EVALUATION OF EXPERIMENTAL FLEXIBLE PAVEMENTS

Interim Report No. 2

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 is 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 to this point 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.)

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

1. The Department is encouraged 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 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.

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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 wherein 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 begins when substantial portions of the subgrade for a given project have been prepared. At that time, dynaflect deflection tests are conducted on the subgrade. Similar tests follow the placement of subsequent pavement layers, including the final riding surface. Further steps in the evaluation of each project are 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, and
   C. visual defects such as cracking, rutting, patching, and settlements; and

4. compilation of records of major maintenance operations (bituminous concrete overlays, for example) and their costs.

Before a meaningful display of information can be presented, it is necessary to outline some of the more subtle features of the performance evaluations. The following discussion has particular reference to item 3 above.

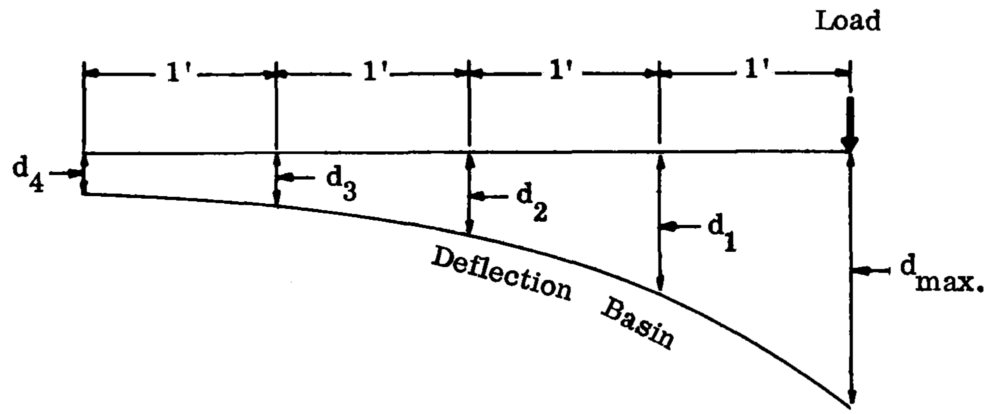
Traffic Characteristics

While Virginia's present design method utilizes the 18-kip (8165 kg) equivalency concept defined by AASHTO,[6] most of the pavements currently in the study were designed on the basis of traffic categories reflecting average daily trailer trucks and buses in both directions (T.T.&B.). Furthermore, T.T.&B. data are routinely collected by the Traffic and Safety Division while 18-kip (8165 kg) equivalence determinations are obtained only through weight studies and are too expensive for other than special requirements. Vaswani and Thacker, however, have developed correlation equations relating average truck weight, the T.T.&B. counts mentioned above, and 18-kip (8165 kg) equivalent axle loadings (EWL-18) on Virginia's highways.[7] Their equations are used to estimate the accumulated EWL-18 the experimental pavements have sustained during the study period.

Structural Capability

In this, as in many earlier reports, rebound deflections are used as an indication of the structural capabilities of the various flexible pavement systems.[1,2,3,4] While earlier studies reported deflection tests conducted with the Benkelman beam all recent work has been with the dynaflect.

This method provides for deflection measurements directly at the point of load application and at distances of 1, 2, 3, and 4 feet from that point. The plot of all five deflections defines the deflection basin as shown in Figure 1. Recent studies have shown that the shape of the deflection basin may be of more importance than the maximum deflection.[4] As a means of interpreting the shape of the basin a bending factor, or a "spreadability", has been defined and is also shown in Figure 1. This factor is the ratio of the average deflection to the maximum, expressed as a percentage. An increase in the factor indicates an ability of the pavement to spread the load over a wider area. Thus, a bending factor of 65 indicates a much stiffer pavement than does a factor of 45. The use of a bending factor in assessing flexible pavement performance has been discussed in an earlier report.[4]



$$\text{Spreadability} = \frac{d_0 + d_1 + d_2 + d_3 + d_4}{5\,d_0} \times 100$$

Figure 1. Dynaflect deflection basin.

Visual Defects

Periodic inspections of the study pavements have resulted in the accumulation of considerable data reflecting various kinds of physical defects, the most common of which is cracking. Other defects noted are rutting, patching, and settlements.

Rutting of flexible pavements, once fairly common in Virginia, seems to have been nearly eliminated over the past few years with the advent of cement and lime stabilization and the resultant more stable subgrades. Rutting is, thus, seldom a factor in performance surveys but is noted as to extent and frequency, as are patching and settlements.

To make the data on cracking more usable, a crack factor (CF) has been defined for flexible pavements and it is determined for each of the study projects at the time of each inspection.[(4)] To determine the factor, the project is separated into 1,000-ft. (305 m) sections and each section is surveyed for cracking. Each incidence of cracking has been arbitrarily assigned a value of 15 units and 20 units for longitudinal cracking and pattern or alligator cracking, respectively. The transverse cracking of flexible pavements is so often related to cement stabilization that its presence is not considered detrimental. Thus, a section with five incidences of pattern cracking would have a crack factor of 100. Similarly, two incidences of longitudinal cracking and one of pattern cracking yield a factor of 50. An upper limit of 100 units per 1,000-ft. (305 m) section is imposed on the data. After all sections within a project have been surveyed, the average crack factor is determined and designated as the factor for the project.

Clearly, the crack factor as used in this study is somewhat arbitrary and would not be adaptable to strict quantitative analysis. It is, however, the opinion of the researcher that the data are useful on a qualitative basis to determine whether or not a project is performing well. For example, other factors being equal, one can say that a crack factor of 5 for a 10-year old project clearly indicates better performance than, say, a crack factor of 50 for a 5-year old project.

PROJECT NO. 1

ALTAVISTA BYPASS — U. S. RTE. 29

The Altavista project is an all new location, 4-lane divided bypass of Altavista, Virginia, by U. S. Route 29. 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, the 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 predominately 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 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. While a more detailed discussion of these tests may be found in Interim Report No. 1, a summary is given below.

