Past Performance of Composite Pavements

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This report reviews some of the existing experimental composite pavements. In particular, the design features and performance over the first three years of such a pavement at Milton, Ont. are described.

•ONE of the main objectives of the Composite Pavement Design Committee is to encourage the construction of experimental composite pavements so that performance may be studied in relation to design, construction, traffic and other factors. As a prelude, it may be useful to record in brief some of the facts about such pavements that have already been constructed.

One composite pavement, purposely constructed as an experiment as distinct from projects involving the resurfacing of an existing pavement with a dissimilar material, is located at Milton, Ont., some 25 miles west of Toronto. This pavement is some two miles long in the westbound lanes of highway 401, which is a major 4-lane controlled-access highway. The pavement was constructed in 1959 and a little over three years performance is available for comment. In planning this experiment, notice was taken of many significant composite pavement studies in Europe, many of them in England, and an outline of these is given for information.

PURPOSE AND DESIGN FEATURES OF THE ONTARIO PAVEMENT

Before the introduction of load transfer devices at the joints and new types of finishing machines, difficulties had been experienced in producing a concrete pavement that would give a smooth ride for many years under heavy traffic. A suggested solution to this problem was to combine the ease of producing a smooth ride with asphalt with the load-carrying capacity of a rigid base. A design study was made, and the gaps in knowledge of the behavior of concrete bases surfaced with asphalt at once became obvious. A survey of practice as related to resurfacing existing pavements highlighted the problem of reflection cracking and a study of European practice, in the absence of real North American experience other than for city street work, showed no unanimity as to the desirable design features for composite pavements. It was decided to investigate the following:

1. Could a smooth-riding pavement be more easily built by surfacing a concrete base with asphalt rather than just concrete?
2. The best combinations of thicknesses of concrete base and asphalt top for a high-class type of pavement designed to carry heavy traffic.
3. Should the concrete base be reinforced or not?
4. How could reflection cracking be prevented or cut down?
5. The effect of temperature on the expansion, warping, of the base concrete due to the presence of a black surface and the insulating effect of the asphalt surfacing.
6. Longitudinal cracking along centerline joint.

With these factors in mind, it was decided to split the 2-mi length designated for the experimental pavement into 7 different designs of concrete base surfaced with asphalt. By way of comparison, control sections of conventional jointed and continu-

TRAFFIC VOLUME	1960	1961	1962
A.A.D.T.	10,000	10,500	15,500
% COMMERCIAL VEHICLES	27	22	25

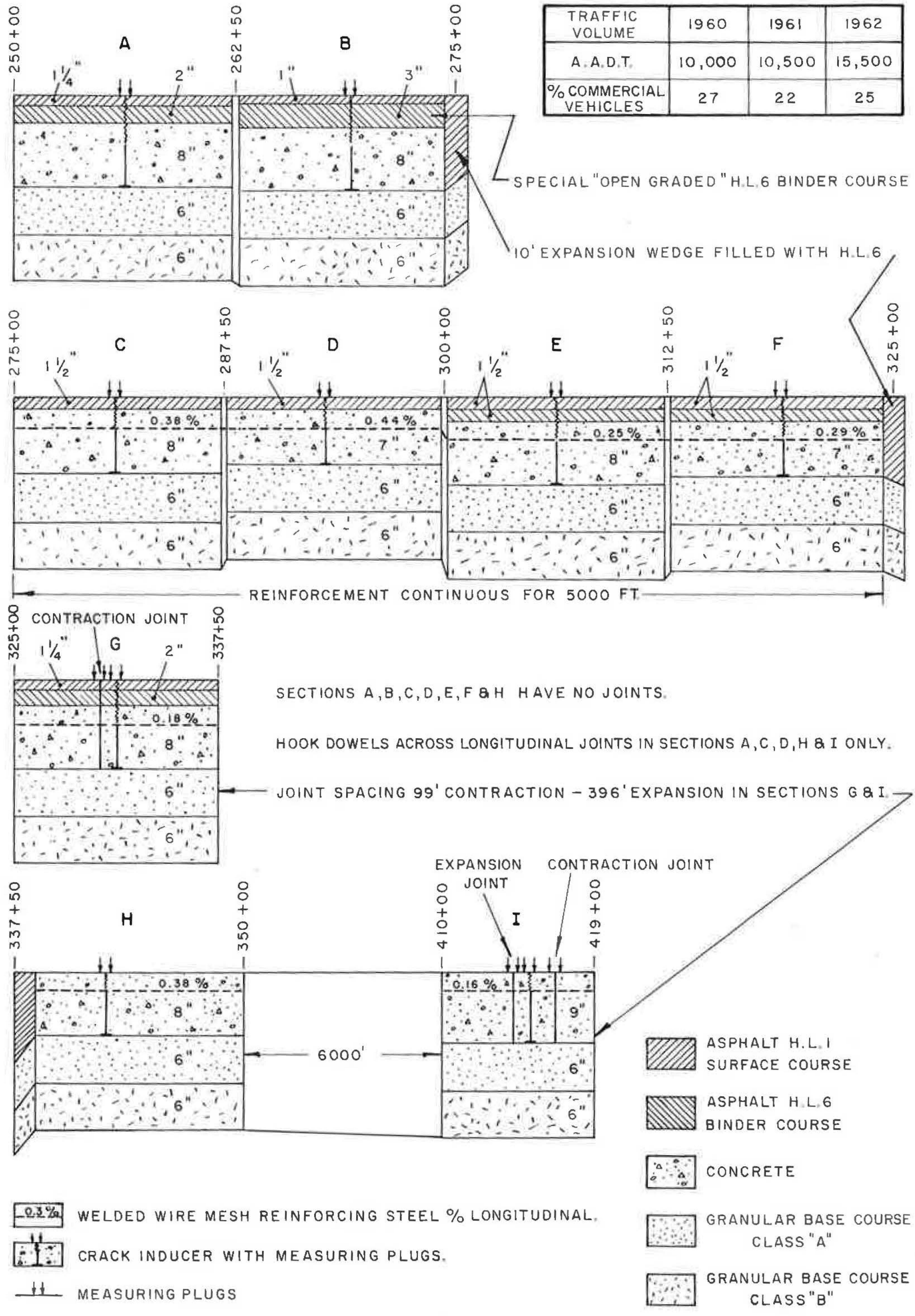

Figure 1. Design features of different sections, experimental composite pavement.

ously reinforced concrete pavement were included (Fig. 1). Under each section a uniform thickness of 12-in. of granular material placed in two lifts was called for. The subgrade throughout was a uniform clay till (liquid limit, 29 percent; plasticity index, 11 percent) essentially not susceptible to frost heaving. The grade itself was almost level and was in a shallow cut section for all of the test area.

