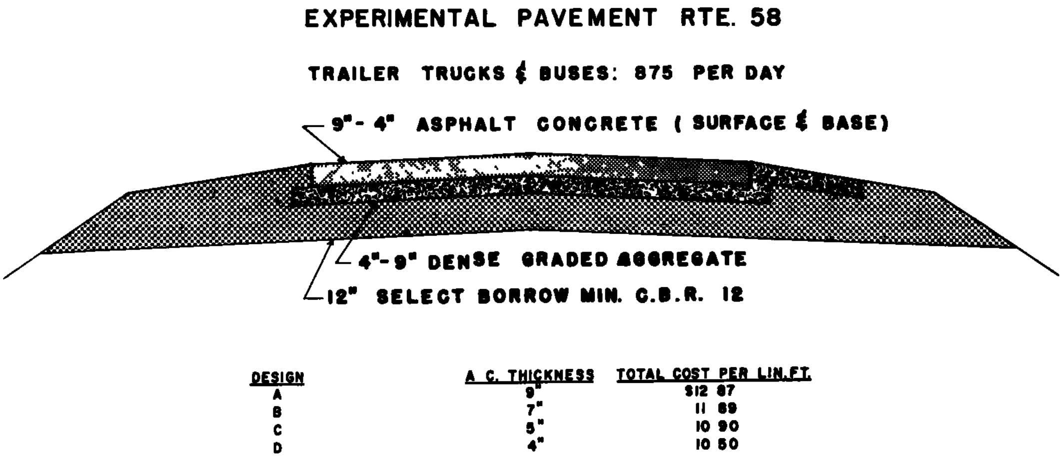


Figure 4. Typical pavement sections for experimental project on Route 58 west of South Boston.

from 80 F in July to 40 F in January; rainfall on the average totals approximately 44 in. per year and is rather uniformly distributed from month to month. These are long-term average figures, however, and extremes diverge markedly. In 1958, for example, monthly precipitation ranged from 1.66 in. in September to 6.48 in. in June, and extremes in temperature ranged from 98 deg to minus 2 deg. The variable investigated here was thickness of asphaltic concrete. The total structure above the native soil subgrade was 25 in. thick in each of four designs. The top 13 in. consisted of dense-graded aggregate, in thicknesses of 4, 6, 8, or 9 in., covered with asphaltic concrete in corresponding thicknesses of 9, 7, 5, or 4 in. Dense-graded aggregate was used also to surface the outer shoulder of this divided highway. The balance of the typical section was composed of select borrow, minimum specified CBR value of 12, obtained from a local pit near the east end of the project. (Although the borrow was only required to have a CBR value of 12, in actuality the CBR values on twelve samples tested averaged 33. The pit was in a deposit of disintegrated granite with a high percentage of coarse particles greater than 2 in. in size.)

The four design variables will be identified hereafter as Designs A, B, C, or D as shown in Figure 4. The costs per lineal foot for a single roadway for each of the four designs, based on contract unit prices, are given in Table 1.

TABLE 1

ROUTE 58 EXPERIMENT: SUMMARY OF CONSTRUCTION COST PER LINEAL FOOT FOR ONE ROADWAY (PAVEMENT AND SHOULDERS)[1]

Design A.C. Thickness	A 9 in.	B 7 in.	C 5 in.	D 4 in.
	($)	($)	($)	($)
Course:				
Asphaltic concrete	8.88	6.98	5.01	4.21
MC-O prime	0.27	0.27	0.27	0.27
Graded aggregate	2.05	2.98	3.96	4.36
Select borrow	1.24	1.23	1.23	1.23
Shoulder surface	0.43	0.43	0.43	0.43
Total	12.87	11.89	10.90	10.50

[1] Bid prices in usual units of tons, gallons, cubic yards, etc. By computation all prices converted to unit of one lineal foot.

The four guiding principles established for experimental construction were adhered to closely. Because standard specifications and conventional equipment were used and because the number of combinations was held to four, contract unit costs ran very close to the statewide averages for primary construction. The 4½-mi project was divided into eight subsections ranging from 1,830 to 3,550 ft in length, and replication was provided in that each design occurred twice. The ever increasing volume of trailer trucks and buses using this portion of US 58, totaling over 425 daily in each direction at the latest count, gives assurance that the heavy traffic requirement of principle 4 will be met.

Considerable detailed information on the classification, both engineering and agricultural, of the native soils, has been gathered by the Research Council. All available information regarding source, type, and quality of select borrow and other paving materials also has been catalogued for future reference. Such details have been generally omitted from this paper.

The test project was built in 1958 and opened to traffic in December of that year though not finally accepted from the contractor until January 15, 1959.

RESEARCH EVALUATION STUDIES

Realizing that conclusions based entirely on performance under traffic might not be available for some years, it was decided that certain observations and measurements should be made by the Research Council in an attempt to secure an earlier, though tentative, evaluation of the four pavement designs in this first experimental project.