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 dynaflect deflection tests conducted on this section generally followed the expected pattern, i.e., deflections were reduced as each succeeding pavement layer was constructed. The results of these layer tests, however, were not as enlightening as had been hoped before construction of the experimental pavement began. While it had been expected that deflection tests on each successive layer would give an immediate indication of how much each layer contributed to the total pavement strength, the tests showed that such an early indication was not pratical. It is evident from the test results that time is an important factor in the development of the pavement's ultimate strength. While this is no doubt primarily due to the hydration and strength development time required for the cement treated subgrade, it is likely that variations in moisture content and increased consolidation of pavement layers as construction proceeds also are factors.

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 change in deflections or in thickness index were detected when the pavement was 1 year old.

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 bituminous concrete base course. Furthermore, since from 4 weeks to 10 months elapsed between tests on the cement stabilized subgrade and those on the bituminous concrete base, it is reasonable to assume that the stabilized subgrade had developed most of its strength before the bituminous concrete was applied and tested.[8]

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

At an age of 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 deflection 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]

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 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.

The gain in pavement strength with the addition of pavement layers had some of the characteristics seen for Design A. Note that again in Design C it appears, at first glance, that the 1 1/2-inch (38 mm) surface course contributed almost as much to pavement strength as the 6-inch (150 mm) bituminous concrete base. However, a study of the testing dates showed that the crushed stone and the bituminous concrete base layers were applied shortly after cement treatment of the 4-inch (100 mm) subbase layer.[8] Tests on the surface course were conducted several months later in most cases. Hence, it is likely that much of the increase in thickness index apparently due to the surface course was really due to strength gained by the cement treated subbase.

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. It remains to be seen whether this variation is a seasonal effect, as mentioned earlier, or a permanent increase in pavement strength.

Design D

Design D, which contains two cement stabilized 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.

The gain in pavement strength with time as noted for Designs A and C is again evident for Design D in that the surface course applied some months after the other pavement layers seemed to increase pavement strengths inordinately.

It is important to note here that while this particular design may perform extremely well, there will be 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 will be watched closely for any evidence of this phenomenon.

Cost Comparison

Direct cost comparisons of the standard and the three experimental pavement sections are readily available from the appended Figures A-1 through A-4 where contract bid prices have been used to compute actual construction costs. Note that sections A and C were the most costly and happened to cost the same. Section D, with two cement stabilized layers but a relatively thin asphaltic concrete course, was the least costly by some $16,000 per mile. It should be kept in mind that the bid prices given were effective in late 1971 and in no way reflect current construction costs. It is conservatively estimated that the Altavista pavement costs would be doubled if the contracts had been let in mid-1974. Furthermore, the relative costs of the four experimental sections may have changed because all highway materials have not increased in cost at the same rate.

Conclusions

The following conclusions are based on tests conducted during construction and shortly after 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.

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.

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 7 for the first 4 years the project has been open to traffic.[7]

Soil Condition

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 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/m^2).

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.[11]

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 3 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 1.0 is applicable to full-depth asphalt pavements within the depth limitations found in the current project. Virginia designers have sometimes used a 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 that close to the surface.

Only one minor problem was noted on the 5-year old full-depth pavement. This consisted of longitudinal cracking on a fill near the north end of the project and is attributed to fill settlement rather than to a pavement deficiency.

Conclusions

Based on 5 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 may not be justified when the total pavement thickness is no more than 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 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 5 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 an age of 3 years, but present no significant performance problem even after 5 years. No transverse cracking has been detected in the section containing the cushioning layer.

While the cushioning layer has served its purpose of preventing 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 Figure 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 cushioning 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 materials' location within the pavement structure.[5] Vaswani has investigated the effects of a sandwich layer more fully in another study.[13]

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 detriment to pavement performance for the first 5 years. While performance differences may develop later they are unlikely to do so 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 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 proviso, the following conclusions appear reasonable.

1. An unstabilized sandwich layer placed between a cement stabilized subbase and asphaltic concrete layers 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 has been approximated very closely by actual values of from 19-27 for the first 7 years the pavement has been in service.

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 is 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 7 years it has been in service show that it has performed extremely well. A few 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 9.1.

Thickness indices determined from deflection tests show that the design index has been realized in service. Measured indices range from 8.5 to 13.5 and reflect the seasonal variations mentioned on other projects.

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 shows no transverse cracking.

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 is effective in preventing reflection cracks.

2. The pavement develops 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 project, 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 should provide 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 A-2-4(0) 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 through its completion in July 1973.

The performance of Project No. 5, the standard design, has been highly disappointing in that after only 3 1/2 years of service the asphaltic concrete surface is extensively cracked longitudinally and has occasional areas of alligator or "map" cracking that are close to the pothole stage. The project is now close to a condition where an overlay will be required. This condition has developed after only about 425,000 load repetitions as opposed to an expected 2.5 million repetitions before an overlay would be necessary.

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 indicated index of 11.0 should, however, be 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 leads to speculation that a subsurface drainage problem exists. The "trench" construction and densely graded subbase material used on this project compare well with several similar projects where exploratory diggings have 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. The present project will be explored further before the final report of the study is written.

The full-depth asphalt pavement (Project 6) has performed extremely well for its first year 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 which was undectedted during construction. Several saturated areas which 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.

Conclusions

The extremely poor performance of the standard design and the relative newness of the full-depth project prohibit any definite conclusions at this time.

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 9 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 but has again increased to 518 in 1974.

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 3.1 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, is 19 inches (475 mm) thick. Thickness indices[9] are 11.5, 10.8, 13.4 and 13.4 for Designs A, B, C, and D respectively.

Project Objectives

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 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]

Recent studies of the project after some 9 years under traffic (approximately 1.3 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 the 6-inch (150 mm) layer of crushed stone between the cement stabilized subgrade and the asphaltic concrete layers was expected to prevent the reflection of shrinkage cracks. It is expected that exploratory diggings will be conducted on this section in the near future to determine whether or not the transverse cracks are indeed of the reflection type.

