

FINAL REPORT

EVALUATION OF EXPERIMENTAL FLEXIBLE PAVEMENTS

by

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(The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the sponsoring agencies.)

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SUMMARY AND RECOMMENDATIONS

A program of construction and performance evaluation of seven Virginia flexible pavements containing at least some experimental features is reported. The objective of the program was to evaluate the performance of the pavements incorporating new or timely design concepts and to assess the flexibility of these concepts for further use.

Among the major findings of the study are the following.

1. Pavements having equivalent design thickness indices are not necessarily equivalent in construction cost or in early structural strength.
2. Very early deflection tests do not give good indications of the ultimate strength characteristics of pavements having cement stabilized layers.
3. Full-depth asphaltic concrete pavements can give excellent performance in very poor soil areas, especially when the design is modified through the provision of a cement stabilized subgrade.
4. An unstabilized sandwich layer placed between a cement stabilized layer and asphaltic concrete layers is effective in significantly delaying the reflection of transverse cracking from the cement stabilized layer through the asphaltic concrete layers. There is some evidence that reflective cracks may develop after many years under heavy truck traffic.
5. Such a sandwich layer is weaker than either of the two layers it contacts and can cause a net reduction in pavement strength as compared with the situation where the weaker layer is on the bottom.
6. Transverse shrinkage cracks reflect from a cement treated stone subbase through 3 inches (75 mm) of bituminous concrete in as little as 18 months and through 7 inches (175 mm) of bituminous concrete in less than 5 years.
7. Cement treatment of stone subbases can be omitted in passing lanes with no detriment to performance. (This may not be true with traffic volumes near capacity because of the change in distribution of truck usage as that point is approached.)

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8. The thickness equivalency values used in Virginia pavement design computations have been confirmed as reasonable although there is evidence that --
 - a) the equivalency values of cement treated layers could be increased, and
 - b) it may be appropriate to reduce the equivalency values of sandwich layers.

The three following recommendations for consideration by administrators of the Highway and Transportation Department seem appropriate at this time.

1. The Department is encouraged to continue to consider the full-depth asphalt concept as a desirable alternative in flexible pavement design. In poor soil areas the designs should be modified to provide cement (or lime) stabilization of the native subgrade soil. Although full-depth design may be considerably more expensive than many alternatives, there is strong evidence reported herein that the full-depth pavements can provide performance somewhat better than that provided by most of these alternatives.
2. In cases where it is deemed appropriate to stabilize aggregate base materials on divided highways with four or more lanes where truck traffic is normally channeled into the outer lanes, it is structurally feasible to omit such stabilization from the inner or passing lanes. While in many cases there may be no economic advantage in such a practice because of construction difficulties, the concept is recommended for cases where it may be practically feasible.
3. The sandwich-layer approach to pavement design should be used with caution. There is strong evidence that such systems, as currently used, may not provide the desired long pavement life with minimal maintenance costs. Studies should be undertaken to confirm or dispel such suspicions and to develop criteria for the optimum use of these systems.

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INTRODUCTION

For a number of years the Virginia Highway & Transportation Research Council and the Federal Highway Administration cooperated in comprehensive performance studies of highway pavements of all types located in all sections of Virginia. The studies, which at one time included nearly 200 projects, resulted either directly or indirectly in an almost total modification of the Virginia approach to flexible pavement design. As a result of the studies highway engineers in Virginia are much more cognizant of soil resiliency, the benefits of cement or lime stabilization, and of the value of thick bituminous concrete layers.(1,2,3,4) In addition, Vaswani has utilized the results of the studies and those of the AASHTO road test in developing a strength coefficient design method for use in Virginia.(5)

The comprehensive studies were phased out at the end of calendar 1971, because most of the projects had reached the age where further study would not be profitable. On the other hand, recent innovations in pavement design are receiving attention so that occasionally new construction projects have features quite different from anything in the past. Examples are full-depth asphalt pavements and pavement systems in which the layers have been switched from their usual positions. Clearly, the evaluation of such projects is crucial to the determination of whether or not the experimental features should be adopted for routine pavement designs.

PURPOSE AND SCOPE

Since its conclusion, many highway agencies have been guided in their design practices by the results from the much publicized AASHTO road test of the late 1950's and early 1960's. The analysis of data from these tests has led to the gradual evolution of design practices involving strength coefficients of paving materials.(6) While the coefficients resulting directly from the road test are in wide use, there is general agreement that highway agencies should

evaluate their own materials and develop strength coefficients for them. The coefficients have been developed for Virginia materials and are in use for routine design purposes,⁽⁵⁾ but there is a need for continued evaluation to account for new concepts in pavement design. The evaluation of pavements where- in these concepts are applied is the thrust of the present study.

Specific objectives of the study are: (1) To evaluate the performance of pavements containing trial or experimental design features, and (2) to better define the strength coefficients of pavement components with respect to both materials and the location of those materials within the pavement structure.

Seven projects representing the Piedmont and Coastal Plains physiographic provinces are included in the study. Typical sections showing variations in types and locations of pavement elements are appended. The details of each project will be discussed later.

EVALUATION PROCEDURES

In general, the evaluation of experimental features began when substantial portions of the subgrade for a given project were prepared. At that time, dynaflect deflection tests were conducted on the subgrade. Similar tests followed the placement of subsequent pavement layers, including the final riding surface. Further steps in the evaluation of each project were as follows:

1. Procurement of final plans and cross sections, materials descriptions, construction costs, and date of acceptance from the contractor;
2. establishment of easily identified project limits by the use of roadside markers and written descriptions;
3. initial and periodic collection of data reflecting--
 - a. traffic characteristics,
 - b. structural capability as indicated by deflection tests,
 - c. visual defects such as cracking, rutting, patching, and settlements, and
 - d. occasional measurements of ride quality; and
4. compilation of records of major maintenance operations (bituminous concrete overlays, for example).

PROJECT NO. 1

U. S. ROUTE 29 BYPASS - ALTAVISTA

U. S. Route 29 Bypass at Altavista is a 4-lane roadway on an all new location. Completed in early 1974, the project is approximately 10 miles (16 km) long and is located in Campbell and Pittsylvania Counties. A detailed description of this project and of tests conducted while it was under construction were given in the first interim report.⁽⁸⁾ For completeness of the present report, some of the more important facets of the earlier report are synthesized below.

Soil Conditions

The nature of the embankment and subgrade soils on the Altavista project were determined during the preliminary engineering phase of the project. The details of these soil conditions are available in the project records. Briefly, the subgrade soils are predominantly micaceous silts from A-2-4(0) through A-5(10) and with California bearing ratio (CBR) values of from 5 to 16. Since these soils are categorized as highly resilient with poor bearing capacity, where they are used as subgrades in Virginia it has been conventional to stabilize the top 6 inches (150 mm) with portland cement. As has been reported elsewhere this stabilization has been found to provide a good working platform for construction equipment and to enhance long-term pavement performance.⁽⁴⁾ As indicated in the cross sections such stabilization was provided on three of the experimental sections on the present project, while the fourth design specified compacted native subgrade without stabilization.

Project Objectives

Specific objectives of the Altavista project are:

1. To evaluate the original relative structural strengths of the four pavement designs,
2. to evaluate the relative long-term performance of the four designs, and
3. to evaluate the comparative construction and maintenance costs for the four designs.

Results and Discussion

Deflections

Deflection tests were conducted at regular time intervals from the beginning of subgrade preparation until the project was opened to traffic. The first tests were conducted on the raw subgrade of Section B and the cement stabilized subgrade of Section A on August 8, 1972. Tests were then run on each layer of each section as test locations became available. Final tests on the finished surfaces of all sections were conducted during the fall of 1973. A detailed discussion of these earlier tests may be found in Interim Report No. 1. Annual deflection tests have been conducted on the project since its completion.

Design A

Design A, shown in Figure A-1, is of the general type often considered as the standard for many areas of the state and for that reason was chosen as the standard of comparison for the present study. The design sometimes is referred to as the sandwich system, indicating a weak layer of crushed stone between strong layers of bituminous concrete and a cement stabilized material.

The total pavement strength developed by Design A by the time construction was completed appears to be at least as high as required by design parameters, as evidenced by the thickness index of 15.0 determined from the deflection tests compared to a design index of 12.0. It should be noted here that the thickness index computed from deflection tests is only an approximation of the true index because the computations involve certain assumptions concerning the strength characteristics of the various materials used in the pavement structure. No appreciable changes in deflections or in thickness index were detected when the pavement was 1 year old. However, by the time this design was 3 years old there was a substantial increase in deflections and a corresponding reduction in the effective thickness index. While this change may have resulted from normal wear and tear there were some unusual weather conditions during the third and fourth years of the pavement's life. The third year was unusually wet and the fourth was unusually cold.

Deflections and thickness indices for 1978 and 1979 show considerable fluctuation suggesting changes in moisture content in lower pavement layers.

By the spring of 1977 some distress in the form of alligator cracking was becoming apparent in the Campbell County portion of the design type. Substantial differences in the performance between the Campbell and Pittsylvania County portions of the design suggest some differences in soils not detected in the design phase of the project. The two counties are separated by the Roanoke River such that soils of different characteristics would not be unusual.

A thorough reexamination of original soil survey test reports showed no significant difference in CBR, dry density, and soils classification between the Campbell and Pittsylvania County sections. By 1979 cracking had begun to develop in both the Campbell and Pittsylvania sections.

Design B

Design B, sometimes referred to as a modified full-depth asphalt pavement because, except for the cement stabilized subgrade, the pavement is made up entirely of bituminous concrete, shows strength development under construction very much as would be expected. As can be seen in Figure A-2, the majority of the strength development can be attributed to the 8-inch (200 mm) bituminous concrete base course.

In this case, as with Design A, the thickness index data show that the completed pavement had a structural strength at least equivalent to design requirements. Again, the measured thickness index, 14.0, compared favorably with the design value, 11.9.

After approximately 1 year, the deflections on this section were substantially lower than when the pavement was first completed. It is speculated that this reduction in deflections was brought about by seasonal variations in subgrade moisture and in the stiffness of the thick asphaltic concrete. Vaswani has shown these variations to have a significant influence on both the deflections and indicated thickness index of typical Virginia pavements.⁽⁹⁾ When the pavement was 3 and 4 years of age the deflections had increased to approximately the values measured immediately after construction and there were some small variations in the thickness index values.

Longitudinal and alligator cracking appeared by the spring of 1977 in Campbell County and in Pittsylvania County by the spring of 1979.

Design C

Experimental Design C (Figure A-3) is the only one of the four constructed with an unstabilized subgrade soil, and as a result a poorer working platform was provided for the contractor's equipment. This factor, together with extremely wet construction seasons and very poor soil conditions, resulted in some construction difficulties. These problems were evidenced (1) in the need to apply lime as a drying agent to certain saturated portions of the subgrade soil, (2) in the distortion of the prepared subgrade soils under construction traffic, and (3) primarily in the very early failure of one segment of the 4-inch (100 mm) cement treated crushed stone subbase under construction traffic. This subbase failure was corrected by the provision of 4% cement by weight to the previously unstabilized layer of crushed stone base.

Apparently because of the soil problems and construction difficulties discussed above, Design C had a completed average thickness index of 9.5, which did not compare well with the design index of 12.0. However, after about one year of service the pavement was significantly stronger, with an indicated thickness index of 13.0. Deflections for the third through sixth years suggest that this variation was a seasonal effect, as mentioned earlier, and not a permanent increase in pavement strength.

This design, as did Designs A and B, showed some longitudinal cracking by the spring of 1977 in Campbell County and in both counties by the spring of 1979.

Design D

Design D, which contains two cement layers, produced the strongest finished product of the four designs. The deflection data shown in Figure A-4 are self-explanatory, and it suffices to point out that the final deflections are the lowest and the final thickness index the highest of those measured for the four designs. Clearly, the final thickness index of 15+ compares very favorably with the design index of 11.9.

It is important to note here that while this type of design has been known to perform extremely well, there are reflection cracks from the cement treated crushed stone underlying the relatively thin bituminous concrete course. Furthermore, as has been reported earlier from a study of a similar pavement,⁽¹⁰⁾ the extreme rigidity of this design coupled with the tendency of the cement treated stone to crack both transversely and longitudinally can result in behavior very much like that of a concrete pavement. Thus, if water becomes trapped under the stabilized stone a pumping

action can occur to the detriment of pavement performance. The performance of this particular section has been watched closely for any evidence of this phenomenon. By the spring of 1977 some transverse and longitudinal cracks were evident in only the Campbell County portions of this design. By 1979 transverse cracks were evident in Pittsylvania County and very numerous in Campbell County.