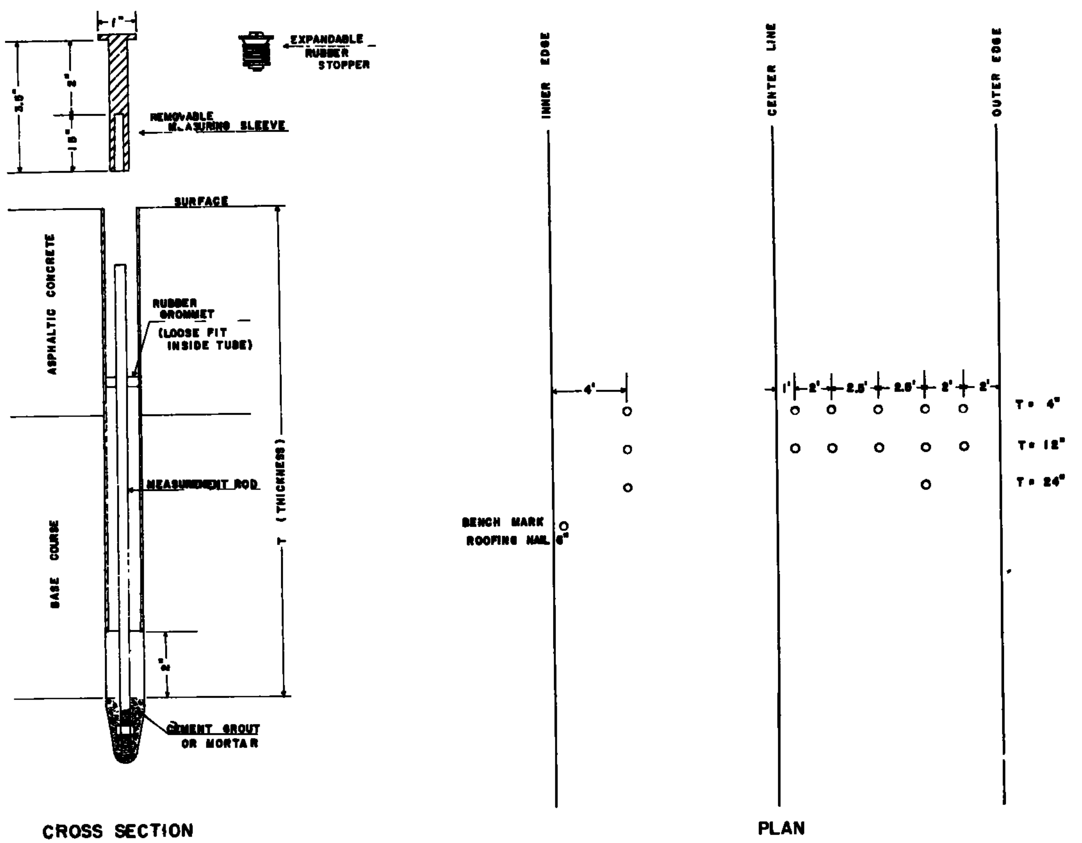
Council technicians were assigned to the test project during most of the paving operations to secure data on the compaction of the select borrow and of the dense-graded aggregate base course. Test sites were established in groups of five at regular intervals of 1,000 ft; in each group the individual sites were spaced 50 ft apart. At each site, with only two or three exceptions made necessary by the contractor's sequence of operations, measurements were made of the final construction density of the borrow and base courses just prior to the placing of the subsequent layer.

In place density values were determined by AASHO standard method T-147-54, except that test hole volumes were determined with a small water balloon volumeter. Research Council investigations have indicated that this instrument is superior in both accuracy and precision to the sandcone method in testing coarse-grained base materials. In place densities are expressed in terms of percent of laboratory standard density. Standard density is determined in Virginia by AASHO Standard T-99-57, method A, on the minus No. 4 material only. Correction for coarser aggregate assumes no voids associated with the plus No. 4 fraction and that minus No. 4 material remains at standard density. Research is under way aimed toward an improved method of establishing standard density in the case of aggregate materials.

Later, after the final surface had been laid, settlement rods were installed at 1,000-ft intervals to permit measurement of progressive changes in elevation of layers 4, 12, and 24 in. beneath the surface. The idea for these rods was borrowed from the WASHO Test Road (5). The rods were installed transversely across the pavement at one of the five sites in each test group (Fig. 5). Six are placed at a depth of 4 in., and two at 24 in. below the surface. The determination of the elevation of the top of each rod was facilitated by the use of a sliding scale graduated to the nearest 5 one-hundredths of an inch, attached to a standard level rod. As all sights were from less than 30 ft away, this scale could be read quite easily with a standard eye level. Since the rod could be adjusted to read zero at the bench mark each time, all changes in elevation became readily apparent by direct comparison with readings taken previously.

Additionally, soon after the pavement had been opened to traffic, extensive studies of pavement deflection were begun, measurements being made with the Benkelman beam and a test load of 18,000 lb on a single axle. The first series of tests was made in February and a second series in October 1959. Three separate techniques were used in these tests to obtain the following:

1. Maximum deflection at the surface, the conventional test with truck moving at



Installation: Hole drilled through asphaltic concrete pavement into base to exact outside diameter of protective metal tube. Hole continued to desired depth "T" by use of special driving rod. Proper quantity of cement mortar dropped to bottom of hole to cement rod to bottom. Protective tube driven into place and capped to prevent entry of water and foreign material. Measurements: Expandable rubber cap removed and special extension sleeve placed on top of rod. Level rod or Benkelman beam probe set in place on extension sleeve cap for determination of elevation or deflection of bottom of rod.

Figure 5. Settlement measuring rod installations, plan and cross-section.

creep speed past probe. Initial, maximum, and final dial readings recorded. Tests made at each site where base and borrow density had been determined, both in February and in October.

2. Longitudinal distribution of deflection. Truck stopped at 2-ft intervals both approaching and departing from the probe, dial reading recorded at each stop. Tests made in February at only one site in each group of five, in October at all sites.