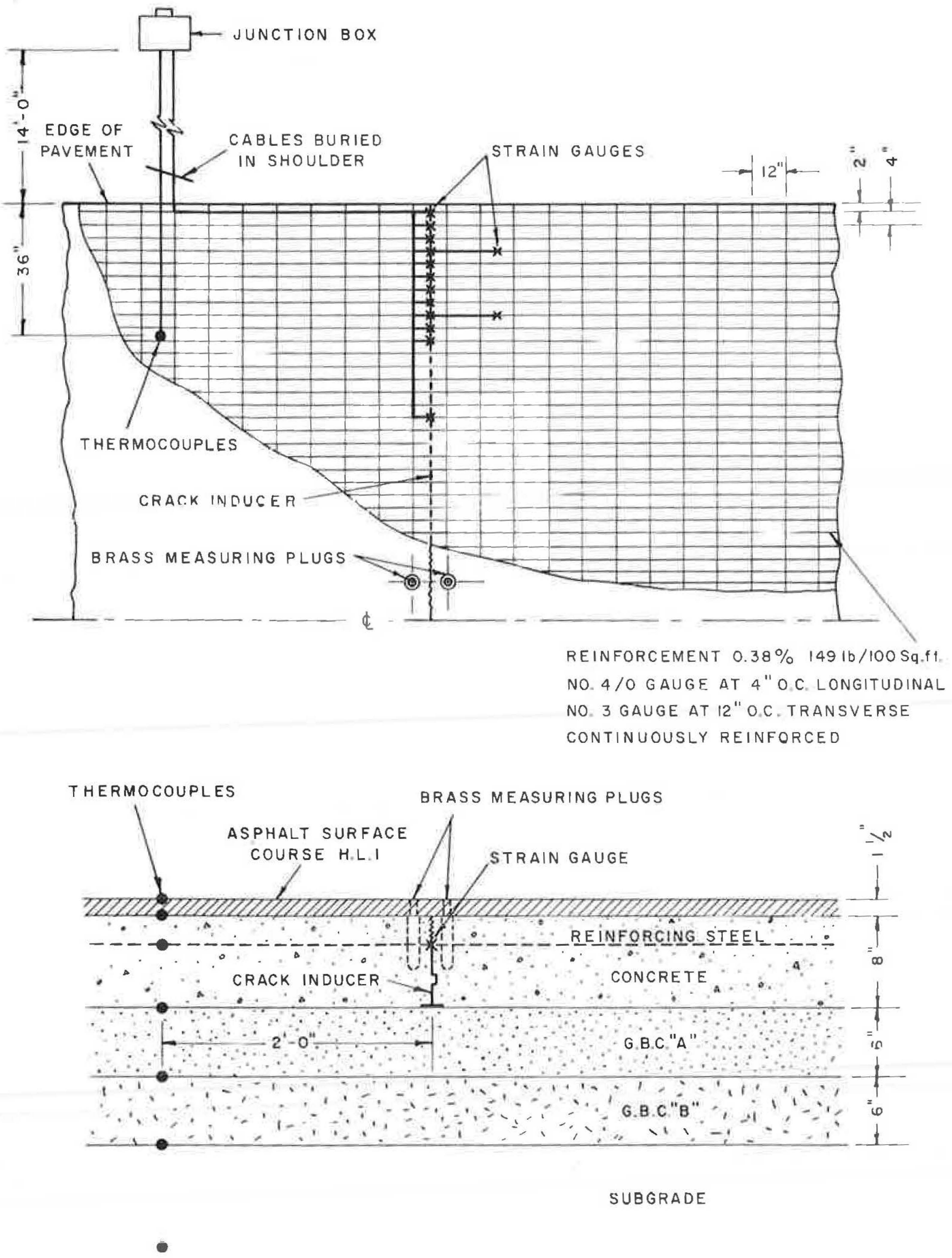

Figure 2. Instrumentation, test section C.

Figure 3. Welded-wire reinforcing steel mesh with SR-4 strain gages bonded to longitudinal steel being installed.

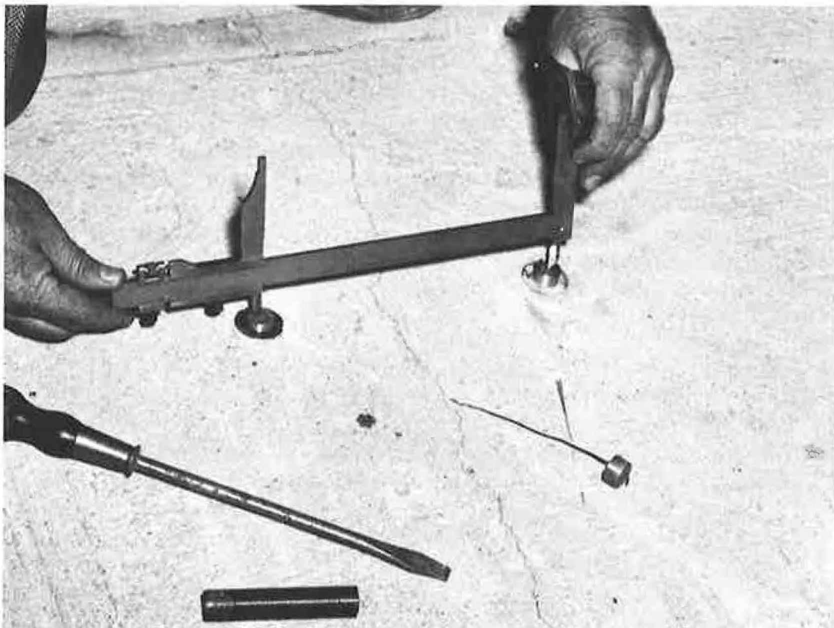

Figure 4. Width of induced crack being measured at brass plug gage points with vernier calipers.

PROPOSED OBSERVATIONS AND INSTRUMENTATION

Although the real proof as to which of the different sections would perform best would only be obtained by a long-time study of performance as shown by such obvious features as extent and width of cracking, it was thought that as full a set of observations as was possible should be made both during and after construction. It was decided to follow the outline suggested in Highway Research Board Circular 372 (1) for the minimum level of basic observations necessary for an experimental pavement. It was also decided to undertake special studies concerned with temperature effects caused by the presence of the asphalt surfacing and concerned with the stresses in reinforcing steel across cracks. Thermocouples were accordingly installed in each test section at various levels from subgrade up to pavement surface. Where reinforcing steel was used SR-4 strain gages were bonded to the longitudinal bars so as to cover the full width of the outside wheelpath, together with gages for temperature and no-load corrections. At each observation station, crack inducers were placed in the concrete base and brass measuring plugs installed. Extension pieces could later be screwed into the plugs to bring them up to the level of the asphalt surface. A typical instrumentation installation is shown in Figures 2, 3, and 4. All wires from thermocouples and strain gages were brought out in a common cable to a junction box clear of the shoulder.

CONSTRUCTION FEATURES AND QUALITY CONTROL

The whole construction operation was treated as a normal contract. Although the contractor was alerted by special provisions to the experimental nature of the work, he was not required to produce work controlled to closer limits of quality than applied to the rest of the contract. Quality control testing during construction was, however, much more extensive than usual in order to determine factors which might subsequently prove relevant to performance.

Initial grading operations were carried out in 1958. All the granular material was placed in the summer of 1959. Concrete was laid late in August and early in September 1959. After standing open for some 6 to 8 weeks, the concrete base sections were surfaced with asphalt.