General Evaluation

A general evaluation of Project No. 1 based on 5 years of service was undertaken in mid-1979. As indicated above, the project had shown some distress earlier than would be anticipated for the design parameters and deflection test results given. The fact that this distress is having an adverse effect on the performance of the project is indicated in Table 1, where May's roughness results for 1977 and for 1979 are summarized. It can be seen from these results that the distress is causing a decrease in ride quality with time.

Table 1
Performance and May's Roughness
Project No. 1

<u>Design</u>	<u>Performance</u>	<u>Roughness (in/mi)*</u>	
		<u>September 1977</u>	<u>May 1979</u>
A	Fair	55	62
B	Good	59	66
C	Fair	64	73
D	Fair	63	71

*Metric conversion, 1 in/mi = 1.6 cm/km.

Based on the degree of cracking and the ride quality in May 1979, the performance ratings of the four designs have been re-evaluated as given in Table 1. These ratings, where only Design B is considered "good", contrast sharply with the 1978 ratings, where all sections were considered "excellent".(11)

With reference to the reduced performance occurring over the past several years, it is noted that the actual traffic (EAL-18) has ranged from two to three times the design value for the first 5 years of the pavement's life. Design computations using actual traffic volumes yield a required thickness index of approximately 14.0 as compared with the original of 12.0.⁽⁵⁾ In light of this development, it may be justifiable to consider all four designs to have performed well.

Evaluation of Thickness Equivalencies

Through the use of a method proposed by Vaswani, the effective thickness equivalency values of the materials used in project No. 1 have been determined.⁽¹²⁾ These effective values are summarized in Table 2, along with those used in design computations.⁽⁵⁾ The effective values given are the averages for the life of project No. 1.

Table 2

Thickness Equivalency Values

<u>Material</u>	<u>Effective "a"</u>	<u>Design "a"</u>
Asphaltic concrete	0.96	1.00
Soil cement	0.91	0.40
Aggregate base	-0.17	0.35
Cement treated base	0.88	0.60-1.00

As may be seen in the tabulation, the effective equivalency values vary somewhat from those assumed in the design computations. Note that the asphaltic concrete is performing very nearly as expected while the soil cement treated subgrade is contributing considerably more to the structural strength of the pavement than it had been assumed it would. Cement treated aggregate base, with an effective "a" value of 0.88, is performing almost precisely as expected (its "a" value depends upon its location in the pavement system). Untreated aggregate base, however, has not contributed as much to the structural strength as it had been expected to. In fact the effective "a" value of -0.17 indicates that, on average, the untreated aggregate has detrimentally affected the pavement strength. This is clearly seen in the effective thickness indices. Design C, for example, would have a theoretical thickness index of 9.9 if the untreated aggregate layer were omitted. The average effective

thickness index for this design, including a 6-inch (150-mm) layer of untreated aggregate, has been 9.7 for 5 years. No doubt this design is manifesting the sandwich-layer effects pointed out by Vaswani, where he cautioned that such layers should not exceed a thickness of 4 inches (100 mm).⁽¹³⁾ The reader is cautioned here that the negative thickness equivalency does not necessarily mean that the untreated aggregate is a detriment to overall pavement performance. Thickness equivalencies in Virginia are developed around the concept that reduced deflections and reduced curvature of the deflection basin will prevent premature distress of the upper asphaltic concrete layers. The other design consideration, not reflected by thickness equivalencies, is the possibility that the subgrade may be overstressed if too thin pavements are used. The untreated aggregate, while detrimental in the former case, is no doubt helpful in the latter. For this reason, careful study should be given to any moves to omit the aggregate layer.

Conclusions

The following conclusions are based on tests conducted during construction and since completion of the Altavista project. Because pavement characteristics may change with continued exposure to traffic and changing climatic conditions, no definite indications of ultimate pavement performance are offered in this report. Cost data supporting conclusion number 2 were given in Interim Report No. 1.⁽⁸⁾

1. Pavements having equivalent design thickness indices are not necessarily equivalent in early structural strength.
2. Pavements having equivalent design thickness indices are not necessarily equivalent in construction costs.
3. Very early deflection tests are not good indicators of the ultimate strength characteristics of pavements having cement stabilized layers.
4. Highly resilient soils, especially micaceous silts, must be stabilized to achieve a good working platform for pavement construction and to assure the early development of the design structural strength.
5. A design utilizing a cement stabilized subgrade overlaid with a cement stabilized stone base and bituminous concrete develops the design structural strength more rapidly and at a lower cost than do any of the other three designs.

6. Substantial differences in the performance of all four designs between the two counties the project traverses suggest that some variations in soil characteristics were not detected in the original design provisions.
7. The design thickness equivalency values of the materials used in this project are reasonable with the exception of the untreated aggregate base, which assumed a negative equivalency value due to the sandwich-layer effect.

PROJECT NO. 2

ROUTE 31 - WILLIAMSBURG

Project No. 2 (Figure A-5) was the first full-depth asphaltic concrete project built in the state. The pavement was completed in 1970, is 1 mile (1.6 km) long, and runs from the west corporate limits of Williamsburg toward the downtown area. Because the project is part of the feeder road for the Jamestown ferry it does not carry a large volume of truck traffic. Even though traffic projections called for a design daily EAL-18 of 118 for the year 1978, traffic counts show that the actual daily EAL-18 has been only 4 to 20 for the first 9 years the project has been open to traffic. (7)

Soil Conditions

Soils encountered on the project were silty clays, sandy clays, and clays falling into AASHTO classifications A-2(4), A-6(2), and A-7-6(11). CBR values ranged from 1.3 to 11.7, with an average of 8.0.

Typical Section

The typical section for the Route 31 project, given in Figure A-5, consists of a 9-inch (225 mm) bituminous concrete base built on the native subgrade. This 9-inch (225 mm) base was constructed in one lift. The surface course is bituminous concrete type S-5 applied at a rate of 165 psy (89 kg/m²).

The pavement is 4-lane undivided with a paved width of 48 feet (14.6 m). A crushed stone base was provided under the curb and gutter on either side of the roadway.

Project Objectives

Objectives of the project were to evaluate the performance of a full-depth bituminous concrete pavement on an untreated subgrade and to assess the strength coefficient of the full-depth base. As has been noted earlier, it is customary in Virginia to use cement stabilization in the silty subgrade soils such as those found on this project. However, in an effort to expedite construction in an urban area and to provide a more severe test for the full-depth bituminous concrete, such stabilization was omitted from this project.

Results and Discussion

Placing the full-depth bituminous concrete base in a single lift on the unstabilized subgrade soil presented some construction problems. The absence of a good working platform made it difficult to maintain an accurate grade as the weight of the paving machine was enough to significantly deform the subgrade in weak areas. The 1-1/2-inch (38 mm) surface course was insufficient to iron out these irregularities so that a rough riding surface resulted. The rough condition was aggravated by the fact that the paving contractor was unable to furnish materials to the paving machine with any degree of continuity, because project personnel simply were not accustomed to placing 9 inches (225 mm) of bituminous concrete in one lift and adequate trucking arrangements were not available. Hughes and Maupin have discussed the construction and materials aspects of this project in an earlier report.⁽¹⁴⁾

Deflection tests on the project initiated soon after its completion in the summer of 1970 (Figure A-5) showed a marked decrease in deflections during the first 9 months the pavement was in service. Then, the deflections appeared to level off for the subsequent 4 years and increased somewhat during the seventh through ninth years. It should be noted that the early deflections were in warm weather while the latter were for the cooler weather of spring. Tests at other times on the same project showed that deflections on the full-depth pavement were extremely sensitive to changes in pavement temperature. Thus, it could be expected that summer and spring deflections would differ significantly.

The application of a subgrade evaluation method suggested by Vaswani⁽¹²⁾ shows that the subgrade conditions under the full-depth asphalt have not varied to any appreciable degree since the project was constructed. For this reason, it is surmised that changes in deflection are due to changes in the stiffness of the asphalt layer caused by temperature changes.

It should be noted that the thickness index of this project determined from deflection tests is very much as predicted from design parameters, which suggests that an asphaltic concrete strength coefficient of 0.95 is applicable to full-depth asphalt pavements with the depth limitations found in the current project. Virginia designers have used this reduced coefficient for full-depth asphalt under the assumption that the material deep in the pavement structure contributes less to the total pavement strength than does that close to the surface.

Only one minor problem was noted on the 7-year-old full-depth pavement. This consisted of longitudinal cracking on a fill near the northern end of the project and is attributed to fill settlement rather than to a pavement deficiency.

Conclusions

Based on 9 years of tests and observations, the following conclusions appear applicable to the first full-depth asphalt pavement built in the state. These conclusions are somewhat tempered by the fact that actual truck traffic using the pavement is nowhere near that for which the pavement was designed.

1. Full-depth asphaltic concrete pavements in a poor soil area can give excellent performance.
2. A reduction in strength coefficient for full-depth asphalt appears to be justified, when the total pavement thickness is as much as the 10-1/2 inches (265 mm) on the project reported herein.

PROJECT NO. 3

ROUTES 3 AND 33 — MIDDLESEX COUNTY

Project No. 3 is included in the study as an example of pavement performance when the positions of pavement layers are altered within the pavement structure. In this case, a cement stabilized sand and gravel subbase layer was used in two different positions. As originally designed, the stabilized subbase material was to be located directly under the asphaltic concrete layer. However, in an effort to reduce the reflection of transverse cracks from the stabilized layer through the asphaltic concrete layers the Research Council provided impetus to the construction of a 1-mile section of the pavement with an unstabilized subbase layer sandwiched between the asphaltic concrete and the stabilized subbase.

In suggesting this test section it was envisioned that the unstabilized subbase layer would function as a cushioning layer to prevent the reflection of the transverse cracks. The design traffic for the project was 115 daily EAL-18.

Soil Conditions

Soils encountered on the project were sands and sandy clays falling into AASHTO classifications A-2-4(0) to A-6(7). CBR values ranged from 15 to 65, with an average of 28.

Typical Sections

Typical pavement sections for project No. 3 are given in Figures A-6 and A-7, where it may be seen that the layers described above consist of 6 inches (150 mm) of select borrow having a minimum CBR value of 25. In each case the stabilized layer contains 8% portland cement by volume. Actual construction consisted of the provision of 12 inches (300 mm) of the select borrow with either the top or bottom one-half stabilized as required.

Both the experimental and the standard sections are overlaid by a 3 inch (75 mm) asphaltic concrete base course and a 1-1/2 inch (38 mm) asphaltic concrete surface course.

Project Objectives

The primary objective of the project was to assess the effectiveness of the cushioning layer of unstabilized material in preventing the reflection of transverse cracks through the asphaltic concrete layers. Secondary objectives were to assess the effects on the pavement structure of switching the positions of the stabilized and unstabilized select borrow layers and to assess the overall performance of the two designs, which were of equivalent cost but of different structural properties.

Results and Discussion

Studies of the two pavement designs included in this project for the first 9 years in service have shown that the provision of the unstabilized cushion layer has been effective in preventing the development of reflection cracks in the asphaltic concrete layers. However, such cracking has not been as severe as anticipated even in the sections where the stabilized material is directly under the asphaltic concrete. In this section, the first

reflection cracks were noted at 3 years, but present no significant performance problem even after 9 years. At 7 years of age a small amount of transverse cracking was detected in the section containing the cushioning layer.

While the cushioning layer has served its purpose of reducing reflection cracks, deflection tests on the two test sections have shown that this layer has also served to weaken the pavement structure. As can be seen in Figures A-6 and A-7, tests conducted on both sections on the same dates show deflections on the section with the cushion layer to be significantly higher than those on the other section. At the same time, differences in spreadability values indicate that the pavement with the stabilized subbase close to the surface is much stiffer than the one with the sandwich, or cushion, layer. The end result is that the thickness index as determined from deflection tests is much lower for the pavement with the sandwich layer, as would be predicted from the design index, which gives consideration to a material's location within the pavement structure.(5) By the spring of 1977, the design with the sandwich layer showed a substantial amount of longitudinal cracking and by 1979 a noticeable decrease in effective thickness index had occurred. The effective thickness equivalency for the select borrow sandwich layer was determined to be -0.20.