3. Subsurface deflections. Test truck backed into position over probe, then pulled forward again. Initial, maximum, and final dial readings taken with probe on surface and on top of extensions of settlement rods at 4-, 12-, and 24-in. depths. Tests made at one site in each group of five in February only.

Table 2 summarizes the results of the density determinations made during construction and of the maximum surface deflection measurements made in both February and October. From this it may be seen that the magnitude of deflections determined

TABLE 2

ROUTE 58 EXPERIMENT RELATIONSHIP BETWEEN FINAL CONSTRUCTION DENSITY AND PAVEMENT DEFLECTION UNDER 18,000-LB SINGLE AXLE LOAD

Pavement Design	Average Density (% of Standard)			Average Surface Deflections Thousandths of an Inch			
	Select Borrow	Aggregate Base	Borrow-Base Combined	Outer Wheel Path		Inner Wheel Path	
				Feb.	Oct.	Feb.	Oct.
B(7 in. A.C.)	98.8	96.6	97.7	33	35	30	32
D(4 in. A.C.)	96.6	97.7	97.2	38	37	37	36
C(5 in. A.C.)	97.6	95.9	96.8	45	46	44	43
A(9 in. A.C.)	95.3	94.4	94.9	47	55	46	50

in the conventional manner bore no relationship to the thickness of asphaltic concrete, but rather that high deflections seemed to be more the result of low density in the base and borrow materials. In other words, the highest deflections, on the average, occurred in the sections built to Design A, which had the greatest thickness of asphaltic concrete, 9 in., but had the lowest average construction density in both the base course and the select borrow. The fact that the combined average of base and borrow densities falls in perfect inverse order to the average magnitude of deflections (240 measurements in each wheel path at each of two seasons of the year) probably is the most important finding from the experiment to date.

The tests showing the longitudinal distribution or rate of occurrence of deflection give some indication of the relative abilities of the four pavement designs to act as a slab and distribute loads from the surface to underlying layers. This technique, described as number 2, was designed to ascertain the abruptness of the deflection, it being felt that a given deflection occurring gradually as the load approaches the point of measurement should not be as destructive as a similar deflection occurring suddenly. Analysis of the actual values, however, was complicated by two uncontrollable variables, (a) the wide range in maximum deflections from site to site and from design to design, as given in Table 2, and (b) a similarly wide range in residual or more or less permanent deformations, recorded as the differences between the initial and final dial readings. But these variables become less troublesome when all values are expressed as percentages of a maximum value (Table 3).

Study of Table 3 reveals that the pavement with the greatest thickness of asphaltic concrete, Design A, does exhibit some slab action, because deflections occur more gradually. In general, deflections are less gradual as plant mix thickness decreases. In October, the figures for Designs B and C were not noticeably different; the reason for this will become more apparent from a study of the section on Early Performance Observations.

In both series of tests, the most abrupt deflections occurred in Design D. Therefore it is assumed that were it possible to construct these four pavements on identical subgrades and compact all courses to identical densities, the destructive effect of pavement deflection would be in inverse proportion to the thickness of asphaltic concrete.

Analysis of the deflection values measured at the various subsurface levels (technique 3) also is complicated by the same variables that hindered analysis of the "rate of occurrence" figures. Again the percentage approach seems more understandable, and has been adopted in Table 4.

Here it seems that in Design A the lowest percentage of the total deflection originates in the pavement structure itself. Attempts to rationalize this in light of other evidence from density and settlement measurements have not been entirely successful. Therefore no apparent significance is evident from these figures, but it may be of more than passing interest to note that from 40 to 54 percent of the total deflection seems to originate in the subgrade below the 24-in. level.

TABLE 3

ROUTE 58 EXPERIMENT DISTRIBUTION OF DEFLECTION AHEAD OF AND BEHIND LOADED AXLE

Pavement Design	Deflection at Indicated Distance Ahead of Load % of Maximum		Maximum Deflection	Deflection Remaining at Indicated Distance Behind Load % of Recovery		
	4 ft	2 ft	Load Over Probe	2 ft	4 ft	6 ft
(a) February 1959 Series						
A(9 in. A.C.)	15	37	100	56	18	5
B(7 in. A.C.)	10	25	100	43	11	6
C(5 in. A.C.)	9	23	100	38	7	2
D(4 in. A.C.)	8	18	100	28	6	2
(b) October 1959 Series						
A(9 in. A.C.)	18	43	100	37	9	3
B(7 in. A.C.)	11	27	100	31	6	2
C(5 in. A.C.)	12	28	100	27	6	2
D(4 in. A.C.)	9	22	100	24	4	1

TABLE 4

DEFLECTIONS BENEATH PAVEMENT SURFACE

Pavement Design	Percentage of Total Surface Deflection Measured Within Layers Indicated		
	Top 4 in.	Top 12 in.	Top 24 in.
A(9 in. A.C.)	5	18	46
B(7 in. A.C.)	5	26	57
C(5 in. A.C.)	6	26	60
D(4 in. A.C.)	5	25	58

EARLY PERFORMANCE OBSERVATIONS

The Research Council makes periodic surveys, most of which are of the "quickie" type in which only the more obvious defects are logged. An odometer which measures distance in feet is attached to an ordinary automobile so that the location of such defects can be recorded on especially prepared log sheets which may be used over and over again. In each survey of a given project, a different colored pencil is used for recording; in this manner the progressive growth of distress can be noted. Two such surveys have been made of the US 58 project.