The compaction of the previously constructed subgrade was checked before placing the 6-in. subbase of granular material (2-in. maximum size, 5 percent passing No. 270 screen). When this in turn had been compacted, it was covered with a 6-in. base of a well-graded granular material. The base itself was compacted with conventional equipment, and following this, it was finally shaped by a subgrade planer before concrete was placed. Generally speaking, the subgrade and granular base courses were thus similar throughout the length of the experimental pavement, and were compacted to an acceptable degree of uniformity.

The concrete throughout was of the same mix proportions designed to meet the following requirements: compressive strength at 28 days, 3,500 psi; flexural strength at 10 days, 550 psi; slump 2 in.; and air content, $4\frac{1}{2}$ percent ± $\frac{1}{2}$ percent.

A fixed cement factor of 569 lb per cu yd was used. The aggregates were a natural sand and a crushed dolomitic limestone of $1\frac{1}{2}$-in. nominal maximum size, a water-reducing set-retarding admixture of the calcium lignosulphonate type was incorporated in the mix. The concrete was laid in two 12-ft lanes using conventional paving mixers, spreading and finishing equipment.

The asphalt surfacings and construction methods were conventional other than in Section B. The H. L. 6 binder course used in Sections A, E, F and G, had the following gradation:

Tyler Std. Sieve	% Retained	Tyler Std. Sieve	% Retained
$\frac{3}{4}$-in.	4	No. 14	74
$\frac{5}{8}$-in.	15	No. 28	81
$\frac{1}{2}$-in.	29	No. 48	89
$\frac{3}{8}$-in.	40	No. 100	95
No. 4	58	No. 200	98
No. 8	65		

The coarse aggregate was a crushed dolomitic limestone and the fine aggregate a natural sand. The asphalt cement was 85-100 penetration used at 5.3 percent by weight of the total mix.

The H. L. 1 surface course used in Sections A, B, C, D, E, F, and G, had the following gradation:

Tyler Std. Sieve	% Retained
$\frac{1}{2}$-in.	0
$\frac{3}{8}$-in.	14
No. 4	45
No. 8	56
No. 14	68
No. 28	79
No. 48	88
No. 100	97
No. 200	98

The coarse aggregate was a traprock and the fine aggregate a natural sand. The asphalt cement was 85-100 penetration used at 5.7 percent by weight of the total mix.

For Section B open-graded binder course it was intended to use aggregate of the following grading:

Screen Size	% Retained
$2\frac{1}{2}$-in.	0
2-in.	14
$1\frac{1}{2}$-in.	40
1-in.	70
$\frac{5}{8}$-in.	88
$\frac{1}{2}$-in.	94
No. 4	100

The asphalt cement was 85-100 penetration grade at 3 percent by weight of the total mix.

After some 500 ft had been laid in the passing lane, plant mixing difficulties and segregation on laying, due to the large size of the aggregate, led to the following grading for the balance of the work:

Screen Size	% Retained
$1\frac{1}{2}$-in.	0
1-in.	50
$\frac{5}{8}$-in.	80
$\frac{1}{2}$-in.	90
No. 4	99

This mix had an asphalt content of 3.5 percent and was more cohesive than the first one used. No problems in mixing or laying were experienced and subsequently no noticeable difference in performance due to change in mix has been detected. An idea of the nature of this special binder course is given by Figure 5.

The completed pavement was opened to traffic in December 1959.

PERFORMANCE STUDIES

Because the object of this report is to review briefly the general existence and performance of known experimental pavements, the results of the special studies on the Ontario pavement will not be described. Performance of the different sections over the first three years is shown in Figures 6 to 14. The method of plotting is intended to bring out the development of cracks with time. Figures 15 and 16 show the crack width distributions and incidence of cracking in each section after three years.

Figure 5. Open textured asphaltic concrete base course, test section B.

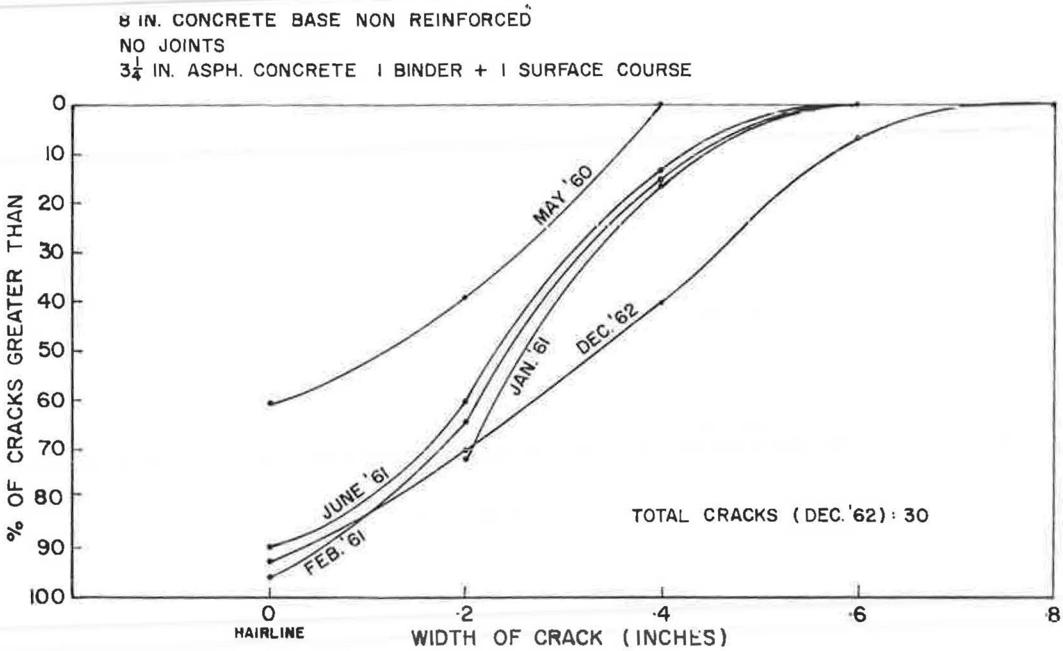

Figure 6. Section A crack width distributions.