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 (5 m) to 20 feet (6 m) in the traffic lanes. 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 4 years. The areas of the passing lanes where the cement stabilization was omitted are still in excellent shape. It is apparent from this finding that, from a performance standpoint, it is feasible to omit such stabilization from the passing lanes.

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. 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 excellent condition, having only trace longitudinal cracking in the outer wheelpath of the traffic lanes. This design has performed markedly better than the other three.

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 these deflections tests also show that Design A 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.

General Appraisal

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.

These classifications have changed somewhat since 1971 when Designs A and B were classified as good and poor, respectively. The change in classification has come about because of the significant deterioration in Design A since the 1971 report while Design B has changed little in the same time period.

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 while 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 1975. 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 or more 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 75 mm for example) to a bituminous pavement surface under heavy traffic conditions.

4. On the basis of over 9 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

ACKNOWLEDGEMENTS

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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 and the late Dr. Tilton E. Shelburne, state highway research engineers. The study was financed from HPR funds in cooperation with the U. S. Federal Highway Administration.

REFERENCES

1. Nichols, F. P., Progress Report Number One, Experimental Flexible Pavements, Virginia Highway Research Council, October 1959.

2. ____________., Progress Report Number Three, Experimental Flexible Pavements, Virginia Highway Research Council, November 1963.

3. Hughes, C. S., Interim Report Number One, Phase C of Pavement Design and Performance Studies, "Experimental Flexible Pavements," Virginia Highway Research Council, February 1968.

4. McGhee, K. H., Final Report on Phase A, Performance Study of Typical Virginia Pavements, Virginia Highway Research Council, April 1972.

5. Vaswani, N. K., AASHO Road Test Findings Applied to Flexible Pavements in Virginia, Virginia Highway Research Council, April 1969.

6. Interim Guide for the Design of Flexible Pavement Structures, American Association of State Highway Officials, Washington, D. C., October 1961.

7. Vaswani, N. K., and D. E. Thacker, Estimation of 18-kip Equivalent on Primary and Interstate Road Systems in Virginia, Virginia Highway Research Council, May 1972.

8. McGhee, K. H., Evaluation of Experimental Flexible Pavements, Interim Report No. 1, Construction of Altavista Bypass Experimental Pavement, Virginia Highway Research Council, June 1974.

9. Vaswani, N. K., Evaluation of Pavement Design in Virginia Based on Layered Deflections, Subgrade and Its Moisture Content, Virginia Highway Research Council, May 1974.

10. McGhee, K. H., Final Report on Phase C, Experimental Flexible Pavements, Virginia Highway Research Council, May 1971.

11. Hughes, C. S., and G. W. Maupin, Jr., Installation Report — Thick Lift Bituminous Base, Construction and Materials Phase, Virginia Highway Research Council, November 1972.

12. Vaswani, N. K., "Method for Separately Evaluating the Structural Performance of Subgrade and Overlying Flexible Pavements," TRB Record No. 362, 1971.

13. ____________, "Optimum Structural Strength of Materials in Flexible Pavements," Highway Research Record 329, Highway Research Board, 1970.

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APPENDIX

PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C501
6029-071-111, C501

Study Project No.: 1-A

County: Campbell-Pittsylvania

Length: 9.70 mi.

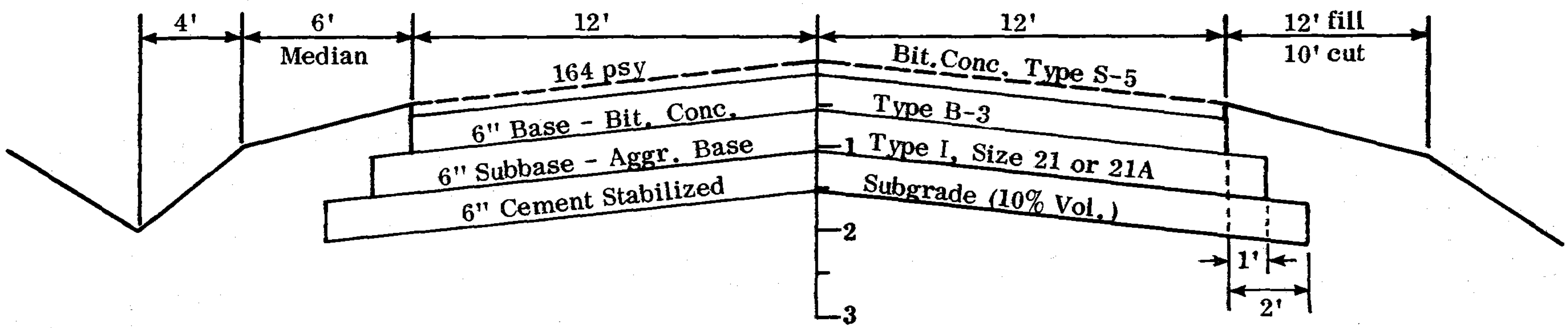
From: 5.239 mi. N. Campbell-Pittsylvania C.L.

Design Traffic (EAL-18): 134

To: 4.651 mi. S. Campbell-Pittsylvania C.L.

Design Thickness Index: 12.0

Completion Date: April 1974



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	1.587	126	45	—	0	0
C.S. Subgrade	1.423	283	49	4.6	0	0
Agg. 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
3-26-74 4-8-74	0.355	47	69	15+	246	0

Performance Rating: Excellent

Cost per mi.: $98,525

Remarks: Excellent condition 5-12-75.

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

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

PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C501
6029-071-111, C501

Study Project No.: 1-B

County: Campbell-Pittsylvania

Length: 9.70 mi.