It should be noted that truck traffic volumes on these pavements are so low (20-30 daily EAL-18) that the weaker sandwich layer has not been a serious detriment to pavement performance for the first 9 years. While substantial differences in performance seem to be developing, they are unlikely to be serious unless there is a substantial increase in truck traffic, which may never reach the 115 daily EAL-18 design value.

Conclusions

As with some of the previous pavements discussed, the conclusions drawn from project No. 3 must be tempered with the knowledge that truck volumes (EAL-18) are low so that such conclusions could not necessarily apply to similar pavements under more severe conditions. With this provision, the following conclusions appear reasonable.

1. An unstabilized sandwich layer placed between a cement stabilized subbase and asphaltic concrete layer is effective in preventing the reflection of transverse cracks from the cement stabilized layer through the asphaltic concrete layers.

2. Such a sandwich layer is weaker than either of the two layers it contacts and causes a net reduction in pavement strength when compared with the situation where the weaker layer is on the bottom.

PROJECT NO. 4

ROUTE 122 - BEDFORD COUNTY

Project No. 4 was included in the study to provide an opportunity to evaluate the performance of a pavement in which a cement stabilized, commercial crushed stone subbase was used in lieu of cement treatment of a micaceous, highly resilient subgrade soil. While the project is very similar to No. 1-C it was built earlier (1968), and is in a lighter traffic corridor. A design daily average EAL-18 of 25 was approximated very closely by actual values of from 19 to 23 for the first 9 years the pavement was in service. During the last 2 years the EAL-18 has increased to 44.

The project is also similar to No. 3 in that a cushioning layer is expected to prevent reflection cracking from the lower cement stabilized layer. The difference in this case is that a commercial crushed stone was used rather than a local select borrow.

Soil Conditions

Soils encountered on the project were micaceous silts and silty clays characteristic of the Piedmont Province. These soils ranged in CBR value from 0.6 to 7.2, with an average of 4.7.

Typical Section

The typical section for the project given in Figure A-8 shows two layers of aggregate base, the lower of which is stabilized with portland cement at a rate of 4% by weight. Both layers were constructed of a commercial crushed stone meeting the requirements of Virginia specifications in effect in 1968. The stone layers are overlaid by 5-1/4 inches (132 mm) of asphaltic concrete comprising 3 inches (75 mm) of base, 1-1/4 inches (31 mm) of binder, and a 1 inch (25 mm) surface course.

Project Objectives

Objectives of the project were (1) to assess the ability of the untreated stone layer to prevent the reflection of cracks in the stabilized stone through the asphaltic concrete layers, and (2) to evaluate the performance of the design in which a cement treated crushed aggregate subbase was used in lieu of the usual practice of stabilizing the highly resilient subgrade soil.

Results and Discussion

Observations and deflection tests on the project for the first 9 years it has been in service show that it has performed extremely well. Most instances of observed distress are related to fill settlements and embankment slides rather than to defects in the pavement structure. Again, it should be recalled that the pavement is subject to a low volume of truck traffic. This volume has been accounted for in the design of the pavement structure as evidenced by the design thickness index of 9.7.

Thickness indices determined from deflection tests show that the design index has been realized in service. Measured indices range from 8.5 to 14.0 and reflect the seasonal variations mentioned on other projects.

The computed average thickness equivalency value for the crushed stone sandwich layer is 0.22, much better than the value determined earlier for select borrow and select material, and still substantially less than the 0.35 used in design computations.

The unstabilized layer has been successful in preventing the reflection of transverse cracks through the asphaltic concrete layers. After 7 years in service the project showed only a small amount of transverse cracking. Longitudinal cracking began to develop during the eighth year and was becoming substantial by the spring of 1977. The project was sufficiently distressed to justify resurfacing by 1979, its eleventh year in service.

Conclusions

The following conclusions appear reasonable for this relatively low traffic road in which a sandwich layer of crushed stone is provided.

1. The sandwich layer has been effective in preventing reflection cracks.
2. The pavement has developed adequate structural strength to give excellent performance under the prevailing conditions.

PROJECTS NO. 5 AND 6

ROUTE 207, CAROLINE COUNTY

Projects No. 5 and 6 were incorporated in the study to provide a comparison of a full-depth asphalt design with a more conventional design. While the two projects were built at different times, they are adjoining and both serve the same heavy truck traffic, having a design EAL-18 projection of 919 per day for the year 1978. The projects provide trucking access from U. S. 301 near Bowling Green to Interstate 95. Traffic for the first few years the pavements have been in service does not approach the design projections, but does exceed 300 daily EAL-18, making the road one of the heavier trucked primary highways in the state. The similarity of both soil and traffic conditions provides a good comparison of the two pavement designs.

Soil Conditions

The soils encountered on the two projects are typical Piedmont - Coastal Plains transitional soils consisting of silts, clays, and sands. CBR values range from 5 to over 100, with an average of 18. AASHTO soil classifications on both projects range from A-1-a(0) to A-7-6(9), with the A-2-4(0) being predominant.

Typical sections for Projects 5 and 6 are given in Figures A-9 and A-10, respectively. As shown in these sketches the lower 6-inches (150 mm) of subgrade soils on both projects were stabilized with 8% portland cement by volume. The standard design (project 5) has a 6-inch (150 mm) layer of subbase material consisting of a commercial crushed stone size 21-A. The asphaltic concrete layers comprise 6-inches (150 mm) of base, 1-1/2-inches (38 mm) of binder, and 1-inch (25 mm) of surface.

The full-depth asphalt on project 6 consists of 9 inches (225 mm) of base, 1-1/2 inches (38 mm) of binder, and 1 inch (25 mm) of surface. Notwithstanding the similarities in soils and anticipated traffic conditions, a modification in design procedure between the times the two projects were designed resulted in design thickness indices of 13 and 15 for projects 5 and 6, respectively.

Objective

The objective of the two projects was to contrast the performances of the full-depth asphalt and standard pavement designs under similar soil and traffic conditions.

Results and Discussion

Performance studies have been conducted on project 5 since its completion in late 1971 and on project 6 from the beginning of construction.

The performance of project 5, the standard design, has been highly disappointing in that after only 3-1/2 years of service the asphaltic concrete surface was extensively cracked longitudinally and had occasional areas of alligator, or "map," cracking that were close to the pothole stage. The project was resurfaced with 150 psy (81 kg/m²) of bituminous concrete type S-5 in October 1977. The overlay was applied after only about 700,000 18-kip axle load repetitions as opposed to an expected 2.5 million repetitions before an overlay would be necessary. Some improvement in the effective thickness index resulted from the overlay.

The cause of the poor performance of this pavement is not immediately evident although the deflection tests show that the design thickness index has never been developed by the pavement structure. The project again uses the sandwich-layer approach, which may account for the lower than anticipated effective thickness index. The 1976 and 1977 deflection tests reflect the observed deterioration of this pavement as shown by the increased deflection and declining effective thickness index. The early indicated index of 11.0 should, however, have been adequate for the traffic the pavement has experienced to date. This lower than expected effective thickness index coupled with the observation that moisture is present on the surface of the pavement at least a day after precipitation led to speculation that a subsurface drainage problem existed. The "trench" construction and densely graded subbase material used on this project compare well with several similar projects where exploratory diggings had shown the subbase to be saturated and water to be trapped in the asphaltic concrete layers. In such cases, failure of the asphaltic concrete layers can be extremely rapid. In an effort to determine the subdrainage conditions of this pavement, in June 1977 two sites in the outer wheel path of the traffic lane were opened to the soil cement treated subgrade. One site is shown in Figure 1; the other was in somewhat better condition. At both sites it was found that the bituminous concrete base material was moist and had stripped so badly that in the wheel paths the material was removable with a hand shovel. The condition of this base material is indicated in Figure 2.

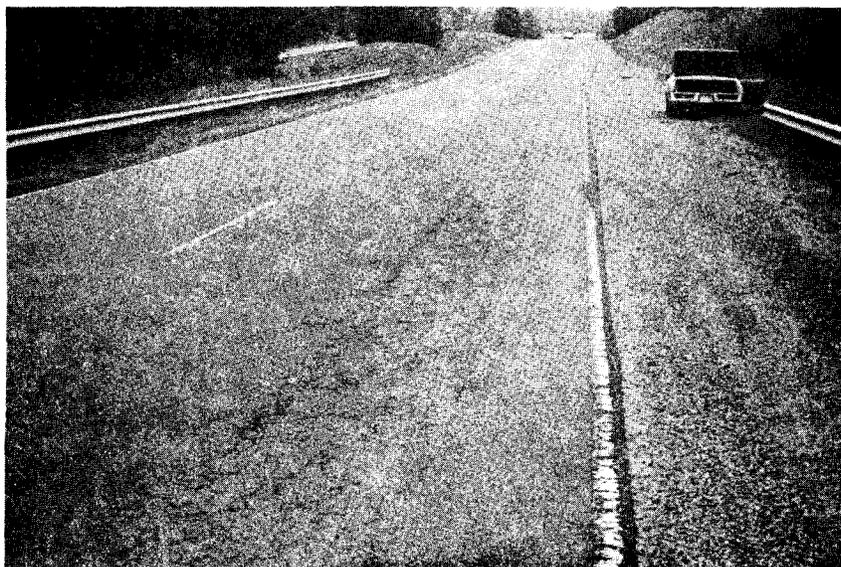


Figure 1. Surface condition of section analyzed.

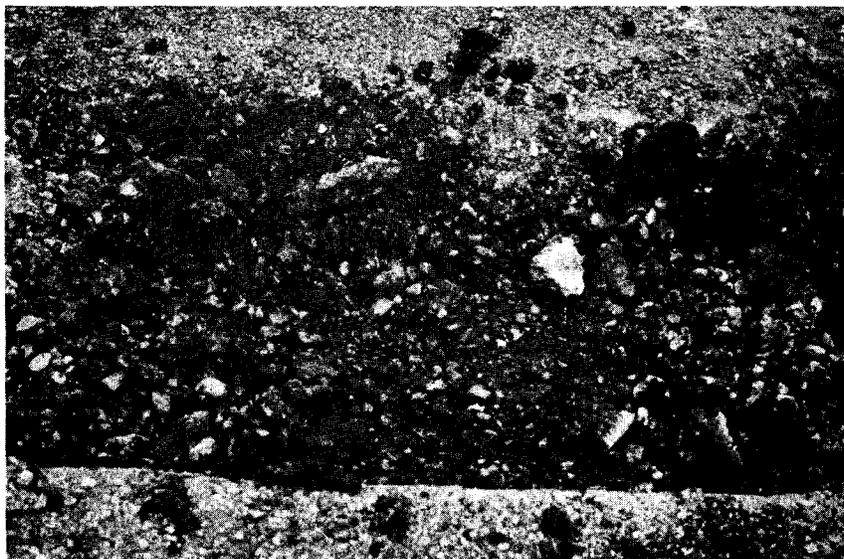


Figure 2. Condition of bituminous concrete base.

The aggregate subbase was damp at both sites, even though the project was located in an area experiencing a severe drought. Gradations and Atterburg limits on samples of the aggregate showed that the material was nonplastic with approximately 6% minus 200. Experience in Virginia has shown that such materials will drain adequately, if slowly, as long as there is an outlet for the water. In the case of the two sites examined on this pavement the shoulders were built of a clayey sand which would doubtless be of very low permeability as compared to the subbase material. Thus, it is likely that the shoulder material forms an impervious dam at the pavement edge such that surface water is held in the lower pavement layers and results in stripping and loss of integrity of the bituminous layers. Any potential concerns about the condition of the cement stabilized subgrade soil were allayed when the aggregate base was removed to reveal the hard, dense material indicated in Figure 3.

The full-depth asphalt pavement (Project 6) has performed extremely well for its 5 years under traffic. One isolated problem consists of what appears to be a saturated area where water occasionally flows from the pavement surface. This condition may be caused by a spring that was undetected during construction.



Figure 3. Condition of soil cement subgrade.

Several saturated areas that were detected during construction have not become evident in the completed pavement. While this project is still too new to provide definite conclusions, it is evident from the deflection tests that the design thickness index of 15 is being realized in the pavement in service. The effective thickness equivalency of the soil cement on this project is 0.58 as compared to a design value of 0.4.

