HIGHWAY RESEARCH RECORD

Number 37

Composite Pavement Design

9 Reports

Presented at the
42nd ANNUAL MEETING
January 7-11, 1963

HIGHWAY RESEARCH BOARD
of the
Division of Engineering and Industrial Research
National Academy of Sciences—
National Research Council
Washington, D. C.
1963

Department of Design

T. E. Shelburne, Chairman
Director of Research
Virginia Department of Highways, Charlottesville

COMMITTEE ON COMPOSITE PAVEMENT DESIGN

Robert F. Baker, Chairman
Director, Office of Research and Development
U.S. Bureau of Public Roads, Washington, D.C.

Harold Allen, Chief, Division of Physical Research, Office of Research, U.S. Bureau of Public Roads, Washington, D.C.
Fred J Benson, Dean, School of Engineering, Texas A & M College, College Station
William N. Carey, Jr., Deputy Executive Director, Highway Research Board, Washington, D.C.
W. E. Chastain, Sr., Assistant Engineer of Research and Planning, Research Branch, Illinois Division of Highways, Springfield
Phil Fordyce, Supervising Engineer, Pavement Engineering, Portland Cement Association, Chicago, Illinois
William A. Goodwin, Assistant Program Engineer, National Cooperative Highway Research Program, Highway Research Board, Washington, D.C.
John M. Griffith, Director of Research and Development, The Asphalt Institute, University of Maryland, College Park
James H. Havens, Assistant Director of Research, Kentucky Department of Highways, Lexington
R. E. Livingston, Planning and Research Engineer, Colorado State Highway Department, Denver
Thomas B. Pringle, Chief, Civil Engineering Branch, Engineering Division, Military Construction, Office, Chief of Engineers, Department of the Army, Washington, D.C.
Robert L. Schiffman, Associate Professor of Soil Mechanics, Rensselaer Polytechnic Institute, Troy, New York
Rollin J. Smith, Engineer - Manager, Asphalt Department, Skelly Oil Company, Kansas City, Missouri
John H. Swanberg, Chief Engineer, Minnesota Department of Highways, St. Paul
William Van Breemen, Research Engineer, Engineering Research, New Jersey State Highway Department, Trenton
Eldon J. Yoder, Joint Highway Research Project, Purdue University, Lafayette, Indiana
Ernest Zube, Supervising Materials and Research Engineer, California Division of Highways, Sacramento

Preface

The following symposium was developed by members of the Composite Pavement Design Committee in order to provide highway engineers with a broad view of the status of the use of composite pavements. This is the second document developed by the Committee, the earlier report being the Research Correlation Service Circular No. 473, "Suggestions for the Study of Composite Pavements."

The Committee is indebted to Peter Smith, Sr., Materials Engineer of the Ontario Highway Department, for his participation in the symposium.

The symposium was assembled and edited by a task force including Thomas B. Pringle, William A. Goodwin, Rollin J. Smith, and William Van Breemen, with Mr. Pringle acting as principal coordinator. The writing contains essentially the original expressions of the various authors. No effort was made to obtain an edited version which would represent the consensus of the entire Committee.

There is no opinion within the Committee that composite pavements are superior to other types. Rather, it is the hope that composite pavements will provide alternate solutions which will be economical and capable of good performance, particularly in areas where aggregate quality is marginal. Good coordination has existed with the rigid and flexible pavement committees of the Board through the active membership of William Van Breemen and R. E. Livingston, Chairmen, respectively, of the Rigid and Flexible Pavement Design Committees.

Comments and suggestions are solicited by the Committee, particularly in the realm of experience records. Correspondence may be directed to any member of the Committee.

Contents

)efinition of Composite ²avement Structures

:OLLIN J. SMITH, Engineer-Manager, Asphalt Department, Skelly Oil Company, Kansas City, Mo.

›THE definition, approved by the Subcommittee on Definitions and under consideration ›y the Committee, is as follows:

> Composite pavement structure—A structure comprising multiple, structurally significant, layers of different, sometimes heterogeneous composition. Two layers or more must employ dissimilar, manufactured binding agents. (Note: surface treatments, thin overlays, membranes, lamina, and the like having no significant structural strength, shall not be considered layers in arriving at the type of pavement structure.)

A search for positive rules, a modus operandi, and principles to use in the approach to formulation of definitions of words and terms, became an immediate necessity in the initial efforts of the task group assigned to this project. The final objective was to prepare a definition for what was then described as a composite pavement, an objective which was soon found to be a laborious task. Actually, the task became one of an intimate study of every word which might possibly be used in a definition of the term, and a brief summary of the steps that were taken in the development of a philosophy governing the approach to the definitions is a necessity to its understanding.