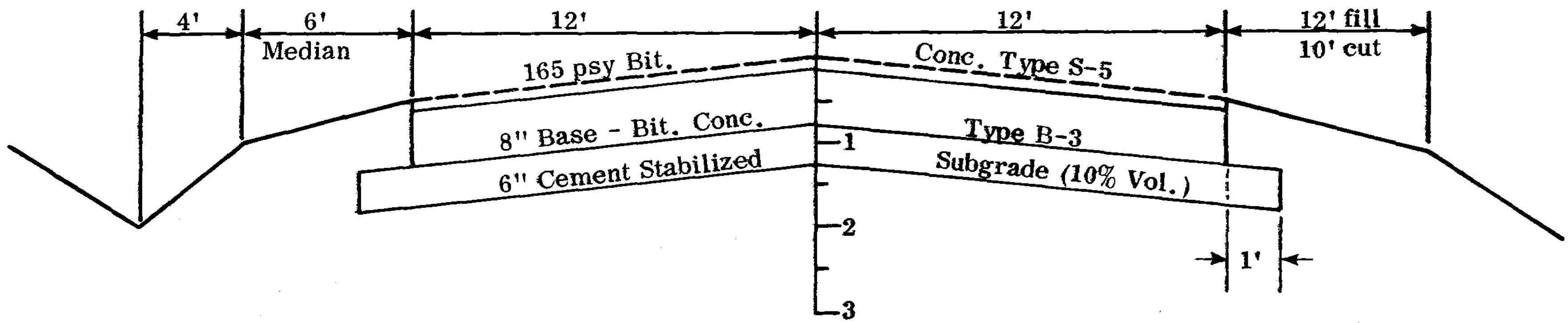
From: 5.239 mi. N. Campbell-Pittsylvania C. L.

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 3-26-74	0.603	85	73	14.0	0	0
4-8-74	0.353	46	83	15+	246	0

Performance Rating: Excellent

Cost per mi.: $87,965

Remarks: Execellent condition 5-12-75.

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

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

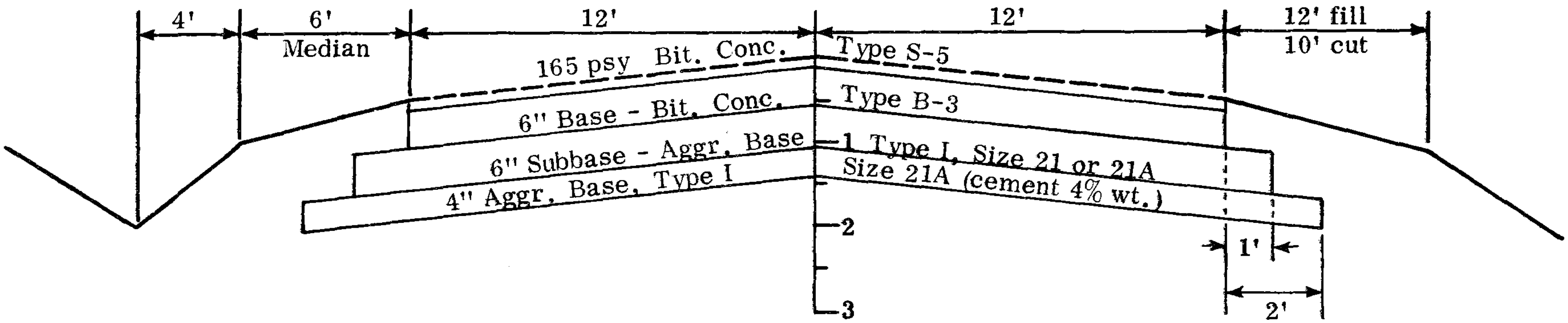
PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C501
6029-071-111, C501

Study Project No.: 1-C

County: Campbell-Pittsylvania
Length: 9.70 mi.
Design Traffic (EAL-18): 134
Design Thickness Index: 12.0

From: 5.239 mi. N. Campbell-Pittsylvania C. L.
To: 4.651 mi. S. Campbell-Pittsylvania C. L.
Completion Date: April 1974



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	2.737	104	38	—	0	0
C.S. Agg. 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
3-26-74 4-8-74	0.524	47	69	13.0	246	0

Performance Rating: Excellent

Cost per mi.: $98,314

Remarks: Excellent condition 5-12-75.

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

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

PERFORMANCE SUMMARY SHEET

Project: 6029-015-103, C501
6029-071-111, C501

Study Project No.: 1-D

County: Campbell-Pittsylvania

Length: 9.70 mi.

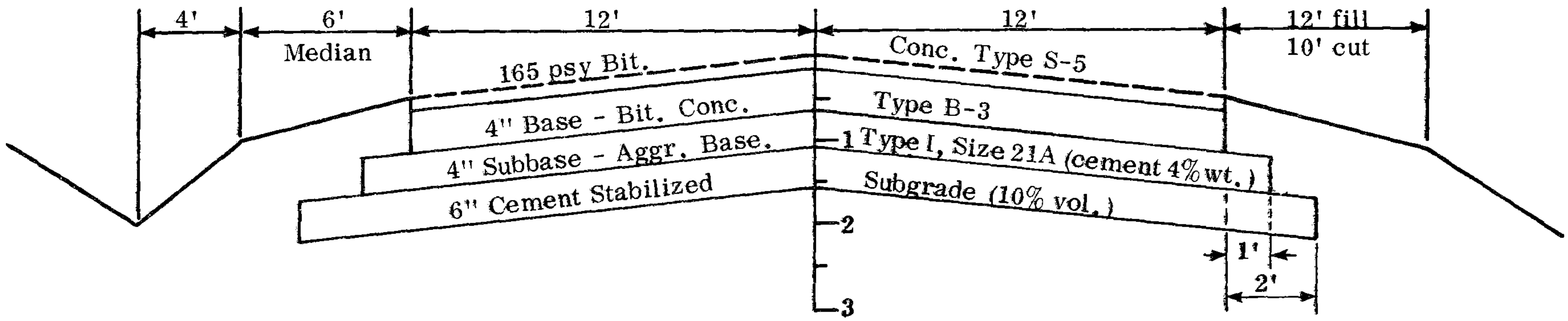
From: 5.239 mi. N. Campbell-Pittsylvania C. L.