Conclusions

The extremely poor performance of the standard design appears related to the "trench" construction method which prohibits proper drainage of lower pavement layers and to the use of a sandwich layer of untreated aggregate base.

PROJECT NO. 7

ROUTE 7360 — KEYSVILLE BYPASS

Project No. 7 is the only one carried over from a previous study of experimental flexible pavements. The project was completed in December 1965 and has generally performed well under heavy truck traffic for over 14 years. The construction features and early performance history of the project were reported in detail earlier. (3,10)

Traffic projections on the project called for a design daily EAL-18 of 977. While this volume has not been reached, there was a steady increase from 1966 through 1971 to a peak of 577 daily EAL-18. The truck volume decreased to 464 daily EAL-18 in 1972, increased to 518 in 1974, and dropped somewhat in 1976-79.

Soil Condition

The soils native to the project are micaceous silts and silty clays of the A-4 to A-7-5 classification. CBR values range from 3.0 to 11.5.

Typical Section

The typical sections for project No. 7 are given in Figures A-11 through A-14, designated Sections A through D. Section A represents the more or less standard design Virginia has used in

the Piedmont area for some years. Sections B through D include design variables in the subbase, base, and asphaltic concrete layers. All sections were considered to be structurally equivalent at the time of construction. The only feature common to all four designs is the cement stabilization of the upper 6 inches (150 mm) of the native subgrade soil.

In this project, for the first time in the experimental studies, advantage was taken of the much lower truck traffic in the passing lanes. Since the additional strength was deemed unnecessary, the cement stabilization was omitted from the crushed stone used in the westernmost eastbound sections of Designs B and C. The feeling was that the somewhat objectionable transverse shrinkage cracks could be avoided with no loss in performance.

Among the structural features indicated earlier are three designs (A, B, and C) having thick bituminous concrete surface and base courses. Seven inches (175 mm) of bituminous concrete in these designs were considered equivalent to 3 inches (75 mm) of bituminous concrete and 4 inches (100 mm) of cement treated stone. Similarly, in the subbase 4 inches (100 mm) of bituminous concrete, 4 inches (100 mm) of cement treated stone, and 6 inches (150 mm) of untreated crushed stone all were considered equivalent.

Thus, also for the first time in the experimental projects, some consideration was given to the thickness index concept developed at the AASHTO road test and reported for Virginia materials by Vaswani.⁽⁹⁾ Note that the total pavement thickness is 17 inches (425 mm) for Designs B, C, and D, while Design A, with only a crushed stone subbase (sandwich layer), is 19 inches (475 mm) thick. Thickness indices⁽⁹⁾ are 9.4, 13.9, 13.4, and 13.4 for Designs A, B, C, and D, respectively.

Project Objective

The objective of the project was to continue the evaluation on older major experimental projects where some definite indications of differences in performance had been detected earlier.⁽¹⁰⁾

The comparisons to be made from the designs were given by Hughes as:

1. "Base — compares 4 inches (100 mm) of cement treated aggregate base (Design B) with 4 inches (100 mm) of asphaltic concrete base (Design C). (The additional 1-1/2 inches (38 mm) of asphaltic concrete base plus the 1-1/2 inch (38 mm) surface are assumed to be equivalent to the 3-inch (75-mm) surface of Design B.)

2. Subbase — compares 4 inches (100 mm) of lean mix asphaltic concrete (Design D) with 4 inches (100 mm) of cement treated aggregate base (Design C).
3. Subbase — compares 4 inches (100 mm) of lean mix asphaltic concrete (Design D) with 6 inches (150 mm) of untreated aggregate base (Design A).
4. Subbase — compares 4 inches (100 mm) of cement treated aggregate (Design C) with 6 inches (150 mm) of untreated aggregate base (Design A).
5. Compares EBPL to EBTL of S. Section of Design B and Design C to determine effect of omitting cement from lightly traveled passing lane."(3)

Results and Discussions

Visual Defects

The first visually apparent defects on this project were transverse shrinkage cracks found in Design B. These were reported by Hughes⁽³⁾ when the project was about 2 years old and had sustained about 0.3 million EAL-18 (one direction). The report indicated that these cracks had reflected through 3 inches (75 mm) of bituminous concrete within the first 1-1/2 years. At the same time one transverse crack was reported in Design C (7-1/2 inches [188 mm] of bituminous concrete overlying cement treated stone).

Later studies showed more fully developed crack patterns after 5 years under traffic (about 0.8 million EAL-18 in one direction). The average spacing of transverse reflection cracks was approximately 25 (8 m) and 75 feet (23 m) for Designs B and C, respectively. As expected, no cracks were present in the westerly sections of Designs B and C, where the cement was omitted from the stone subbase. It is clear that given enough time the cracks will reflect through even 7-1/2 inches (188 mm) of bituminous concrete.

In the thin overlay sections (Design B) the reflective cracking seemed in 1971 to have proceeded beyond a mere visual nuisance. Several sections had developed longitudinal cracks which, combined with the transverse cracks, created slab-like portions of pavement. Slab action was apparent to the extent of horizontal shifting, faulting, and pumping. The sections seriously affected were fairly limited in extent and did not lead to a significant reduction in the service life of this design since the rate of progression was relatively slow. Hughes reported some construction difficulties in a nearby area where an unstable subgrade was undercut and backfilled.(3)

Possibly the most significant finding from the transverse cracking studies is the confirmation of an earlier Route 360 project finding; i.e., that cement treated stone subbases should not be used very close to the surface of bituminous pavements.(10)

Further studies of the project after some 12-14 years under traffic (approximately 2 million EAL-18) have shown that earlier indicated differences in performance have become more pronounced. Section A now has general rutting and extreme alligator cracking throughout. Surprisingly, a pattern of transverse cracking occurring at approximately 25-foot (8-m) intervals has developed. This finding was not expected because it had been thought that the 6-inch (150-mm) layer of crushed stone between the cement stabilized subgrade and the asphaltic concrete layers would prevent the reflection of shrinkage cracks.

Design B, with the cement treated aggregate base close to the surface, has now developed an intensive transverse cracking pattern with a spacing of 15 to 20 feet (5 to 6 m) in the traffic lanes. An example of severe transverse cracking on this design is illustrated in Figure 4. Occasional longitudinal cracking has also developed in these traffic lanes. However, the sections showing slab-like behavior earlier have not become significantly worse over the last 9 years. The areas of the passing lanes where the cement stabilization was omitted are still in good shape. It is apparent from this finding that from a performance standpoint it is feasible to omit such stabilization from the passing lanes. Portions of this design were resurfaced in 1976.

Design C, in which a cement treated aggregate base underlies 7 inches (175 mm) of asphaltic concrete, is still in good condition. Although it has a fully developed pattern of transverse cracking, no slab-like action has been detected. The occasional longitudinal cracking noted is to be expected under the prevailing condition.

Design D, which incorporates a modified full-depth concept (cement stabilized subgrade), is in good condition, having some longitudinal cracking in the outer wheel path of the traffic lanes. This design has performed markedly better than the other three. It is beginning to become somewhat rutted, probably due to densification of the thick bituminous layers.



Figure 4. Severe transverse cracking (sealed) on section B of Project 7.

Deflections

Deflections throughout the project have been generally satisfactory, but those for Design A have been significantly higher than those for the other three. The results of deflection tests also show that Design A (sandwich layer) has never developed more than approximately one-half the effective thickness index found for the other three designs. Note that the indicated index for Design A has, with seasonal variations, ranged from 6.0 to 8.0, while in the other cases indices in excess of 15 are usually indicated. It is probably for this reason that Design A appears now to have performed worse than the other three.

Utilizing methods mentioned earlier, the average effective thickness equivalency values of materials used in the construction of Project No. 7 were determined.⁽¹²⁾ These values are summarized in Table 3, where it is evident that the sandwich layer effect has been strong in the case of the 6-inch (150-mm) layer of select material in Design A. Its "a" value of -0.55 shows that the presence of the layer has been detrimental to the effective thickness index. Again, it is important to keep in mind that this layer may be preventing any overstressing of the subgrade soil.

Table 3
Thickness Equivalency Values

Project 7

<u>Layer</u>	<u>Thickness Equivalency (a)</u>
Asphaltic concrete	1*
Soil cement	0.60
Cement treated aggregate	0.98
Select material	-0.55

*Assumed value

General Appraisal

As a part of other studies, roughness tests have been conducted periodically on Project no. 7. The tests (Table 4) for 1966 were conducted with the BPR roughometer and those for 1977 and 1979 with the Mays Ridemeter.

Table 4
Roughness Results — Project 7

<u>Design</u>	<u>Roughness (in/mi)*</u>		
	<u>1966 — BPR</u>	<u>1977 — Mays</u>	<u>1979 — Mays</u>
A	56	95	99
B	53	129	143
C	54	74	83
D	54	78	80

*Metric conversion — 1 in/mi = 1.6 cm/km.

Since no reliable correlation has been established between the two measuring devices, levels of roughness from 1966 through 1979 cannot be compared. It is evident, however, that on a relative basis Designs A and B have become rougher at a faster rate than Designs C and D. Note that design B, with the thin asphaltic concrete over cement treated stone, gives by far the roughest ride due to the slab-like action mentioned earlier.

Based on the factors discussed above and recognizing that the project has served its expected design (before resurfacing) life of 8 to 10 years, it would have to be concluded that all four design types have performed adequately. However, when the four designs are contrasted with each other, they can be classified as

Design A - fair
Design B - fair
Design C - good
Design D - excellent.

It should be recalled that Design D, which shows excellent performance, is essentially a full-depth asphalt pavement on an improved subgrade. Its initial construction cost was somewhat more than that of the other designs and its performance is considerably better. Based on recent cost increases in the highway industry it is likely that the cost differential would be even more if the experimental project were built in 1979. On the other hand, it is also apparent that the full-depth design may serve for much longer than the accepted 8 to 10 years before an overlay is required. In the judgment of the author, 15 to 20 years of service can be expected from Design D before an overlay will be necessary.

Conclusions

The following conclusions appear to be warranted from experimental project No. 7.

1. Transverse shrinkage cracks reflect from a cement treated stone subbase through 3 inches (75 mm) of bituminous concrete in as little as 18 months and through 7 inches (175 mm) of bituminous concrete in less than 5 years.
2. Cement treatment of stone subbases can be omitted in passing lanes with no detriment to performance. (This may not be true with traffic volumes near capacity because of the change in distribution of truck usage as that point is approached.)
3. It has been reaffirmed that cement treated stone subbases should not be used in close proximity (3 inches or 75mm, for example) to a bituminous pavement surface under heavy traffic conditions.

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4. On the basis of over 14 years' exposure to heavy truck traffic, the pavement layers listed below give the indicated performance when overlaid by 7 inches (175 mm) of asphaltic concrete.
 - 4 inches (100 mm) of asphaltic concrete - excellent
 - 4 inches (100 mm) of cement treated aggregate base - good
 - 6 inches (150 mm) of untreated aggregate base - fair
5. The sandwich system wherein a weak layer is placed between two strong layers can have a detrimental effect on pavement deflections and on the effective thickness index.

ACKNOWLEDGEMENTS

The author gratefully acknowledges the excellent cooperation of the resident engineers and field maintenance personnel who have made essential contributions to the conduct of the study through their assistance in the collection of field data.

C. S. Hughes and Dr. N. K. Vaswani are acknowledged for their conduct of portions of the study and for their technical assistance in other portions. The interest and cooperation of R. W. Gunn and G. V. Leake in the collection and analysis of data are sincerely appreciated.

The work was conducted under the general direction of Jack H. Dillard, head, Virginia Highway & Transportation Research Council, and financed from HPR funds administered by the U. S. Federal Highway Administration.