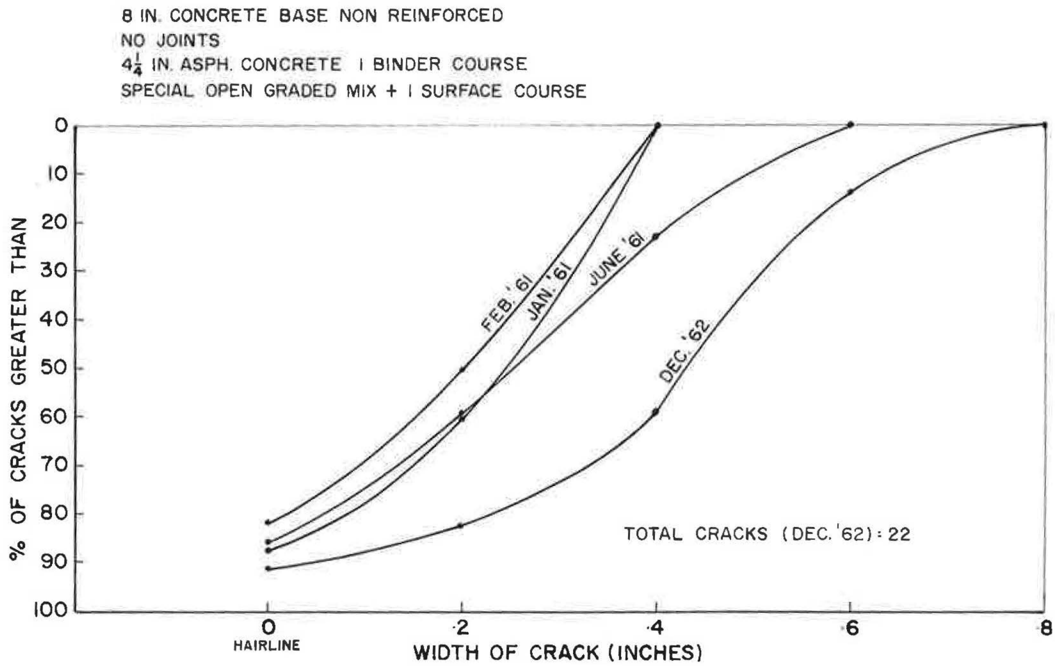

Figure 7. Section B crack width distributions.

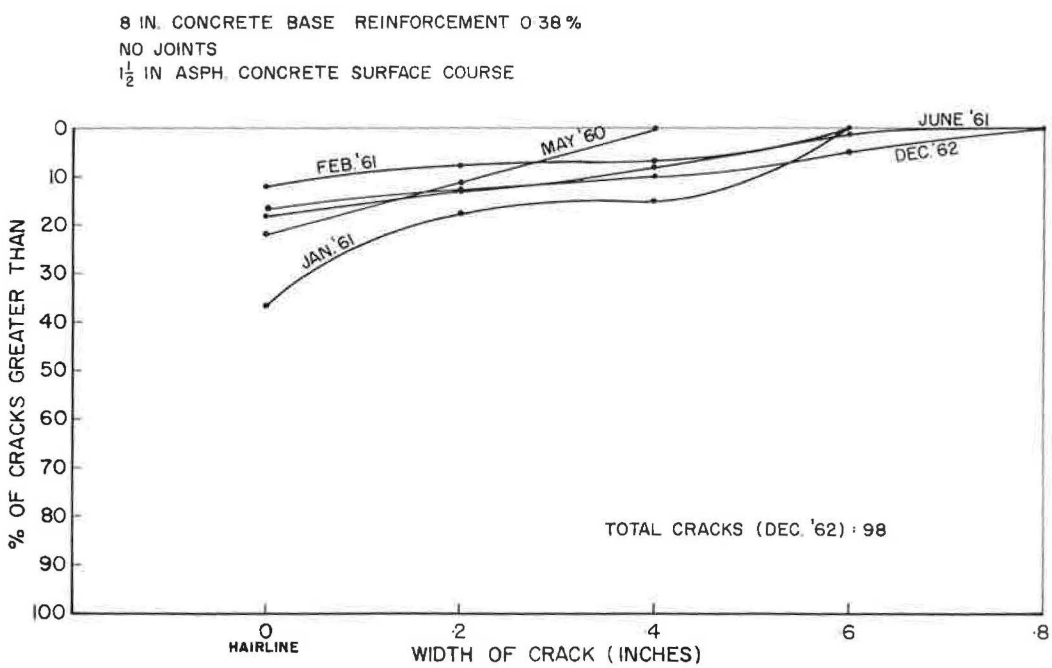

Figure 8. Section C crack width distributions.

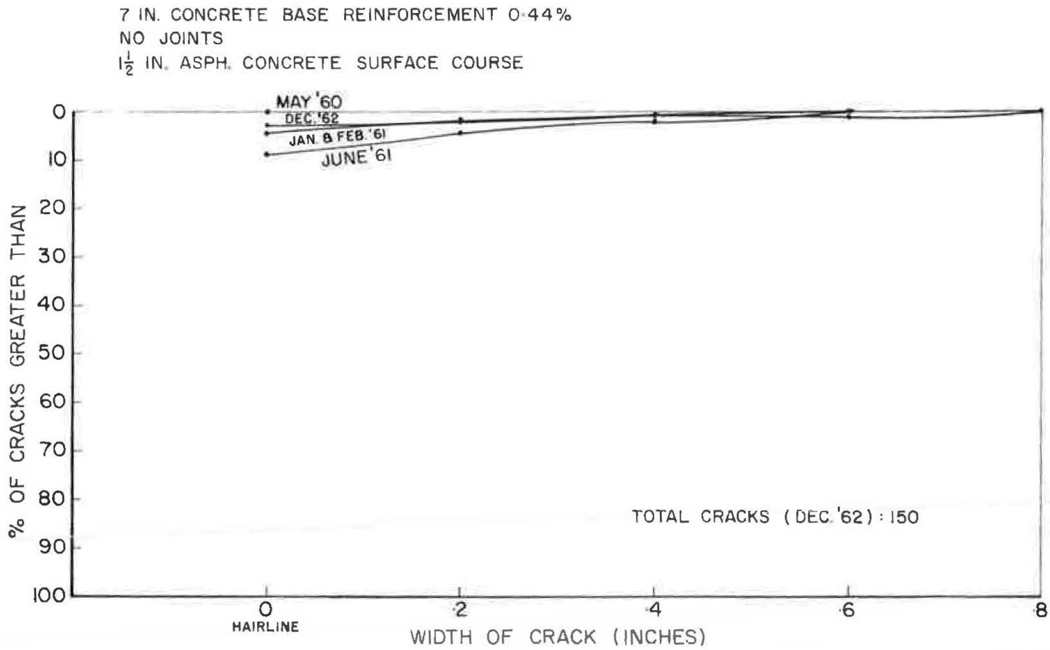

Figure 9. Section D crack width distributions.

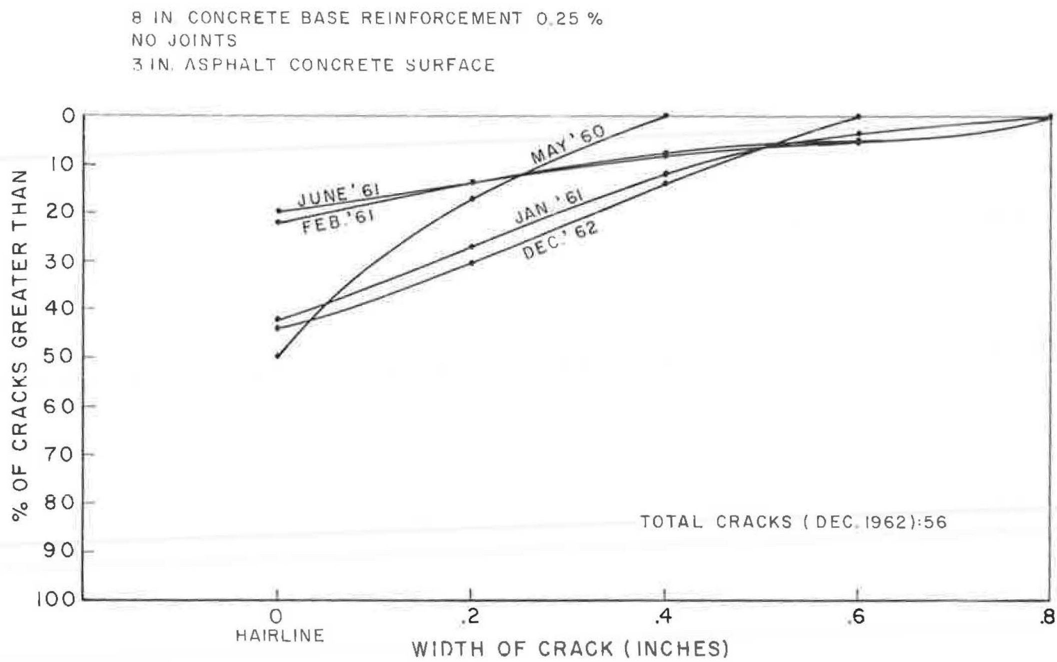

Figure 10. Section E crack width distributions.