More detailed observations made on the US 58 test project have included measurement of changes in elevation of the settlement rods (described earlier under Research Evaluation Studies) and measurement of surface rutting.

After one year under traffic the most pronounced defect which has occurred was a rather serious one near the west end of the project in one of the two sections built to Design C. Since construction of the base and surface was from west to east, this failure occurred within the first 1,500 ft completed. It also occurred between two of the groups of test sites where the highest individual deflection and the lowest individual construction densities on the project had been recorded.

The pavement in this area started to crack in April 1959 and by June it had been heavily patched—in some places to the full depth of the base. After heavy summer showers, the patching crews noted an accumulation of free water in the loosely bonded aggregate base. Later in the summer a stretch several hundred feet long was completely resurfaced with about 1½ in. of asphaltic concrete. This, it is felt, accounts for the change in behavior of Design C with regard to rate of occurrence of deflections, as was noted in the October deflection readings (Table 3).

Aside from this one serious failure in Design C, the only other distress noted to date has consisted of minor alligator cracking at two isolated locations, one in Design A and one in Design D, and more or less general rutting throughout the project. It seems significant that both of the cracked areas also are near test sites where high deflections and low densities were recorded.

Rutting was measured downward from a string line stretched between the crown and the edge of the pavement. Measurements were made at each of the five test sites in each of the 24 groups. Results of the latest readings are as follows:

The most noticeable rutting on the project, at the location in Design A where slight cracking was also evident, was measured to be 1.188 in. deep, but is not included in the above averages because it did not occur at one of the regular test sites. It is noted that rutting on the average is now deepest in both wheel tracks in Design A. Average rutting in Design C was greatly reduced during the summer by the resurfacing of several hundred feet of the most distressed section.

TABLE 5

ROUTE 58 EXPERIMENT RUTTING IN INCHES, OCTOBER 1959

Pavement Design	Inner Wheel Path			Outer Wheel Path		
	Max.	Min.	Average	Max.	Min.	Average
A(9 in. A.C.)	0.750	0.063	0.263	0.875	0.125	0.332
B(7 in. A.C.)	0.313	0.063	0.143	0.375	0.125	0.263
C(5 in. A.C.)	0.563	0.000	0.204	0.438	0.000	0.215
D(4 in. A.C.)	0.250	0.063	0.170	0.438	0.125	0.306

In the attempt to determine the depths to which this rutting extended below the surface, elevations of the tops of all settlement rods (Fig. 5) have been measured. The results of all elevation determinations are shown in Figures 6, 7, 8 and 9. The zero line represents the original as built condition, and all deviations from this line represent elevation changes after six months and ten months of traffic.

It is interesting as well as puzzling to note how many of the so-called "settlement" rods appear to have risen rather than settled. This same condition apparently occurred in the WASHO test (6), and from unpublished correspondence it is understood to be happening again at the AASHO test in Illinois.

Nevertheless, it is felt that these settlement rod readings do present, in a qualitative sense at least, a picture of the movement of the various pavement layers relative to each other. The findings support those from the WASHO test report which states that "by far the greatest change in elevation in any section took place at the surface level (6)." Though settlement, like rutting, seems to be less pronounced in the stronger sections of the project (where compaction was better and deflections lower) some settlement of the surface did occur in the wheel tracks at every location. Inasmuch as the 4-in. level on the US 58 project is not below the asphaltic concrete in any of the designs, it seems obvious that much of this movement is the result of some sort of displacement within the asphaltic concrete. Further research along these lines, including observations of pavements in the field, would be desirable to discover what types of asphaltic concrete have the greatest resistance to this displacement.