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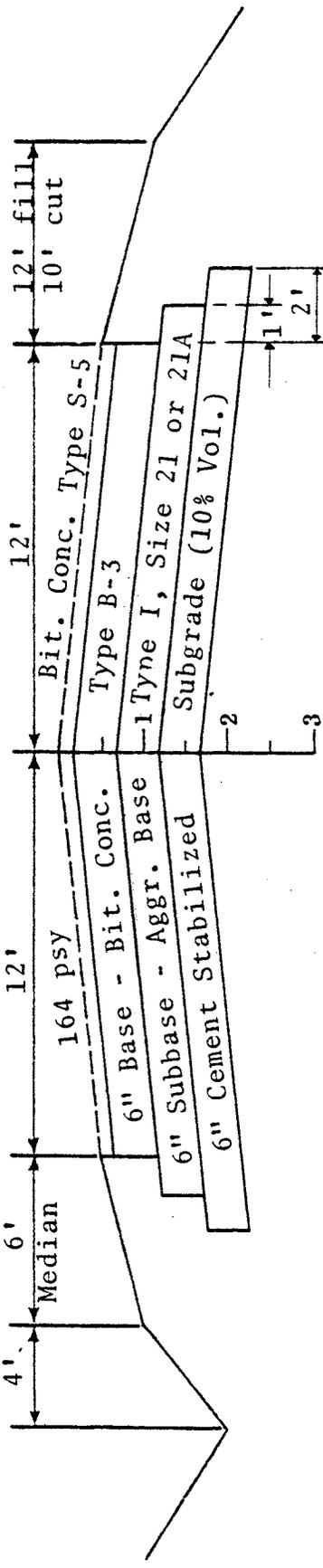
APPENDIX

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PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C-501 County: Campbell-Pittsylvania
6029-071-111, C-501 Length: 9.70 ml.
 From: 5.239 mi. N. Campbell-Pittsylvania C.L. Design Traffic (EAL-18): 134
 To: 4.651 mi. S. Campbell-Pittsylvania C.L. Design Thickness Index: 12.0
 Completion Date: April 1974

Study Project No.: 1-A



* Date	Dynalect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	1.587	126	45	--	0	0
CS Subgrade	1.423	283	49	4.6	0	0
Aggr. Base	1.076	305	53	5.7	0	0
B-3	0.790	163	60	9.6	0	0
S-5	0.389	82	67	15.0	0	0
3-6-74 4-8-74	0.355	47	69	15+	246	0
4-5-76	0.621	45	61	9.5	293	0
3-23-77	0.554	44	67	11.5	323	1
4-12-78	0.533	47	68	13.0	336	35
6-18-79	0.564	48	60	9.5	363 est.	46

Performance Rating: Fair

Cost per ml.: \$98,525

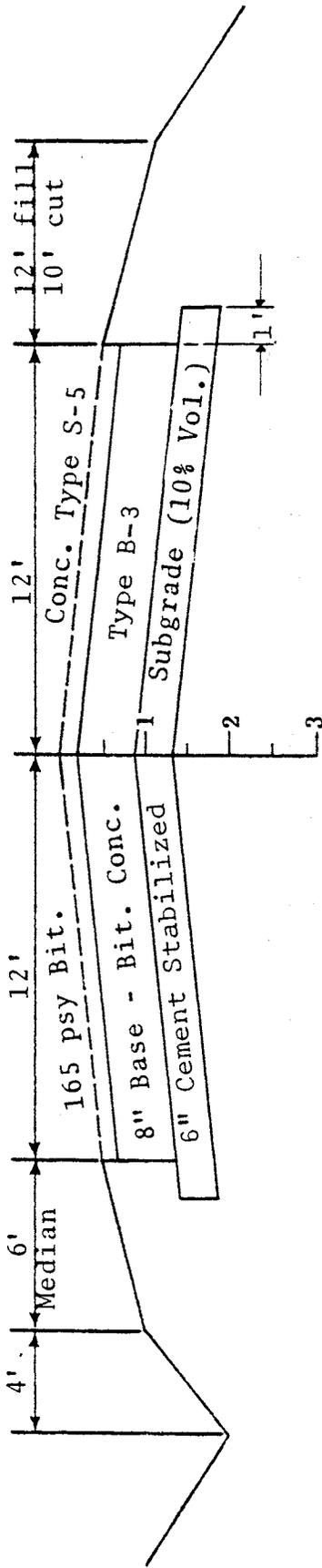
Remarks: Excellent condition 5-12-75, 4-5-76.
April 1977 small amount of alligator cracking in Campbell
Co. only. May 1979 general longitudinal and alligator
cracking in Campbell Co. Small amount of transverse
cracking in Campbell Co. only. Some longitudinal cracking

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in date column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C-501 County: Campbell-Pittsylvania
6029-071-111, C-501 Length: 9.70 mi.
 From: 5.239 mi. N. Campbell-Pittsylvania C.L. Design Traffic (EAL-18): 134
 To: 4.651 mi. S. Campbell-Pittsylvania C.L. Design Thickness Index: 11.9
 Completion Date: April 1974



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	2.092	78	42	-	0	0
C.S. Subgrade	2.089	261	48	3.7	0	0
B-3	0.639	301	70	12.0	0	0
S-5	0.603	85	73	14.0	0	0
4-8-74	0.353	46	83	15+	246	0
4-5-76	0.624	41	72	14.0	293	0
3-13-77	0.531	47	76	15+	323	13
4-12-78	0.555	45	76	15+	336	33
6-18-79	0.574	46	71	14.0	363 est.	49

Performance Rating: Good Cost per ml.: \$87,965

Remarks: Excellent condition 5-12-75. 4-5-76. Metric conversions:
April 1977 some longitudinal and transverse cracking in 1 mi = 1.6 km
Campbell Co. only. April 1978, longitudinal and transverse 1 ft = 0.3 m
Cracking in Campbell Co. May 1979, rare longitudinal cracking 1 in = 25 mm
in Pittsylvania Co., general longitudinal and alligator

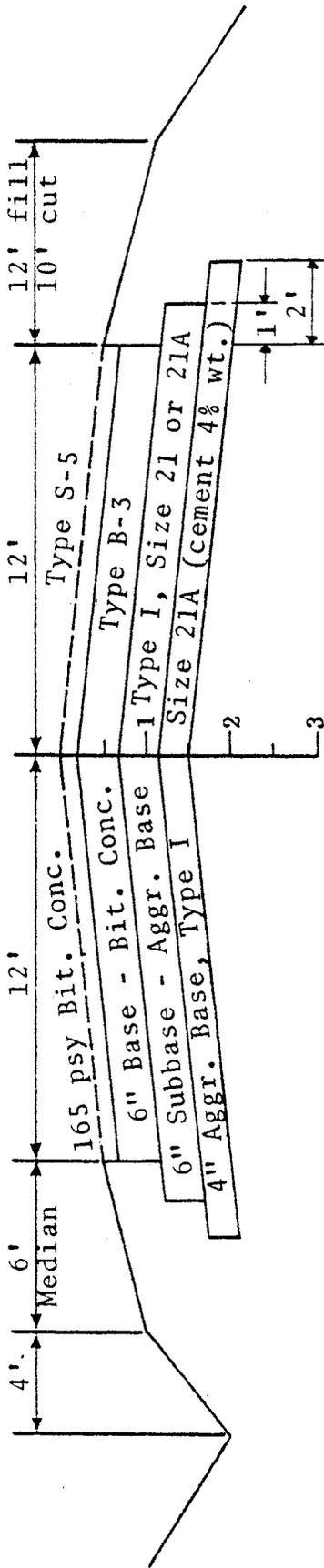
* Pavement elements listed in date column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C-501 County: Campbell-Pittsylvania
6029-071-111, C-501 Length: 9.70 ml.
 From: 5.239 mi. N. Campbell-Pittsylvania C.L. Design Traffic (EAL-18): 134
 To: 4.651 mi. S. Campbell-Pittsylvania C.L. Design Thickness Index: 12.0

Study Project No.: 1-C

Completion Date: April 1974



* Date	Dynaflect Deflection (l/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	2.737	104	38	--	0	0
CS Aggr. Base	2.533	168	43	2.8	0	0
Agg. Base	1.691	95	53	5.0	0	0
B-3	0.894	296	59	7.4	0	0
S-5	0.750	90	64	9.5	0	0
5-6-74	0.524	47	69	13.0	246	0
4-5-76	0.787	33	66	9.5	293	0
3-23-77	0.679	45	59	8.5	323	13
4-12-78	0.774	48	66	9.0	336	39
6-18-79	0.780	48	62	8.5	363 est.	42

Performance Rating: Fair

Cost per ml.: \$98,314

Remarks: Excellent condition 5-12-75. 4-5-76. April 1977, some longitudinal cracking in Campbell Co. only. May 1979, general longitudinal cracking and occasional alligator cracking in Campbell Co. Some longitudinal cracking in Pittsylvania Co.

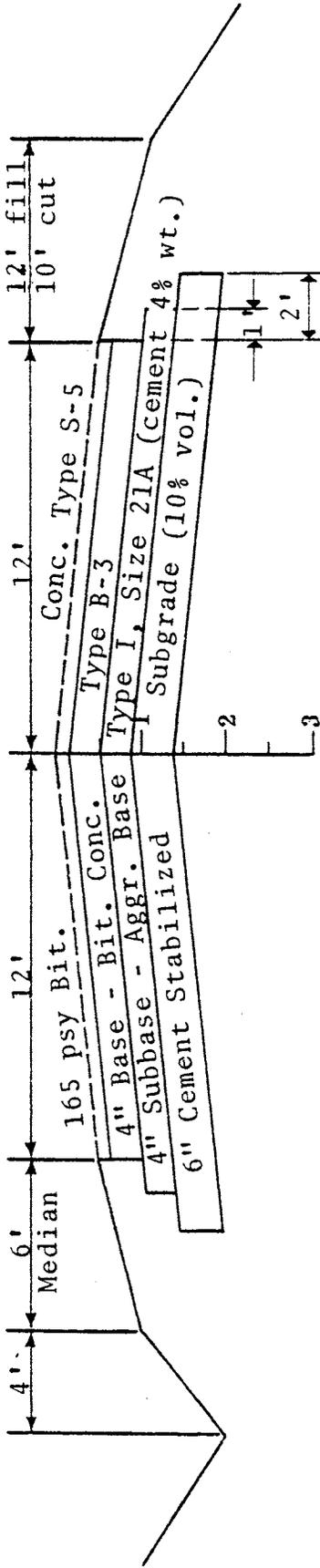
Metric conversions:

- 1 mi = 1.6 km
- 1 ft = 0.3 m
- 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C-501 County: Campbell-Pittsylvania
6029-071-111, C-501 Length: 9.70 mi.
 From: 5.239 mi. N. Campbell-Pittsylvania C.L. Design Traffic (EAL-18): 134
 To: 4.651 mi. S. Campbell-Pittsylvania C.L. Design Thickness Index: 11.9
 Completion Date: April 1974



* Date	Dynalect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	1.768	104	45	--	0	0
CS Subgrade	1.267	253	49	4.7	0	0
CS Agg Base	0.856	17	62	8.1	0	0
B-3	0.560	317	63	11.0	0	0
S-5	0.378	68	72	15+	0	0
3-26-74	0.343	43	75	15+	246	0
4-5-76	0.490	30	72	15.0	293	0
3-23-77	0.519	40	70	13.5	323	5
4-12-78	0.514	40	72	15.0	336	24
6-18-79	0.462	40	67	13.0	363 est.	33

Performance Rating: Fair Cost per mi.: \$82,526

Remarks: Excellent condition 5-12-75. 4-5-76. April 1977 some
longitudinal & transverse cracking in Campbell Co. only.
April 1978 numerous transverse cracks, Campbell Co. May
1979 occasional longitudinal cracking & general transverse
cracking in Campbell Co. Some longitudinal & transverse

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

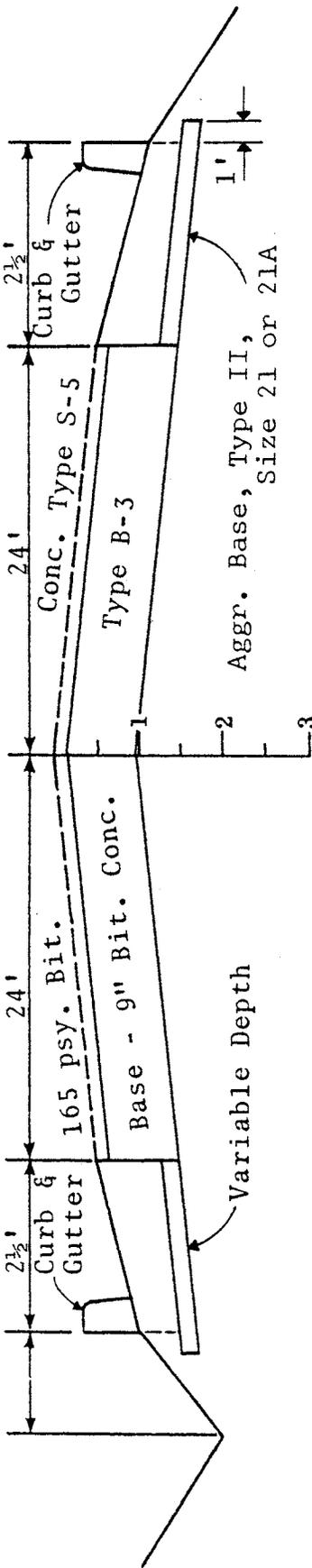
* Pavement elements listed in date column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 0031-137-102, C-501
 From: WCL Williamsburg
 To: 1.0 mi. E. WCL Williamsburg
 Completion Date: Fall 1970