To: 4.651 mi. S. Campbell-Pittsylvania C. L.

Design Traffic (EAL-18): 134

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	1.768	104	45	—	0	0
C.S. Subgrade	1.267	253	49	4.7	0	0
C.S. 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 4-9-74	0.343	43	75	15+	246	0

Performance Rating: Excellent

Cost per mi.: $82,526

Remarks: Excellent condition 5-12-75.

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

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

PERFORMANCE SUMMARY SHEET

Project: 0031-137-102, C501

From: WCL Williamsburg

To: 1.0 mi. E. WCL Williamsburg

Completion Date: Fall 1970

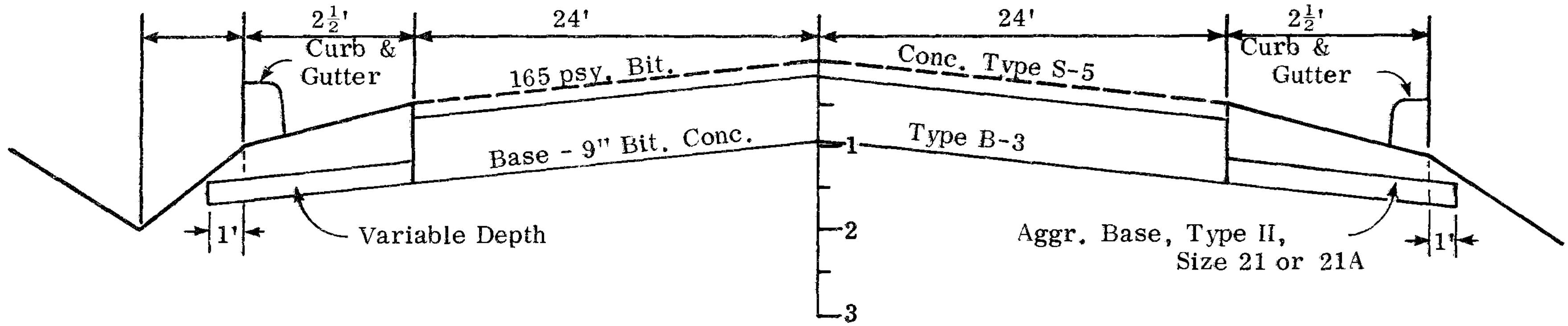
Study Project No.: 2

County: City of Williamsburg

Length: 1.0 mi.

Design Traffic (EAL-18): 118

Design Thickness Index: 10.5



* Date	Dynaflect 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

Performance Rating: Excellent

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

Remarks: May 13, 1975 Poor cross-section (build in). Longitudinal cracking on fill at north end of project. Excellent otherwise.

Metric conversions:

1 mi. = 1.6 km.

1 ft. = 0.3 m.

1 in. = 25 mm.

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

PERFORMANCE SUMMARY SHEET

Project: 0003-059-101, C501
0033-059-103, C501

Study Project No.: 3-A

From: 0.307 mi. E. Int. Rtes. 3 & 33 (Harmony)

To: 2.044 mi. E. Int. Rtes. 3 & 33 (Hartfield)

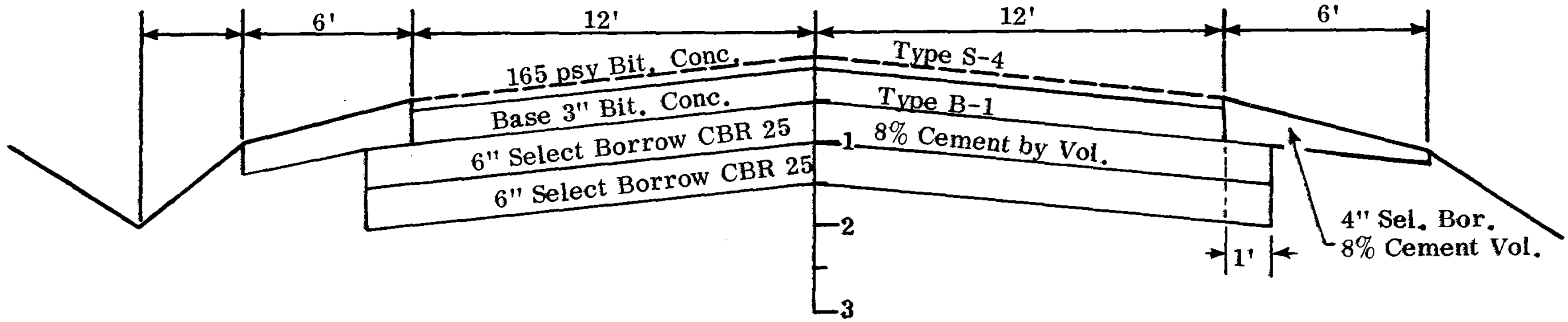
Completion Date: March 1970

County: Middlesex

Length: 5.2 mi.

Design Traffic (EAL-18): 115

Design Thickness Index: 11.7



* Date	Dynaflect 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

Performance Rating: Excellent

Cost per mi.: $48,154

Remarks: First transverse cracking noted on 3-27-73. Occasional transverse cracking, not very pronounced on 5-13-75.

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

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

PERFORMANCE SUMMARY SHEET

Project: 0003-059-101, C501
0033-059-103, C501

Study Project No.: 3-B

County: Middlesex

Length: 5.2 mi.