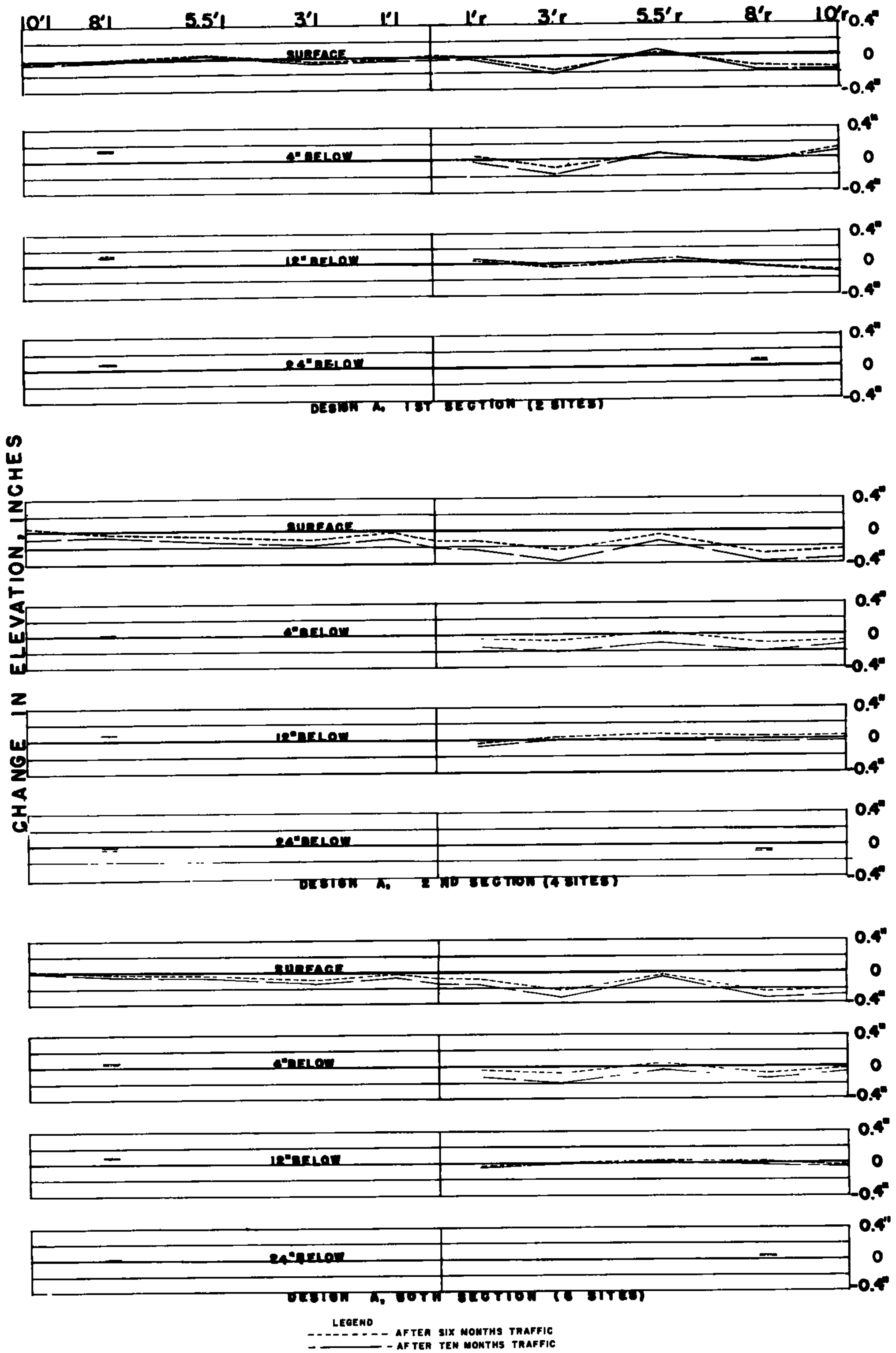


Figure 6. Changes in elevation of various layers in Design A.

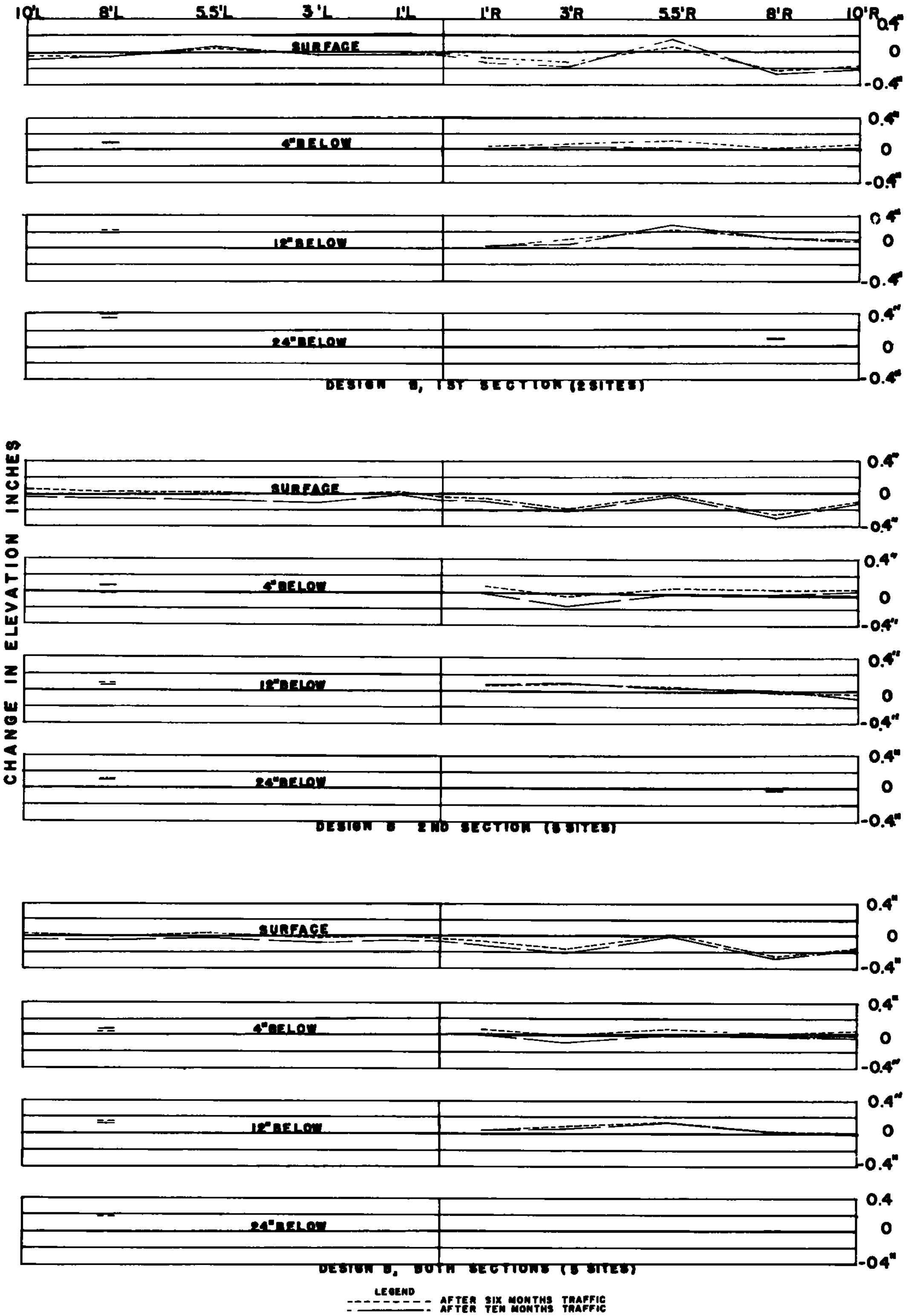


Figure 7. Changes in elevation of various layers in Design B.