County: City of Williamsburg
 Length: 1.0 mi.
 Design Traffic (EAL-18): 118
 Design Thickness Index: 10.0

Study Project No.: 2



* Date	Dynalect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
6-18-70	1.180	51	56	6.0	0	0
9-17-70	0.945	99	60	7.7	7	0
3-10-71	0.697	91	70	11.5	7	0
5-17-72	0.697	90	64	9.5	6	0
3-6-73	0.612	46	68	12.0	4	0
3-28-74	0.639	45	67	11.0	4	0
4-13-76	0.829	42	64	8.8	4	0
3-30-77	0.840	45	67	9.2	10	26
4-4-78	0.853	44	68	9.5	20	51
4-30-79	0.788	45	66	9.3	13 est.	57

Performance Rating: Excellent

Cost per mi.: \$136,910 (4 lanes)

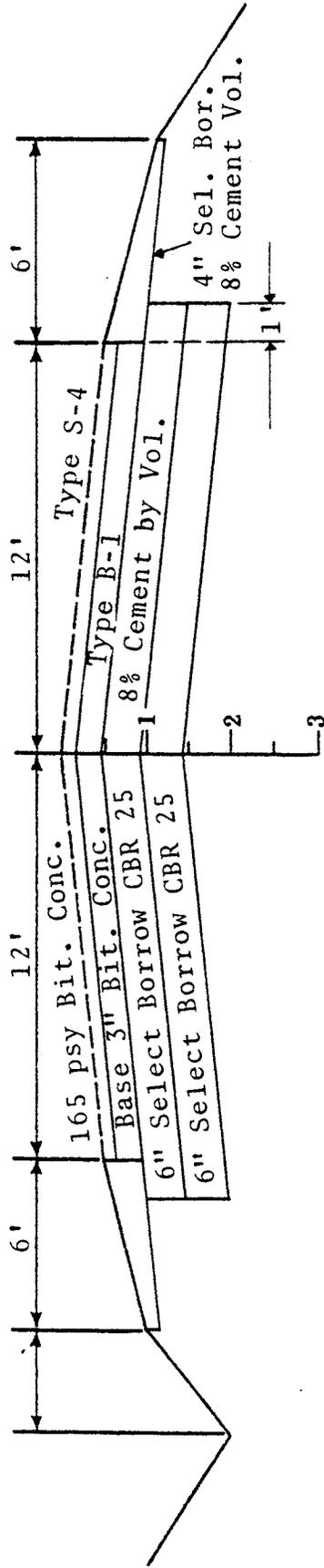
Remarks: May 13, 1975, poor cross section (built in). Longitudinal cracking on fill at north end of project. Excellent otherwise.
 March 1977 longitudinal cracking. Project still in good condition except for very rough ride. May 1979 occasional longitudinal cracking; general in fill section; very poor

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 0003-059-101, C-501 County: Middlesex
 0033-059-103, C-501 Length: 5.2 ml.
 From: 0.307 mi. E. Int. Rtes. 3 & 33 (Harmony) Design Traffic (EAL-18): 115
 To: 2.044 mi. E. Int. Rtes. 3 & 33 (Hartfield) Design Thickness Index: 11.7
 Study Project No.: 3-A
 Completion Date: March 1970



* Date	Dynalect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
8-21-70	0.555	37	61	9.5	21	0
8-31-71	0.661	37	60	9.0	24	0
5-11-72	0.456	38	66	13.0	22	0
3-27-73	0.452	35	65	12.5	30	0
3-27-74	0.430	37	67	13.5	27	0
4-12-76	0.560	37	62	10.0	25	0
3-9-77	0.656	38	60	8.6	24	5
3-21-78	0.491	36	67	12.1	32	8
3-8-79	0.600	40	56	8.0	30 est.	8

Performance Rating: Excellent Cost per ml.: \$48,154

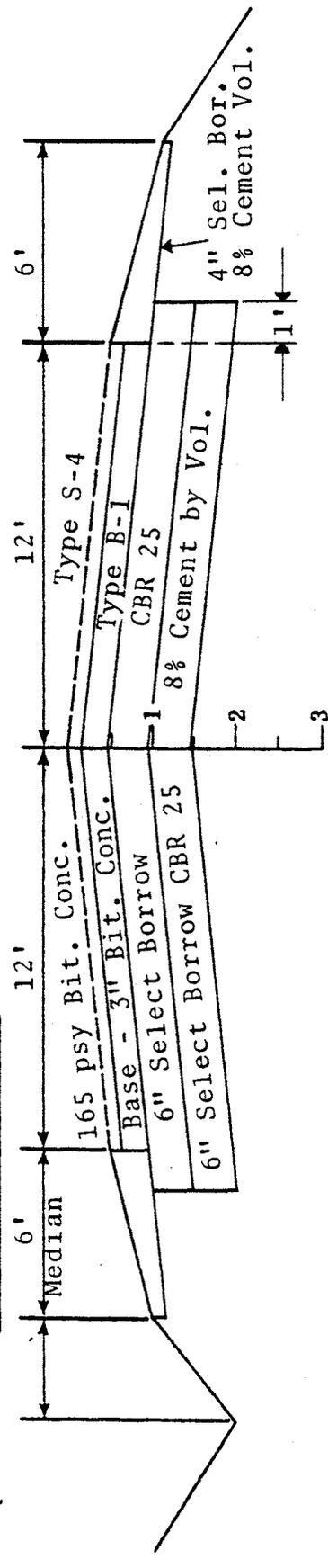
Remarks: First transverse cracking noted on 3-27-73. Occasional transverse cracking, not very pronounced, on 5-13-75. April 1976 transverse cracking at 3 per mile. March 1977, transverse cracking at 13 per mile, some longitudinal cracking. May 1979 transverse cracking at average of 24 per mile, trace of longitudinal cracking

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in date column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 0003-059-101, C-501 County: Middlesex
0033-059-103, C-501 Length: 5.2 ml.
 From: 0.307 mi. E. Int. Rtes. 3 & 33 (Harmony) Design Traffic (EAL-18): 115
 To: 2.044 mi. E. Int. Rtes. 3 & 33 (Hartfield) Design Thickness Index: 9.3
 Completion Date: March 1970



* Date	Dynaflect Deflection(1/1000 In.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
8-21-70	0.624	11	53	7.5	21	0
8-31-71	0.710	11	53	7.0	24	0
5-11-72	0.604	10	52	7.0	22	0
3-27-73	0.640	13	52	6.8	30	0
3-27-74	0.594	13	53	7.5	27	0
4-12-76	0.615	23	56	8.0	25	0
3-9-77	0.826	12	52	6.0	24	27
3-21-78	0.623	11	55	7.7	32	34
3-8-79	0.812	12	46	4.8	30 est.	43

Performance Rating: Excellent Cost per ml.: \$48,154

Remarks: May 13, 1975 -- No defects. April 1976 no transverse cracks. March 1977 transverse cracking 4 per mile, considerable longitudinal cracking. May 1979 transverse cracks at average of 11 per mile, longitudinal cracking.

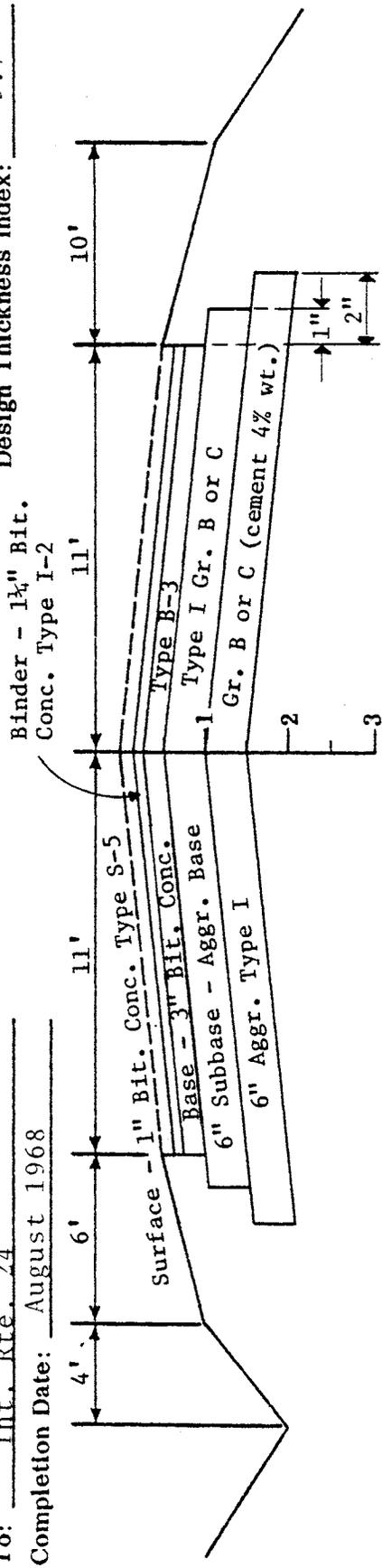
Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

0122

PERFORMANCE SUMMARY SHEET

Project: 0122-009-101, C-502 County: Bedford
 From: 0.3 mi. S. Int. Rte. 747 Length: 3.88 mi.
 To: Int. Rte. 24 Design Traffic (EAL-18): 25
 Completion Date: August 1968 Design Thickness Index: 9.7
 Study Project No.: 4



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
6-5-69	0.640	78	64	9.5	19	0
7-27-71	0.687	73	67	10.0	22	0
5-24-72	0.672	78	60	8.5	27	0
4-3-73	0.631	40	63	9.5	27	0
3-25-74	0.483	40	69	13.5	30	0
3-29-76	0.678	40	61	8.6	33	7
3-29-77	0.551	40	73	14.0	43	22
4-13-78	0.635	41	64	9.9	44	44
4-5-79	0.624	40	63	9.9	45 est.	75

Performance Rating: Excellent Cost per ml.: \$69,860

Remarks: May 12, 1975 - Few minor settlements & cracks due to instability of fills. Pavement in excellent condition.
March 1976 some longitudinal cracking, transverse cracking at 4 per mile. March 1977 considerable longitudinal cracking, transverse cracking at 4 per mile. May 1979 occasional transverse cracking, general longitudinal cracking with some alligator cracking.
 * Pavement elements listed in "date" column indicates pavement under construction.