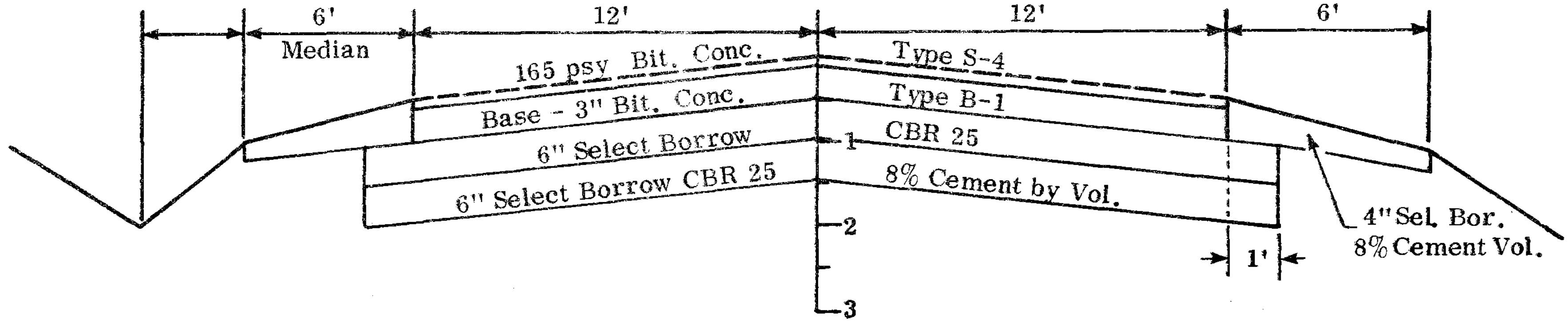
From: 0.307 mi. E. Int. Rtes. 3 & 33 (Harmony)

To: 2.044 mi. E. Int. Rtes. 3 & 33 (Hartfield)

Design Traffic (EAL-18): 115

Design Thickness Index: 9.0

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

Performance Rating: Excellent

Cost per mi.: $48,154

Remarks: May 13, 1975 – No defects.

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

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

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PERFORMANCE SUMMARY SHEET

Project: 0122-009-101, C502

From: 0.3 mi. S. Int. Rte. 747

To: Int. Rte. 24

Completion Date: August 1968

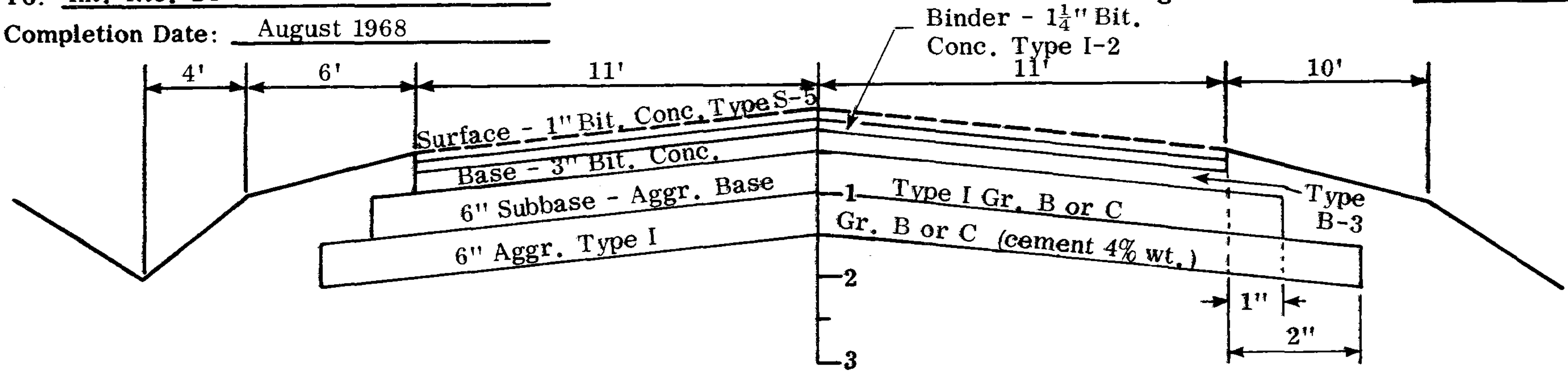
Study Project No.: 4

County: Bedford

Length: 3.88 mi.

Design Traffic (EAL-18): 25

Design Thickness Index: 9.1



* 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	78	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

Performance Rating: Excellent

Cost per mi.: $69,860

Remarks: May 12, 1975 - Few minor settlements and cracks due to instability of fills. Pavement in excellent condition.

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

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

PERFORMANCE SUMMARY SHEET

Project: 0207-016-102, C501

From: 0.34 mi. E. Int. Rte. 95

To: 4.777 mi. E. Int. Rte. 95

Completion Date: Nov. 1971

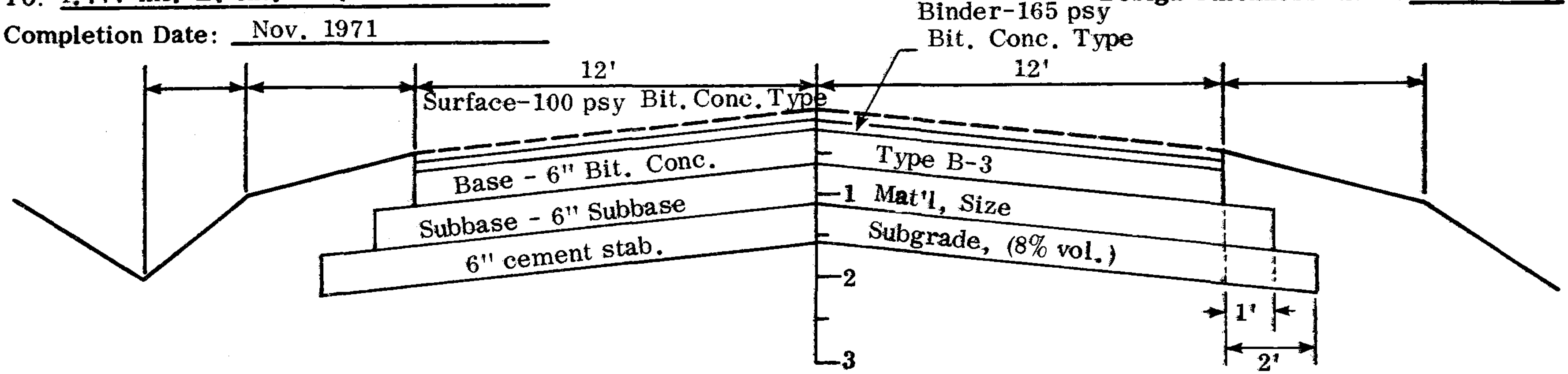
Study Project No.: 5

County: Caroline

Length: 4.437 mi.