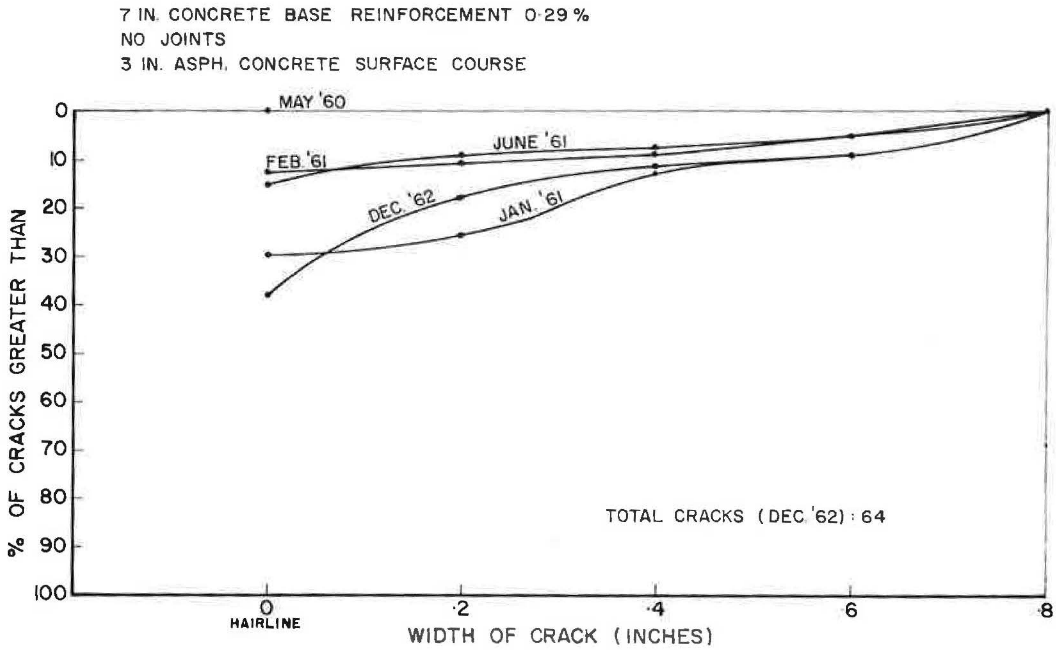

Figure 11. Section F crack width distributions.

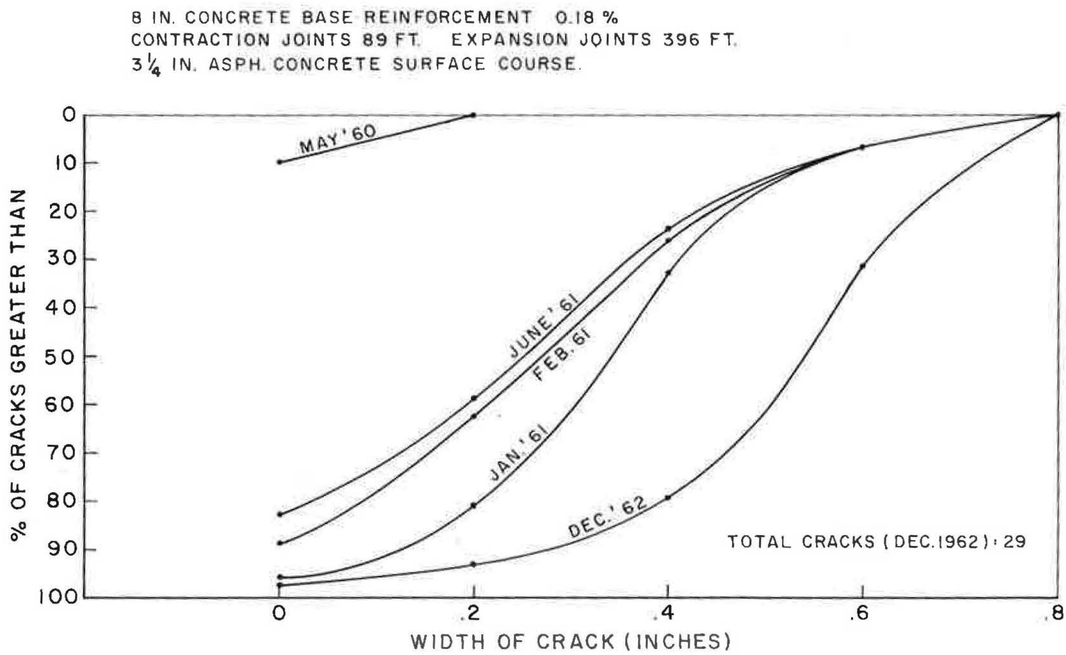

Figure 12. Section G crack width distributions.

8 IN. CONCRETE PAVEMENT CONTINUOUS REINFORCEMENT 0·38 %
NO JOINTS
NO ASPHALT SURFACING

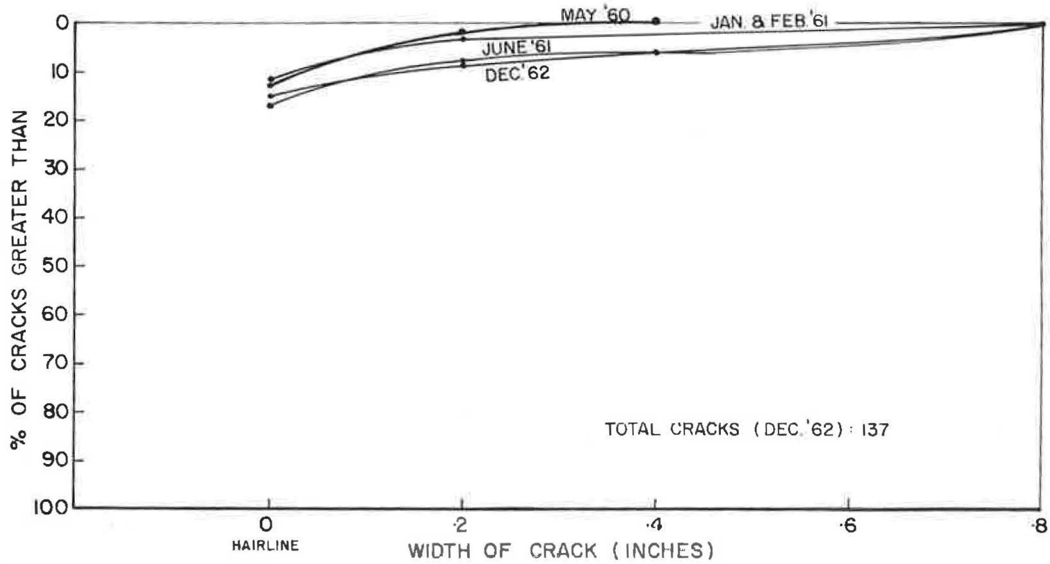

Figure 13. Section H crack width distributions.