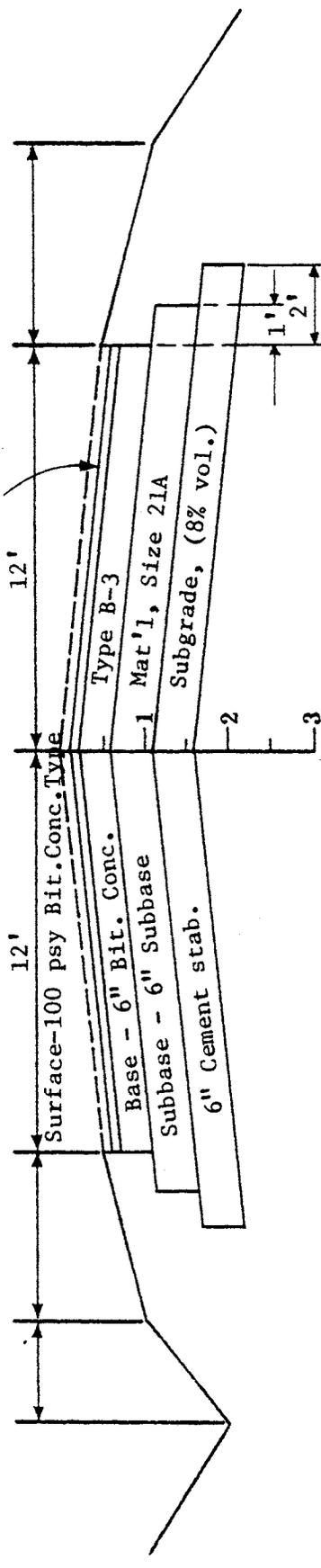
Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

PERFORMANCE SUMMARY SHEET

Project: 0207-016-102, C-501
 From: 0.34 mi. E. Int. Rte. 95
 To: 4.777 mi. E. Int. Rte. 95
 Completion Date: Nov. 1971

County: Caroline
 Length: 4.437 ml.
 Design Traffic (EAL-18): 919
 Design Thickness Index: 13

Binder-165 psy
 Bit. Conc. Type I-2



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
11-4-71	0.747	44	62	9.0	0	0
4-5-73	0.604	46	67	11.0	323	--
3-12-74	0.510	47	64	11.0	323	1.00
3-18-76	0.696	47	62	9.2	375	1.00
3-10-77	0.740	47	61	8.5	404	1.00
3-20-78	0.743	52	64	9.3	381	0
3-6-79	0.639	47	65	10.0	415 est.	0

Performance Rating: Poor Cost per ml.: \$78,552

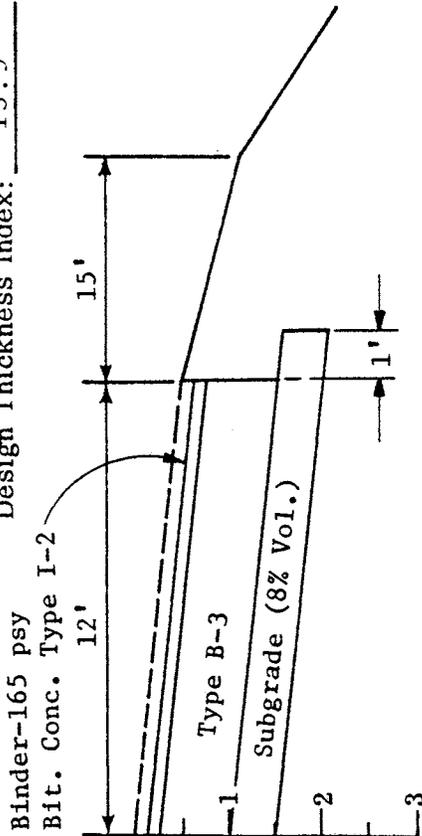
Remarks: May 13, 1975 - Extensive longitudinal cracking throughout in traffic lane. (Passing lane - okay). Occasional alligator cracking. Surface moisture present in some areas from rain the previous day. April 1976, severe alligator cracking. Project resurfaced in October 1977, 150 psy S-5. May 1979 project in good shape since resurface.

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 0207-016-102, C-502 County: Caroline
 Study Project No.: 6 Length: 4.8 mi.
 From: 0.22 mi. S. Int. Rte. 601 Design Traffic (EAL-18): 919
 To: Int. Rte. 619 Design Thickness Index: 13.9
 Completion Date: March 1974



* Date	Dynalect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	1.155	178	47	--	0	0
CS Subgrade	0.964	182	55	6.5	0	0
B-3	0.585	92	65	11.0	0	0
I-2	0.482	278	68	13.5	0	0
S-5	0.367	258	71	15+	0	0
3-12-74	0.260	46	80	15+	323	0
3-18-76	0.328	48	80	15+	375	3
3-10-77	0.361	46	78	15+	404	4
3-21-78	0.382	47	73	15+	381	9
3-6-79	0.323	48	77	15+	415 est.	18

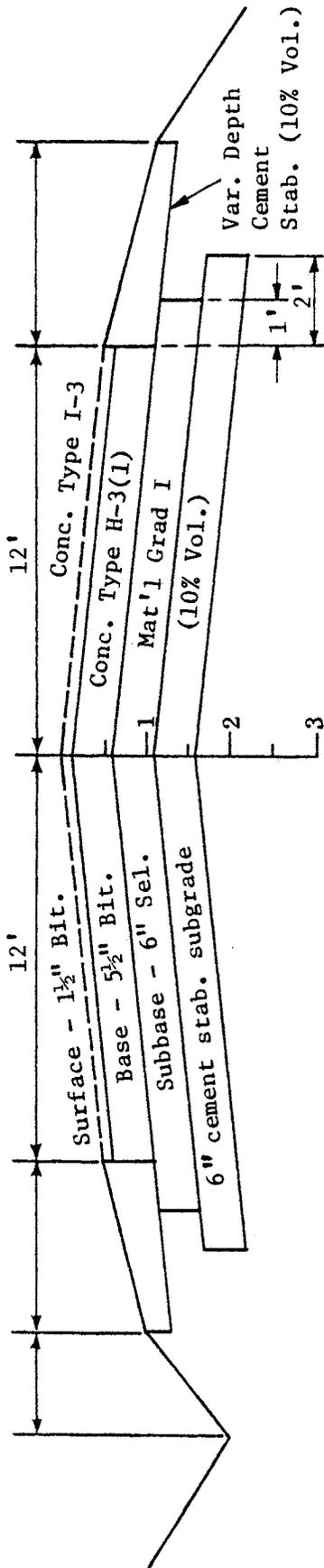
Performance Rating: Excellent Cost per mi.: \$89,707

Remarks: May 13, 1975 - Cracking and water seepage from one saturated area in passing lane. Otherwise in excellent condition. Metric conversions:
March 1976 a few longitudinal cracks. March 1977 a few 1 mi = 1.6 km
longitudinal cracks. May 1979 a few longitudinal cracks, 1 ft = 0.3 m
some patching in isolated "spring" area. 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C-501 County: Charlotte
 Study Project No.: 7A Length: 4.486 mi.
 From: 2.014 mi. S. Int. Rte 40 Design Traffic (EAL-18): 977
 To: 1.263 mi. W. Prince Edward C.L. Design Thickness Index: 9.4
 Completion Date: Dec. 1965



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
4-5-66	0.899	20	--	--	385	0
4-11-67	1.028	20	54	6.0	373	2
5-7-68	0.974	20	62	8.0	383	--
4-21-69	1.200	20	57	6.0	415	15
4-20-71	1.220	20	59	6.5	577	52
5-15-72	0.917	20	60	7.6	464	--
3-28-73	1.078	20	62	7.5	467	--
3-13-74	0.800	20	59	7.8	518	--
3-15-76	1.135	20	60	7.0	427	50
3-8-77	0.876	20	64	8.5	453	65

Performance Rating: Fair Cost per mi.: \$70,752

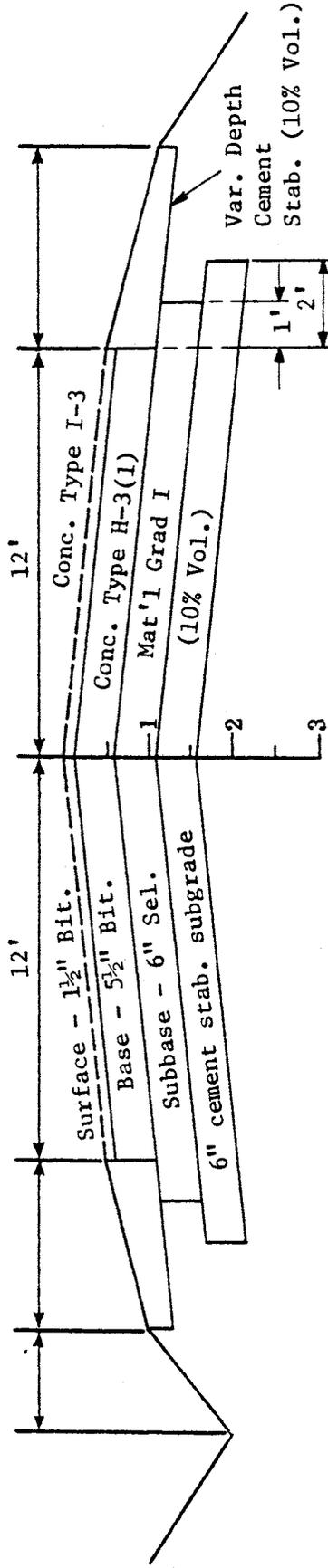
Remarks: _____

 Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: _____ County: _____
 From: _____ Length: _____ ml.
 To: _____ Design Traffic (EAL-18): _____
 Completion Date: _____ Design Thickness Index: _____
 Study Project No.: _____



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
3-8-78	0.934	20	64	8.5	444	69
5-15-79	1.182	20	58	6.8	490 est.	92

Performance Rating: Fair Cost per mi.: \$70,752

Remarks: May 12, 1975 - General rutting, longitudinal and alligator cracking Metric conversions:
throughout, frequent transverse cracking at about 25' intervals. 1 mi = 1.6 km
October 1977, some rutting is evident. May 1979 general 1 ft = 0.3 m
transverse, longitudinal, and alligator cracking. 1 in = 25 mm

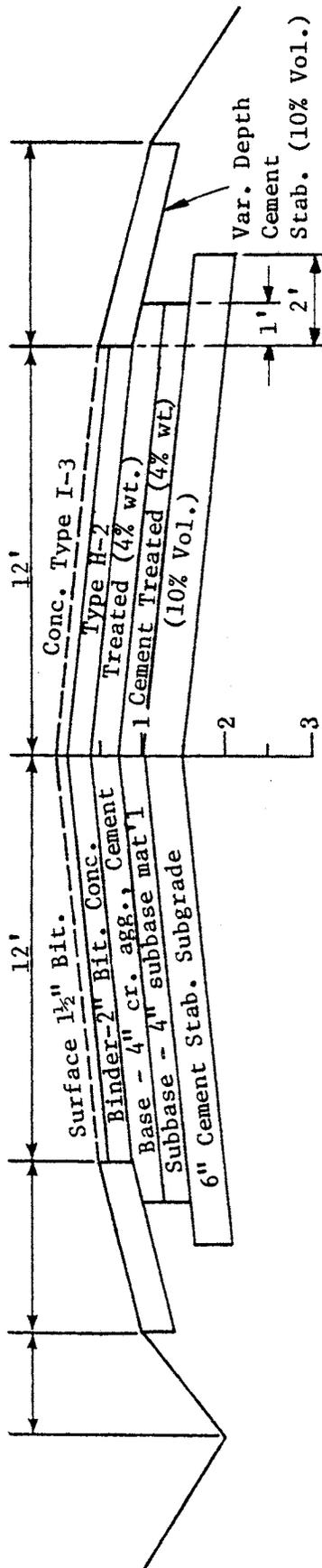
* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C-501
 From: 2.014 mi. S. Int. Rte. 40
 To: 1.263 mi. W. Prince Edward C.L.
 Completion Date: Dec. 1965

County: Charlotte
 Length: 4.486 mi.
 Design Traffic (EAL-18): 977
 Design Thickness Index: 13.9

Study Project No.: 7B



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
4-5-66	0.360	20			385	0
4-11-67	0.430	20	70	15+	373	2
5-6-68	0.419	20	77	15+	383	-
4-21-69	0.465	20	74	15+	415	37
4-20-71	0.485	20	78	15+	577	43
5-15-72	0.534	20	74		464	-
3-28-73	0.610	20	69	13.0	467	-
3-13-74	0.400	20	76	15+	518	-
4-11-67**	0.879	20	50	5.6	---	-
4-21-69**	1.056	20	47	4.5	---	-

Performance Rating: Fair

Cost per mi.: \$73,075

Remarks:

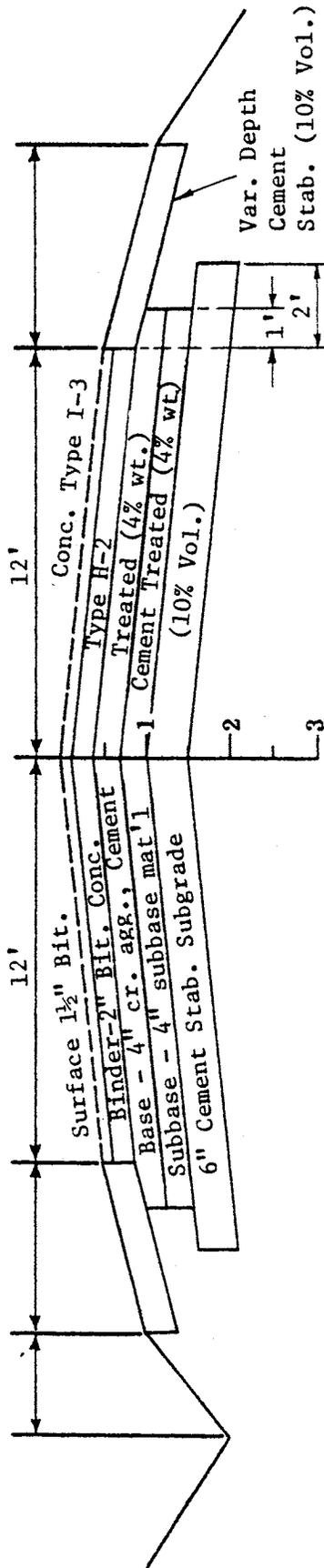
Metric conversions:

- 1 mi = 1.6 km
- 1 ft = 0.3 m
- 1 in = 25 mm

** Pavement elements listed in "date" column indicates pavement under construction (no cement in subbase layer of crushed aggregate).