Design Traffic (EAL-18): 919

Design Thickness Index: 13



* 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	100

Performance Rating: Poor

Cost per mi.: $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.

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

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

PERFORMANCE SUMMARY SHEET

Project: 0207-016-102, C502

From: 0.22 mi. S. Int. Rte. 601

To: Int. Rte. 619

Completion Date: March 1974

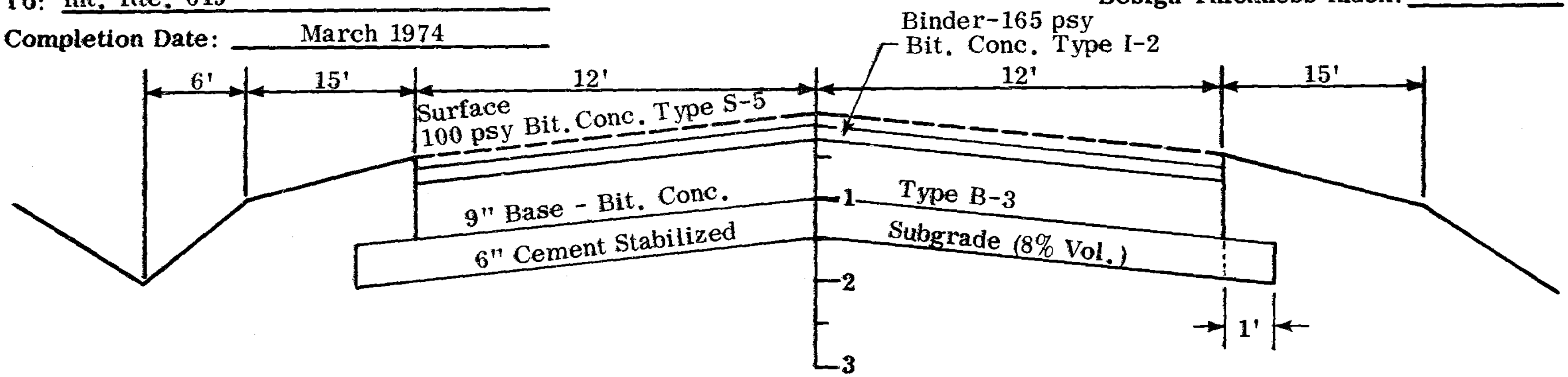
Study Project No.: 6

County: Caroline

Length: 4.8 mi.

Design Traffic (EAL-18): 919

Design Thickness Index: 15



* Date	Dynaflect Deflection(1/1000 in.)	No. Tests	Spreadability	Thickness Index	Avg. Daily EAL-18	Cracking Factor
Subgrade	1.155	178	47	—	0	0
C.S. 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

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

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

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C501

From: 2.014 mi. S. Int. Rte. 40

To: 1.263 mi. W. Prince Edward C.L.

Completion Date: Dec. 1965

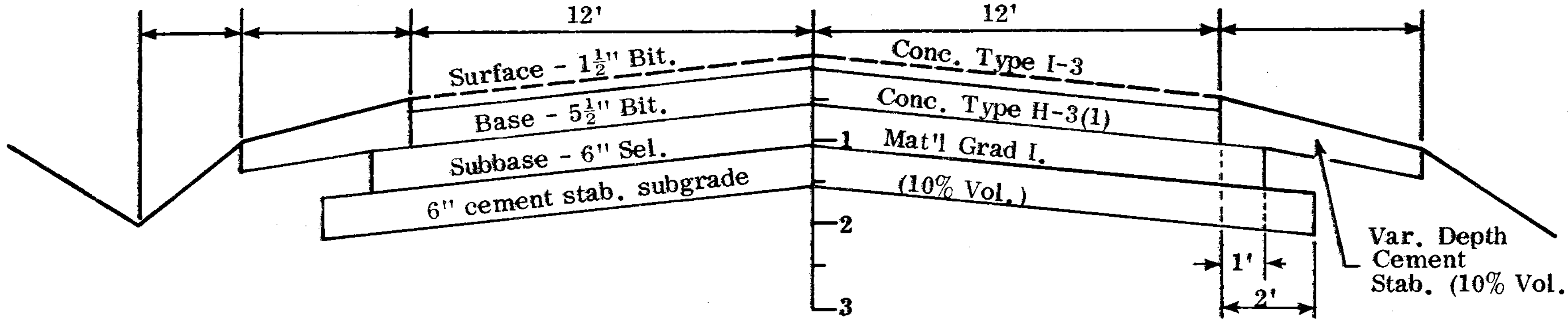
Study Project No.: 7A

County: Charlotte

Length: 4.486 mi.

Design Traffic (EAL-18): 977

Design Thickness Index: 11.5



* 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	—

Performance Rating: Fair

Cost per mi.: $70,752

Remarks: May 12, 1975 - General rutting, longitudinal and alligator cracking throughout, frequent tranverse cracking at about 25' intervals.

Metric conversions:

1 mi. = 1.6 km.

1 ft. = 0.3 m.

1 in. = 25 mm.

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

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C501

From: 2.014 mi. S. Int. Rte. 40

To: 1.263 mi. W. Prince Edward C.L.