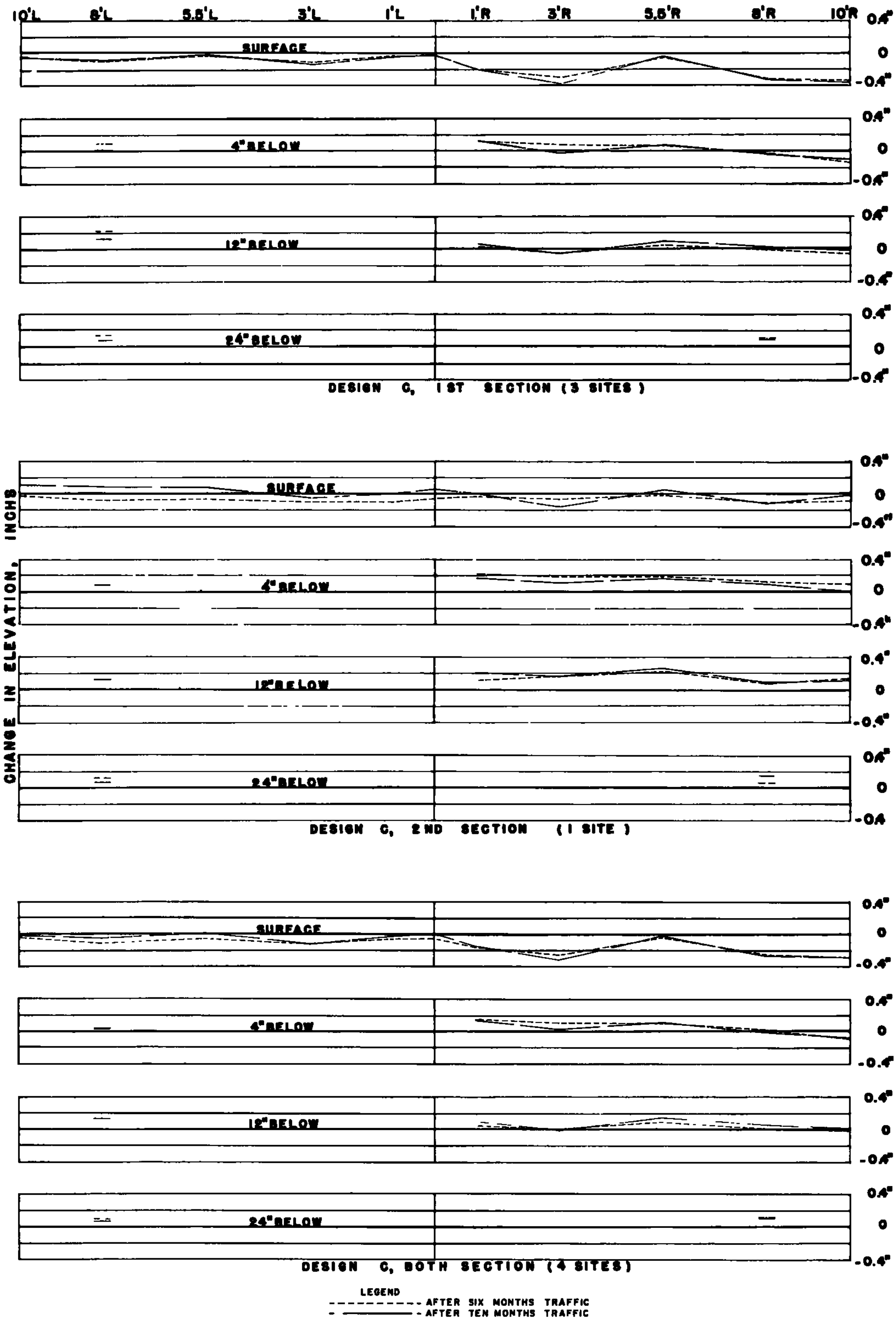


Figure 8. Changes in elevation of various layers in Design C.