PERFORMANCE SUMMARY SHEET

Project: _____ County: _____
 From: _____ Length: _____ ml.
 To: _____ Design Traffic (EAL-18): _____
 Completion Date: _____ Design Thickness Index: _____
 Study Project No.: _____



* Date	Dynaflect Deflection (1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
3-15-76	0.549	20	74	15.0	427	32
3-8-77	0.556	20	70	13.0	453	45
3-8-78	0.611	20	70	12.5	444	42
5-15-79	0.522	20	73	15.0	490 est.	83

Performance Rating: Fair Cost per ml.: \$73,075

Remarks: May 12, 1975 - Transverse cracking at 15'-20' intervals throughout traffic lanes. No cracking in passing lanes. Occasional longitudinal cracking in traffic lanes. Pumping location detected in 1971 seems to have gotten very little worse. This design partially overlaid in 1976.

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

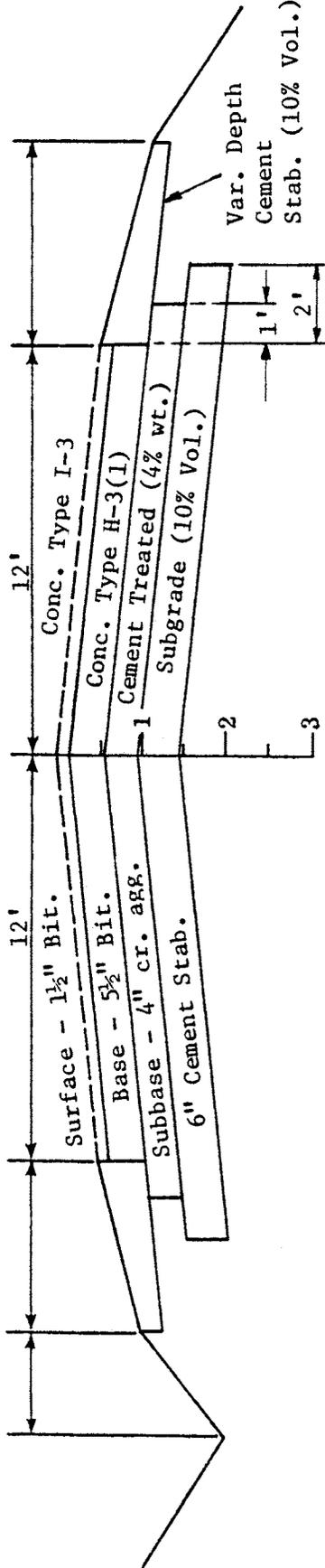
* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C-501
 From: 2.014 mi. S. Int. Rte. 40
 To: 1.263 mi. W. Prince Edward C.L.
 Completion Date: Dec. 1965

County: Charlotte
 Length: 4.486 mi.
 Design Traffic (EAL-18): 977
 Design Thickness Index: 13.4

Study Project No.: 7C



* Date	Dynalect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
4-5-66	0.432	20			385	
4-11-67	0.486	20	70	15+	373	0
5-6-68	0.436	20	79	15+	383	
4-21-69	0.608	20	77	15+	415	16
4-20-71	0.545	20	79	15+	577	22
5-15-72	0.387	20	78	15+	464	
3-28-73	0.497	20	82	15+	467	
3-13-74	0.367	20	80	15+	518	
4-11-67**	0.687	20	63	9.5		
4-21-69**	1.050	20	65	8.5		

Performance Rating: Good

Cost per mi.: \$72,230

Remarks: _____

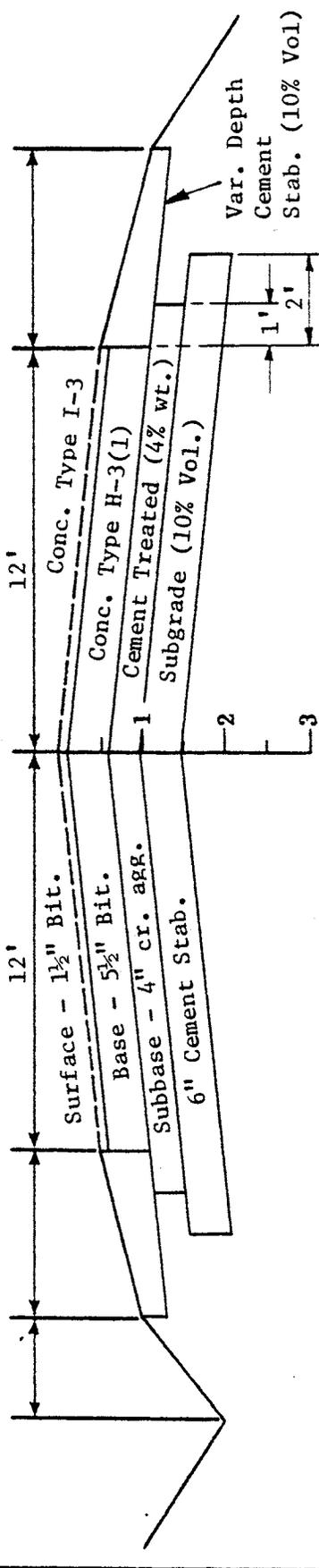
 Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

** Pavement elements listed in "date" column indicates pavement under construction.
 ** Passing lane (no cement in 4" layer of crushed aggregate).

0443

PERFORMANCE SUMMARY SHEET

Project: _____ County: _____
 From: _____ Length: _____ ml.
 To: _____ Design Traffic (EAL-18): _____
 Completion Date: _____ Design Thickness Index: _____
 Study Project No.: _____



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
3-15-76	0.535	20	79	15+	427	15
3-8-77	0.489	20	76	15+	453	35
3-8-78	0.548	20	76	15+	444	57
5-15-79	0.551	20	76	15+	490 est.	56

Performance Rating: Good Cost per mi.: \$72,230

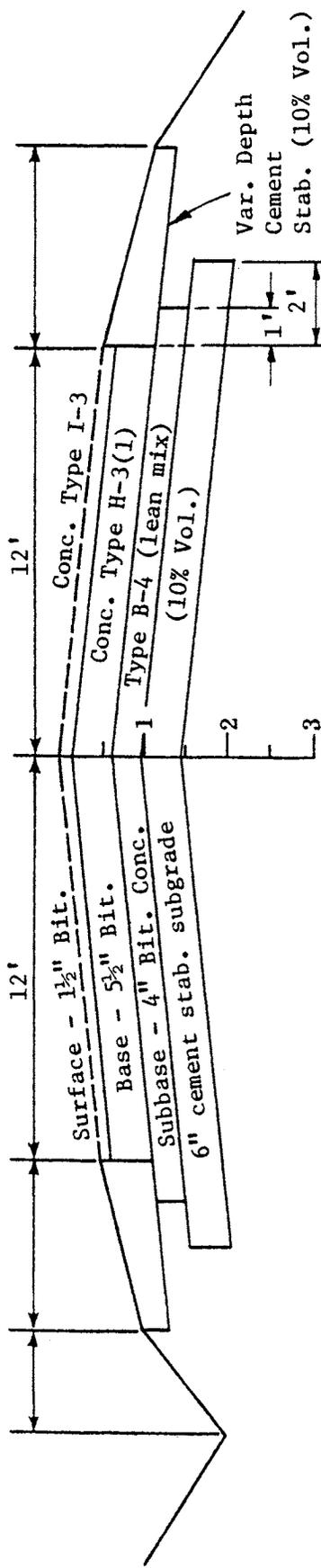
Remarks: May 12, 1975 - Transverse cracking throughout traffic lanes.
Occasional longitudinal cracking in traffic lanes. No cracking
in passing lanes. May 1979 general transverse cracking with
rare occurrences of alligator cracking.

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C-501 County: Charlotte
 From: 2.014 mi. S. Int. Rte. 40 Length: 4.486 mi.
 To: 1.263 mi. W. Prince Edward C.L. Design Traffic (EAL-18): 977
 Completion Date: Dec. 1965 Design Thickness Index: 13.4
 Study Project No.: 7-D



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
4-5-66	0.504	20	-	-	385	0
4-11-67	0.568	20	69	13.0	373	0
5-6-68	0.572	20	76	15+	383	-
4-21-69	0.635	20	75	15.0	415	11
4-20-71	0.687	20	75	14.5	577	21
5-15-72	0.483	20	80	15+	464	-
3-28-73	0.560	20	78	15+	467	-
3-13-74	0.400	20	80	15+	518	-
3-15-76	0.515	20	79	15+	427	23
3-15-77	0.558	20	74	15.0	453	34

Performance Rating: Excellent Cost per mi.: \$77,141

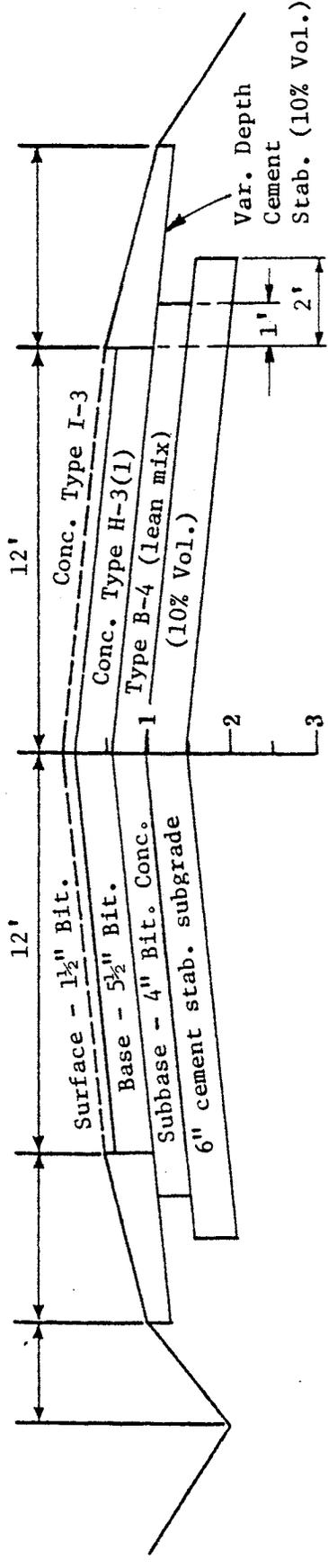
Remarks: _____

 Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.

PERFORMANCE SUMMARY SHEET

Project: _____ County: _____
 From: _____ Length: _____ ml.
 To: _____ Design Traffic (EAL-18): _____
 Completion Date: _____ Design Thickness Index: _____
 Study Project No.: _____



* Date	Dynalect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
3-8-78	0.576	20	74	15.0	444	51
5-15-79	0.614	20	71	13.5	490 est.	76

Performance Rating: Excellent Cost per mt.: \$77,141

Remarks: May 12, 1975 - Trace longitudinal cracking in outer wheel paths of traffic lanes. Excellent condition otherwise.
May 1979 - Rut depth of up to 1 1/2 inches. Rare transverse cracking and occasional longitudinal and alligator cracking.

Metric conversions:
 1 mi = 1.6 km
 1 ft = 0.3 m
 1 in = 25 mm

* Pavement elements listed in "date" column indicates pavement under construction.