9 IN. CONCRETE PAVEMENT REINFORCEMENT 0·16 %
CONTRACTION JOINTS 99 FT. EXPANSION JOINTS 396 FT.
NO ASPHALT SURFACING

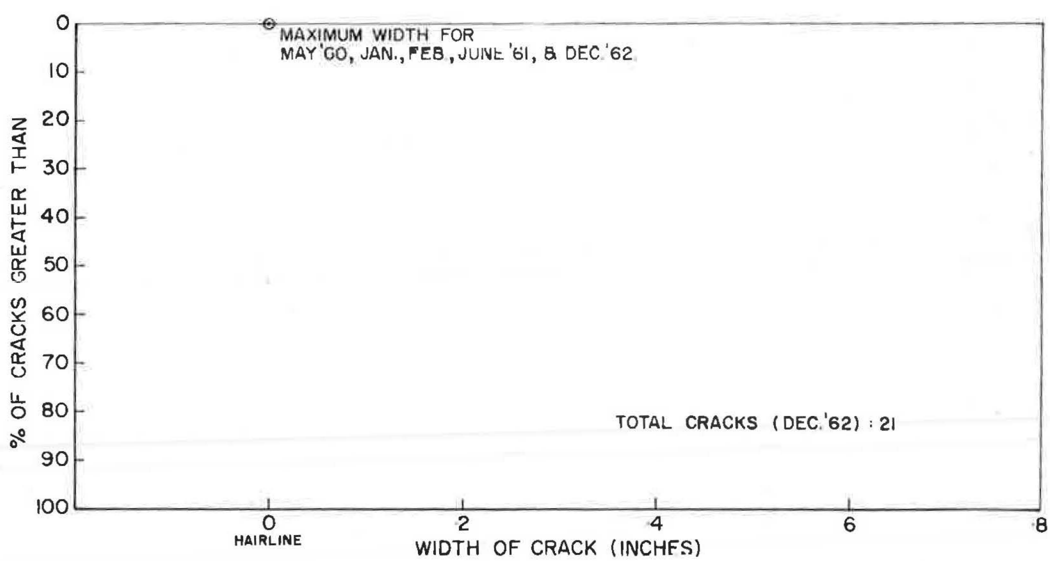

Figure 14. Section I crack width distributions.

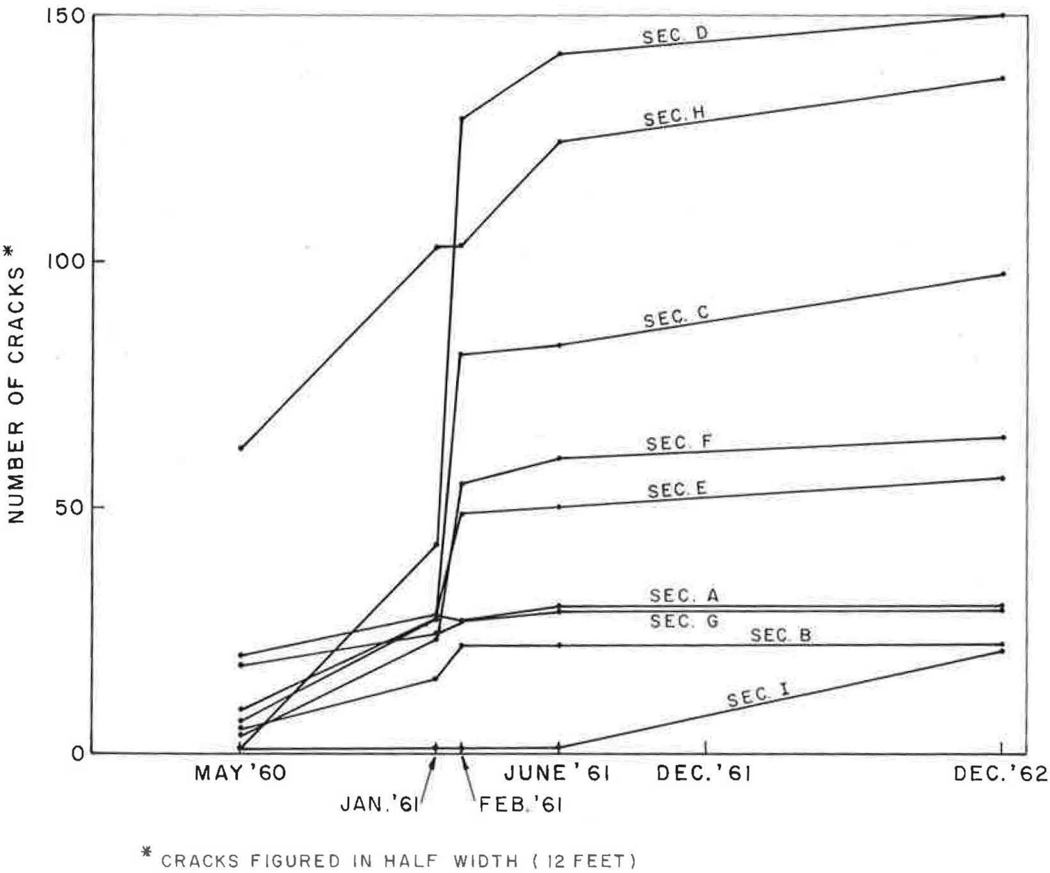

Figure 15. Incidence of cracking during first three years.