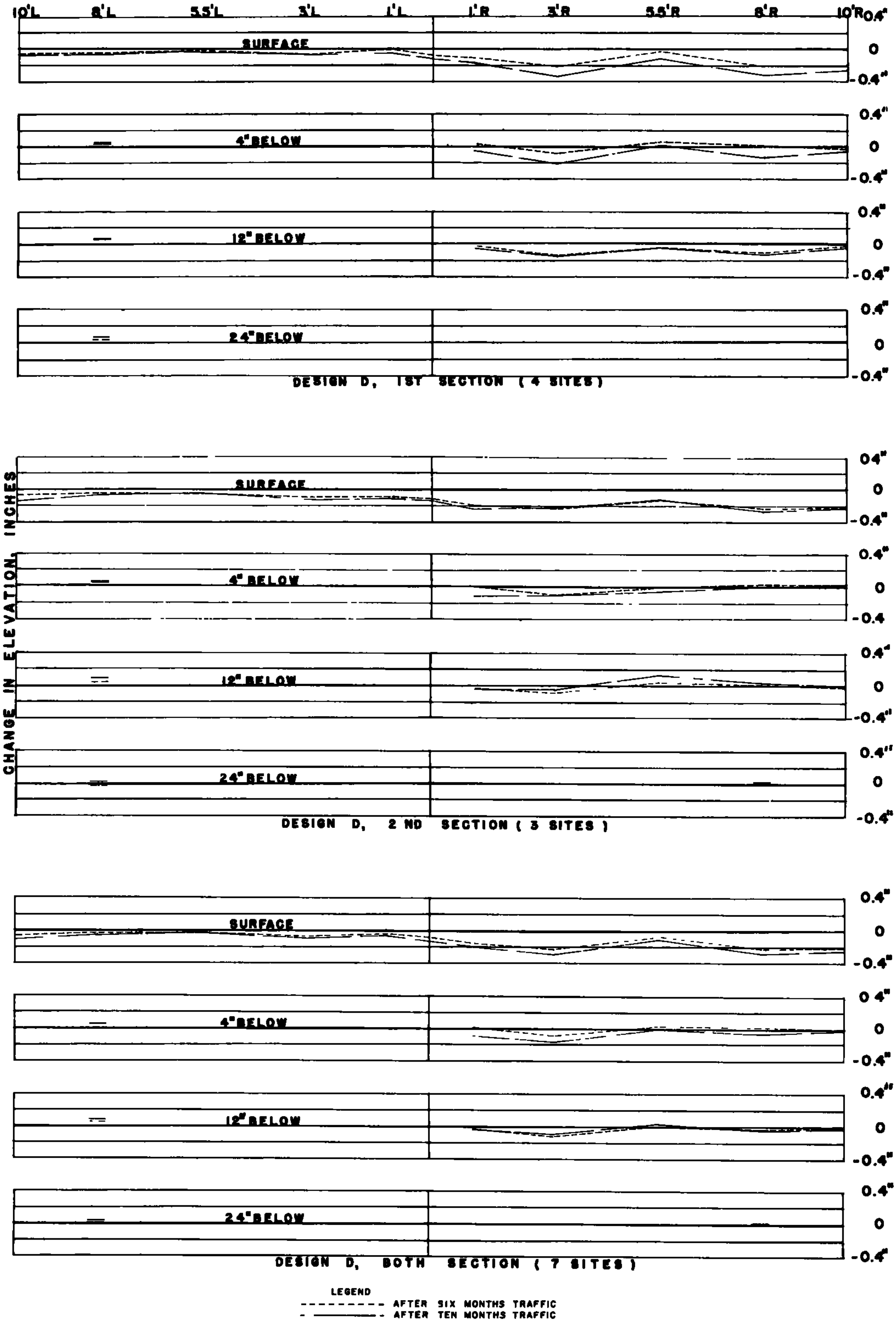


Figure 9. Changes in elevation of various layers in Design D.

SUMMARY AND CONCLUSIONS

To recapitulate briefly, the development of current pavement design practices in Virginia has been traced. Inasmuch as the CBR design method still is used only as a guide in establishing total pavement thickness (3), it was felt that the construction of experimental flexible pavements was in order so that the performance of different pavement designs built at the same time on the same project and subjected to the same traffic might be compared.

The first such experimental pavement, with four different pavement designs (Fig. 4), was completed and opened to traffic in December 1958. Its purpose was to compare thicknesses of asphaltic concrete of 4, 5, 7, and 9 in. in designs of the same total thickness. One major failure which occurred in April 1959, in a section with 5 in. of asphaltic concrete, has been described. Mention has also been made of the incidence of minor cracking at two other locations and of general surface rutting throughout the project. The Research Council's method of securing an earlier evaluation of the relative merits of the four pavement designs have been outlined in some detail.

The principal findings from the first experimental project are that:

1. Deflections and performance seem more closely allied with compaction than with pavement design characteristics. High deflections rather consistently occurred where construction densities were found to be low. Also, the one major failure did not occur in Design D, which had only 4 in. of asphaltic concrete, but in Design C, which had 5 in. It occurred near the point where the highest deflection as well as the lowest base course density on the project had been measured. Minor distress also is beginning in Design A, which had 9 in. of asphaltic concrete, and in Design D, in both cases again at points of high deflection and low density.
2. Deflections are somewhat less abrupt in the sections with greater thickness of asphaltic concrete.
3. Rutting in the wheel tracks has occurred to some extent throughout the project, regardless of pavement design. The deepest rutting on the project has occurred where the asphaltic concrete is the thickest.

It is too early to base definite conclusions on findings from this project. Also, conclusions based on a single experiment should never be considered final. Therefore, a second experiment has been designed and is scheduled for construction in 1960. Two of the designs from the US 58 project, Design B (7 in. of asphaltic concrete) and Design D (4 in. of asphaltic concrete), with minor revisions, will be repeated on the new project. Two other designs will include cement treatment of the crushed aggregate base material. All four designs will be built on a cement-treated subgrade. Special efforts will be exerted toward securing more complete data on compaction of all pavement components in the hope that an even more definite relationship between deflections and pavement densities can be established. Advice is being solicited from a University of Virginia faculty member, who serves as a Research Economist on a part-time basis with the Research Council, to assist in a more proper statistical analysis of the data.