Completion Date: Dec. 1965

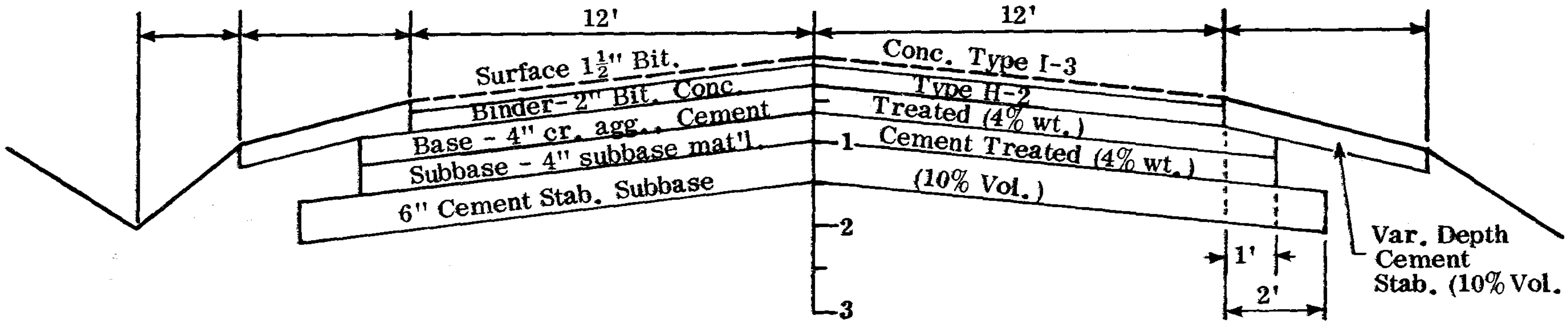
Study Project No.: 7B

County: Charlotte

Length: 4.486 mi.

Design Traffic (EAL-18): 977

Design Thickness Index: 13.4



* 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: 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.

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

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

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C501

From: 2.014 mi. S. Int. Rte. 40

To: 1.263 mi. W. Prince Edward C.L.

Completion Date: Dec. 1965

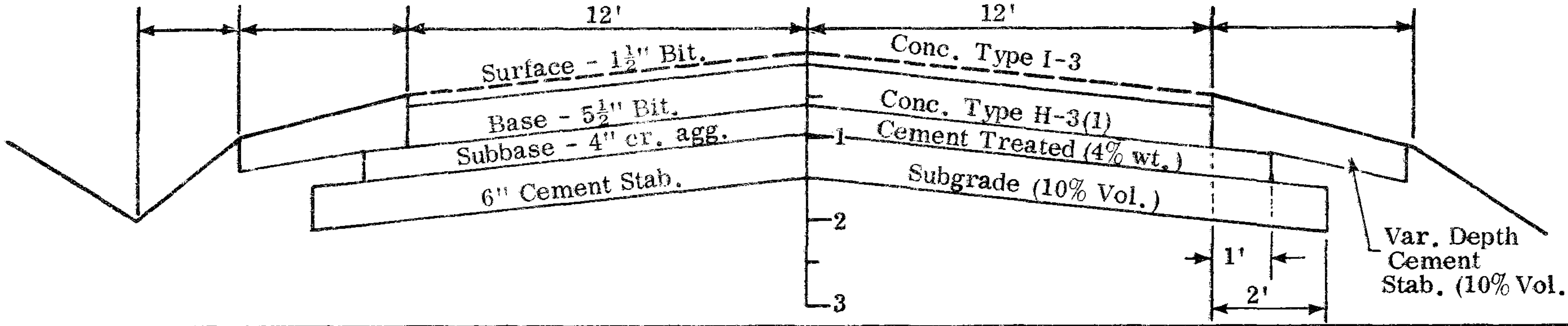
Study Project No.: 7C

County: Charlotte

Length: 4.486 mi.

Design Traffic (EAL-18): 977

Design Thickness Index: 13.4



* Date	Dynaflect 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: May 12, 1975 - Transverse cracking throughout traffic lanes. Occasional longitudinal cracking in traffic lanes. No cracking in passing lanes.

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

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

PERFORMANCE SUMMARY SHEET

Project: 7360-019-102, C501

From: 2.014 mi. S. Int. Rte. 40

To: 1.263 mi. W. Prince Edward C.L.

Completion Date: Dec. 1965

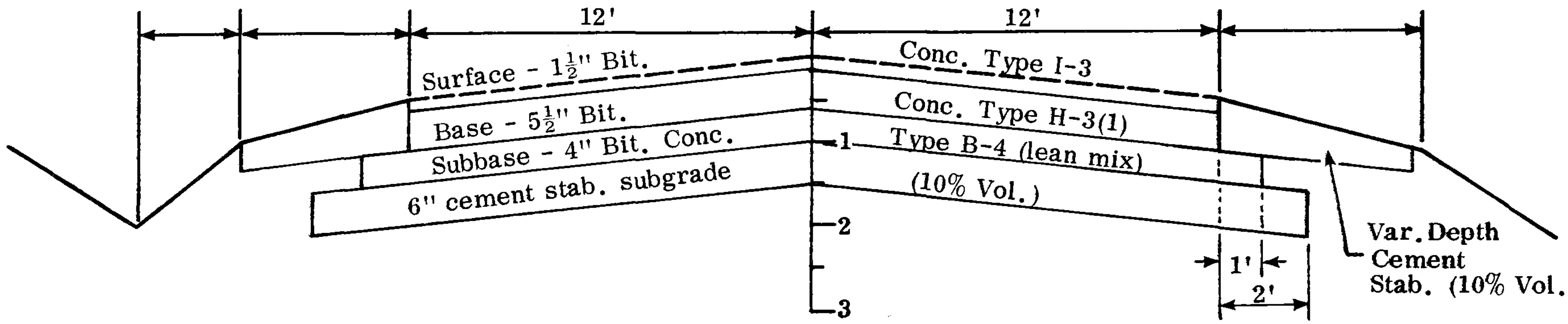
Study Project No.: 7-D

County: Charlotte

Length: 4.486 mi.

Design Traffic (EAL-18): 977

Design Thickness Index: 13.4



* 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	—

Performance Rating: Excellent

Cost per mi.: $77,141

Remarks: May 12, 1975 - Trace longitudinal cracking in outer wheel paths of traffic lanes. Excellent condition otherwise.

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

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