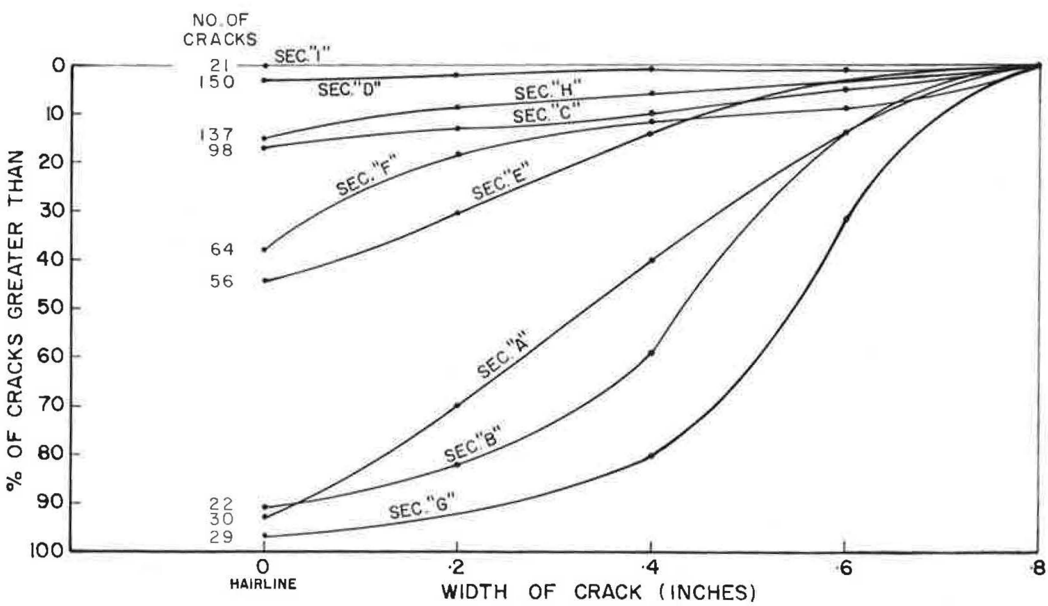

Figure 16. Crack width distributions after three years.

Both nonreinforced bases (Sections A and B) show the fewest number of cracks, with a slight advantage for Section B which has the open-graded asphalt binder course. Section D which has the thinnest overlay and highest percentage of steel has developed approximately six times as many reflection cracks as A or B, although, of course, in Section D nearly all the cracks are narrow. Comparing C, D, E, and F, crack incidence appears directly related to the amount of steel irrespective of the thickness of asphalt surfacing. Purposely jointing the pavement as in G with reinforcement to control intermediate cracking appears to offer no advantage over letting cracks occur naturally in nonreinforced bases such as A or B. Even when the reflection of 12 joints in G is allowed for, the comparison between G and the other jointed Section I, which has no asphalt surfacing, is not favorable to G. The presence of a thin asphalt surface over continuously-reinforced concrete appears, however, to be favorable in that Section C shows considerably less cracking than does Section H. The thicker asphalt surfacings on E and F do not appear to have compensated for the lower percentages of steel if a comparison is made with C and D.

For all sections, the bulk of cracking developed during the second winter with little further development in the third year. Examination of crack width distributions after three years (Fig. 16) (by which time further incidence of cracking had apparently ceased) shows that those pavements with the smallest number of cracks tend in fact to have the widest cracks. This is as would be expected, but it is noteworthy that in this respect too, A and B are performing better than G.

If the reinforced bases E, F, and D are considered in order of decreasing crack widths, this also places them in order of increasing percentages of reinforcing steel, and thus, in the order of increasing cost. However, comparing F with D, or E with C, it is doubtful if even the use of almost half again as much steel in the latter has provided full practical control of cracking. Indeed, taking an economic view of performance, it does appear that the unreinforced bases are performing quite adequately and were obviously considerably cheaper in first cost than were the reinforced ones. It would be interesting to know if their performance could be improved by sawing or forming contraction joints at very frequent intervals, so as at least to equal the 150 cracks in D, of which only 10 percent are wider than hairline. The alternative to this, if the incidence of wide cracks is to be positively avoided, would appear to be to use steel in at least the amount of 0.44 percent as in Section D, or consider if pre-sawing contraction cracks at frequent intervals in a base with a lower percentage of steel might induce a suitably short-spaced crack pattern.

The temperature effects introduced by the presence of a black asphalt surfacing are worthy of comment. Figure 17 is typical of the kind of 24-hr temperature cycle to which a pavement is subject in warm sunny weather. Starting in the early evening, the temperatures of the asphalt surface and the interface with the concrete are the same and are approximately 15 F above ambient. The surface temperature of the asphalt then falls rapidly due to radiation, so that within 3 hr it is at ambient and later in the night is up to 4 F below this. Only 1½ in. below this surface, the temperature of the concrete throughout the night remained 5 F higher. Once the sun came up, the surface rapidly heated up at a rate of 5 to 7 F per hour and the night temperature picture quickly reversed itself, the increase in concrete base temperatures lagging well behind that in the asphalt surfacing.

Throughout the 24 hours, the temperature at the bottom of the concrete base varied by only 10 F, whereas the upper surface of the slab varied 25 F, and the upper surface of the asphalt cycled through a 35 F temperature change. Such a pattern emphasizes the importance of temperature effects in magnifying such problems as slab curling. During a sunny afternoon when the ambient temperature was 87 F, the surface temperature on one of the exposed concrete slabs was 97 F, and the temperature differential through 8 in. of concrete was 23 F. Correspondingly, on an 8-in. concrete base surfaced with 1½ in. of asphalt, the surface temperature was 108 F, with a differential through the asphalt of 20 F, and through the concrete of 14 F. Where the asphalt surfacing was 3¼ in. thick, of the same total temperature drop, 28 F was now taken up in the asphalt and only 6 F by the concrete. This appears to indicate the insulating value of thick asphalt surfacings in reducing the curling problem.