But on the basis of early observations of the US 58 project, the following tentative conclusions have been advanced:

1. That greater slab action, with somewhat wider distribution of pressures to underlying layers, is afforded by increased thicknesses of asphaltic concrete.
2. That total deflection and the resultant poor performance can be minimized more effectively by increased emphasis on compaction of all components of the pavement than by "beefing up" the pavement design.
3. That rutting in the wheel tracks results primarily from displacements which occur within the top 4 in. of the pavement structure, and that some rutting may be expected in most asphaltic concrete regardless of the support offered from underlying layers.

Future research in Virginia will place great emphasis on compaction. Experiments are continuing for the purpose of evaluating and comparing the accuracy and precision of various methods of measuring field density. Better laboratory methods of determining realistic standard densities of all sorts of aggregate materials as well as asphaltic concrete are being sought also. Such research is aimed toward greatly improved control of compaction during construction. However, the evidence so far points strongly toward the necessity that the control of compaction be assigned to trained specialists and not relegated to junior inspectors as a collateral duty.

It is hoped that this report may encourage other agencies to set up experimental projects along similar lines to the one just described, incorporating variables which seem most important in the specific locality, and to report their findings at future meetings of the Highway Research Board.

ACKNOWLEDGMENTS

The planning and construction of an experimental project of this nature could not have been accomplished without the interest and cooperation of a great many people representing the Virginia Department of Highways, the Bureau of Public Roads, and the Thompson-Arthur Paving Company. While it is difficult to single out individuals, it is felt that the following persons deserve mention for their efforts: H.H. Harris, A.B. Cornthwaite, J.E. Johnson and K.E. Ellison, of the Central Highway Office, for their assistance in establishing the pavement designs; E.L. Alsop (now deceased), S.T. Barker, and R. Worthington, Resident Engineers under whom construction was accomplished and who furnished valuable assistance in the performance of research evaluation tests; R.V. Fielding, District Materials Engineer, for furnishing much information on the materials used; and particularly R.W. Gunn and E.C. Snell, Technicians with the Research Council, for performing all the deflection and density tests and most of the computations, drafting and related chores necessary to make this report possible.

Finally, the author wishes to express his appreciation for the encouragement and wise counsel furnished by Tilton E. Shelburne, Director of the Virginia Council of Highway Investigation and Research.

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Discussion

W.H. CAMPEN, Omaha Testing Laboratories—The writer has studied Mr. Nichols' paper on "Flexible Pavement Research in Virginia" and unless the writer has misinterpreted the preliminary performance behavior of the 1958 experimental road, the final results will be disappointing.

Having had many years experience with the design, control and performance of flexible pavements, the writer has come to the following conclusions in regard to the project:

1. The subbase and base were not compacted to proper density.
2. The subbase and base may not possess other desirable qualities even though they may have the desired CBR values.
3. The asphaltic concrete was not compacted to the proper density.

4. The asphaltic concrete may not have sufficient stability or other desirable qualities.

Certainly the subgrade must not be the cause of the distress because its CBR would have to be less than one, which would be too weak to carry the construction equipment for subbase construction.

Experience has shown that flexible pavement subbases and bases must be constructed with good aggregates and compacted to at least 100 percent of maximum laboratory density. The asphaltic concrete must possess high stability and must be compacted to at least 95 percent of maximum theoretical density, and if more than 2 in. thick, the lower layers should contain high percentages of large aggregate (3/4 in. or larger). In addition, at least, the upper 6 in. of the subgrade should be compacted to 100 percent of standard laboratory density.

FRANK P. NICHOLS, JR., Closure—Mr. Campen's many years of experience seem to have led him to some of the same conclusions with regard to this project that the author has attempted to put across in the paper. However, some of Mr. Campen's conclusions must have been reached intuitively, inasmuch as nothing was said in the paper regarding "other desirable qualities" of the base and subbase or the density of the asphaltic concrete. Most of the attributes prescribed by Mr. Campen for a successful flexible pavement are also prescribed in Virginia's Road and Bridge Specifications. It is believed that the quality and gradation of all aggregates and the percentages of asphalt used would have met with Mr. Campen's approval. The fact that in certain sections compaction was not adequate and that this has already affected performance was stressed in the paper.

It should be emphasized that the Research Council did not attempt to control the quality of the construction on this project, but preferred to leave that to the inspectors assigned for the purpose. It was desired that the construction be representative in quality to that of conventional projects, and it is believed that this desire was realized. While in some respects the results may be disappointing, it should be of more than passing interest that the least expensive design with superior construction may outlast the most expensive design with mediocre construction.

Finally, Mr. Campen's CBR design chart obviously is different from the one now in use in Virginia (3). For the volume of traffic now using this road, the 25-in. total structural thickness would be adequate for a subgrade CBR value of 4, but a CBR value of 1 would require a total thickness of 52 in. Fortunately, actual subgrade CBR values generally ran well above 4 on this project, but in certain areas the value was around 4. Actually, the major failure near the west end of the project occurred in one of these areas of low subgrade CBR value. This leads to speculation that weak subgrades contribute to poor pavement performance in two ways: they offer poor support to the finished pavement under traffic loads and they also offer poor support to the rollers attempting to compact subsequent layers.

Mr. Campen's comments are appreciated in that they called attention to these omissions in the original text.