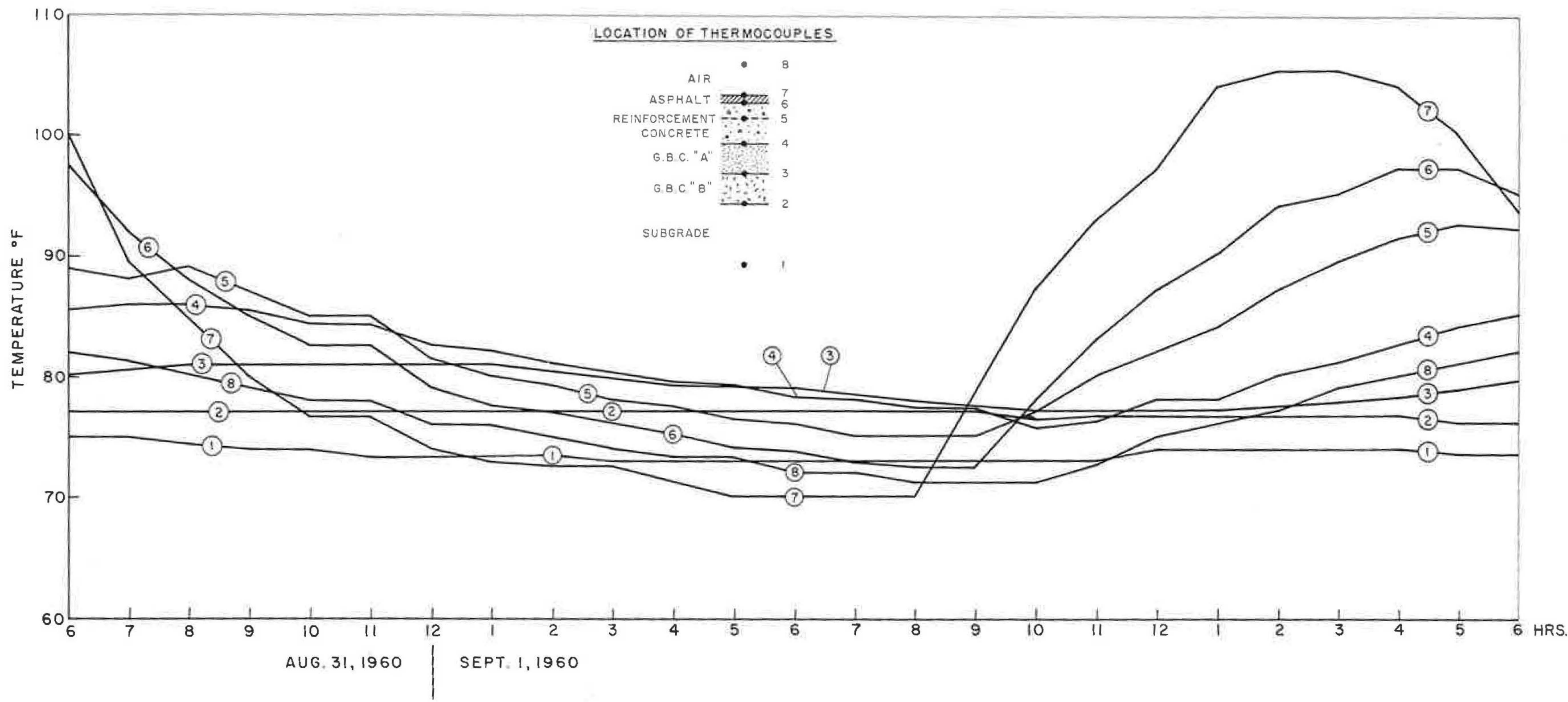

Figure 17. Typical 24-hr summer temperature cycle for concrete base, asphalt-surfaced composite pavement (Section C).

Longitudinal cracking along the pavement centerline has occurred through the asphalt wherever tie bars across the centerline were not used.

The 10-ft long expansion wedges between dissimilar bases have performed very satisfactorily.

RIGID OVERLAY OF A FLEXIBLE PAVEMENT

Because in nearly every case when a new composite pavement is created from an existing pavement it is by overlaying the old concrete with asphalt, it may be of interest to document the "upside-down" case.

A 2-mi section of highway 401 at Prescott, Ont. was reconstructed in 1962 by overlaying the existing flexible pavement with a 9-in. reinforced concrete slab. A short experimental section was built to learn if adequate load transfer between adjacent concrete slabs could be obtained through the existing asphalt base without using load-transfer devices. In addition, some of the joints were skewed 2 ft in 12 ft and others were square to see if skewing of joints maintained a better ride over the years. Some irregularities in the existing pavement and corrections to the crown had to be taken out with a thin, well-compacted layer of crushed limestone screenings. It will be some time before performance can be reported.

EUROPEAN EXPERIMENTAL COMPOSITE PAVEMENTS

The review made of European practice at the time of the design of the Ontario composite pavement showed their concern with the following problems.

Reflection Cracking

Inspections of asphalt-surfaced concrete roads in London, England, have shown that cracking appeared over all types of joints, but was generally worse at expansion joints. The severity of the cracking tended to increase with increases in expansion-joint spacing. Single-course asphalt overlays of up to 3 in. did not appear to be sufficiently thick to prevent such reflection cracking, although it had been noted that where the surfacing was laid in two courses there was a better chance of producing a crack-free wearing course.

Temperature Effects

Skinner and Martin (2) draw attention to the insulating effects or otherwise of an asphalt surfacing. They concluded that at least 3½ in. of asphalt surfacing was required to reduce temperature warping in the base. Such a thickness of surfacing would provide for a 30 F temperature differential, which they felt was required to overcome positively the 25 F higher than ambient temperatures which had been measured on black surfaces. They also found that through the concrete base itself, the temperature differential in summer is usually in the range of 2 to 2½ F per inch of depth.

Design Practice

Skinner and Martin (2) then suggested that considering the previous factors, for a concrete base surfaced with more than 3½ in. of asphalt, in the weakest condition of corner curling the distance through which a load has to act to deflect the bottom of the concrete back onto the subgrade is thus reduced and the slab strengthened. In addition, they felt that there was also a reduction of the actual loads which act on the underlying concrete corners due to the load-spreading property of the asphalt surfacing. They then continued their assessment of design to develop thicknesses of such bituminous surfacings to achieve strengthening of existing rigid pavements on the basis of Load Classification Number required. (The L. C. N. system has been defined elsewhere by Skinner (3) as "being based on a study of the load-bearing performance of various types of pavement in existence, from which it has been possible to derive a classification in which a series of numbers indicate the combinations of weights and associated tire pressures which will produce the same effect on a pavement.") In such cases, the concrete base itself would have been designed on the k value of the subbase. However, for lean-mix concrete or other stabilized bases, the design analysis of the structure has usually been on the basis of CBR values as for a fully flexible pavement with an empirical reduction of 30 percent in the total thickness of the pavement.

Lean-Mix Concrete Bases

One feature of European practice is the use of lean-mix concrete base in much greater depth than has been the practice with stabilized bases in North America. A general review of current practice in Great Britain has been reported by Sharp (4).

A typical application of these bases to highway construction was their use on the M-1 Motorway between London and Birmingham. The basic design for this pavement was 14 in. of lean-mix concrete on a 6-in. granular subbase, surfaced with 4 in. of two-course hot-rolled asphalt. Design and construction of this pavement is fully described in papers by Williams and Williams (5), and Laing, Broadbent and Fisher (6). The pavement was constructed in 1957-1958; there have been some preliminary reports of failures in certain sections.

Some of the considerations have been examined by means of experimental composite pavements. Fuller details and information on performance when studies are completed appear, or will appear, in the reports of the Road Research Laboratory, Department of Industrial and Scientific Research, England. Some of the more significant experimental composite pavements are as follows:

1. Cromwell Road, Hammersmith, London. Constructed in 1955-1956 to determine the joint spacing in concrete that will result in the minimum amount of maintenance of cracks in a rolled-asphalt surface.
2. Crawley New Town, Sussex. Constructed in 1956-1957 to determine whether the inclusion of light reinforcement in a lean-concrete base will reduce the incidence of cracks in the asphalt surfacing. The effect of incorporating a bituminous emulsion in the mixing water for the lean-concrete base was also investigated.
3. Great Cambridge Road, Middlesex. Constructed in 1956-1957 to determine which type of concrete base (reinforced, plain or lean mix) will result in the minimum amount of cracking in a rolled-asphalt surface.
4. Alconbury Hill, Hunts. As part of a full-scale road experiment, certain composite sections were constructed. The whole experiment was to determine the minimum thickness of concrete needed to give satisfactory performance under heavy traffic on a heavy clay subgrade, and to determine the effects of inclusion or not of reinforcing and the type and thickness of base.
5. Klesterbach, Germany. A section of a road was built near the east end of the Cologne to Frankfurt-am-Main Autobahn to determine whether some of the defects that have occurred in exposed concrete roads can be avoided by thick bituminous surfacings. The concrete base was unreinforced, 8-in. thick with a cement-aggregate ratio of 1:15 and normal jointing surfaced with 5 in. of bituminous material.

ACKNOWLEDGMENTS

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R. Collins' assistance with compaction and control instrumentation at the time of construction and the recording of performance data by B. Horvath, both of the Materials and Research Division, are also gratefully acknowledged.

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