The minutes of the first meeting of the HRB Committee on Composite Pavement Design shows that the item of major concern was a proper definition for the term "composite pavement." There was no agreement on a definition at the meeting. The chairman requested each member of the committee to submit (1) a definition of composite pavement, (2) types which he would include in the composite category, (3) definition of the term "flexible," and (4) definition of the term "rigid." The committee responded, and their replies were handed to a committee of two which had been designated by the chairman on December 7, 1960, as a task group on the definition of terms. This task group began immediately to find that the various definitions presented were so divergent as to approach and concept as to appear hopelessly irreconcilable. Identical words used by different members were so obviously different in usage as to contribute to the confusion when taken as a whole. It did not take long for the task force to find that this babel of scientific language was not attributable to individual usage but was a result of centuries of inattention and even conscious resistance to literal interpretations. The task group was already cognizant that this condition existed in a small measure, but it was not until they examined a number of state specifications that they found such common words as pavement, subbase, and subgrade, had entirely different meanings in the various states; and it was decided that this was the proper place to begin a solution to the problem. The word subgrade, for instance, in the state specifications examined, was found to describe collectively every level in a multiple layered pavement structure except the final pavement surface.

The study recognized and included standard definitions of technical terms and words wherever such definitions have been agreed upon and published by professional organizations. In this regard the manuals of AASHO and ASTM, and the major standard technical dictionaries were ready sources of reference. In addition, it seemed highly desirable to include pertinent legal terms, as defined in standard law dictionaries, and to refer to particular interpretations given to engineering terms and concepts by the courts and legislatures.

A progress report was made at the meeting of the Committee on Composite Pavement Design, January 4, 1961, and the problems with which the task group was faced were pointed out. Later a summer meeting at LaSalle, Ill., September 21-22, was called to develop definitions for "rigid," "flexible," and "composite" pavement. Definite rules, the modus operandi, and the principles for which the group had been searc ing were submitted to the committee and unanimously approved as follows:

> 1. Words and terms used in committee communications shall, when possible, carry literal significance.
>
> 2. Definitions of words and terms approved by other Highway Research Board Committees, by the American Association of State Highway Officials, and the American Society for Testing Materials, shall be used by this Committee when not in conflict with each other and when not in conflict with the general policies of this Committee.
>
> 3. Words and terms shall, without exception, be interchangeable when descriptive of layered systems in all types of pavement structures.
>
> 4. In defining terms such as composite pavement structures and the like, this Committee, being committed to literal definitions, may include under this heading too many alternatives for effective consideration. Then too, in certain cases where there is no positive or agreeable distinction between, as for instance, a composite and a flexible type, the Committee shall determine its literal classification, and this classification shall be maintained even though, by agreement, consideration of the type is turned over to some other committee.

At the same meeting the committee approved definitions of a considerable list of words, after thorough study of various authorities on the subject, and agreed to the general policy submitted to them for approval, to-wit:

> A study of words and terms by the subcommittee has disclosed an almost unbelievable number of idiomatic and dialectic expressions of definitions in addition to those which are explicit and generally recognized. There is no alternative than for the subcommittee to point out such situations and make recommendations for the substitution of explicit terminology.
>
> We believe it is high time for engineers to cease being their own lexicographers. There is no good reason for neologism, dialecticism, or vernacularism. Our diction should be literally exact to be universally understood.

The task group was then directed to a further consideration of the definition for composite pavement, and was grateful at this point to receive new inspiration and reassurance that they were on the right track. It is of interest that 1961 marked the 400th anniversary of Sir Francis Bacon's birth. From the learned book by Charles Coulston Gillispie entitled "The Edge of Objectivity," the following paragraphs are quoted:

> The subject matter of Bacon's writings falls into three categories: demonstration of the worth and dignity of learnings; analysis of the obstacles which kept it languishing in futility; and prescriptions for its reformation and advancement. It is not, perhaps,

necessary to insist much on the first point—indeed, it was not so necessary in the early seventeenth century as Bacon would imply. His pleas for learning generally took the form of a rather scornful repudiation of all that passed for such. As for the hindrances, it was trite enough to blame the sterile habit of reliance on authority and the circularity of scholastic logic. But though no student of science, Bacon was an extremely acute student of human beings, and in his discussion of the obstacles raised by the intellect against itself, he showed his mettle. There is that in the very constitution of our understanding which renders the mind a pesky instrument for innovation. "Idols," Bacon called these innate blinders. The "Idols of the Tribe" are distortions which arise from our common nature: "The human understanding is no dry light, but receives an infusion from the will and affections; whence proceed sciences which may be called 'sciences as one would.' For what a man had rather were true he more readily believes." The "Idols of the Cave" compound this common tendency to error with the favorite prejudices or enthusiasms of the individual man, each of whom "has a cave or den of his own, which refracts and discolours the light of nature."

Third, are "Idols formed by the intercourse and association of men with each other, which I call Idols of the Market-Place on account of the commerce and consort of men there. For it is by discourse that men associate, and words are imposed according to the apprehension of the vulgar. And therefore the ill and unfit choice of words wonderfully obstructs the understanding." This was perhaps the most penetrating and valuable of Bacon's observations. Not much can be done about human nature, after all, any more than about gravity or inertia, even when its disadvantages are recognized. But identification of the error that lurks in words was the first step to correction. The attempt to put precision into scientific language has never since been relaxed. Humanists may complain of the jargon of the specialties, sometimes with justice. But no science can flourish until it has its own language in which words denote things or conditions and not qualities, all loaded with vague residues of human experience.

At the annual meeting, January 6, 1963, the task group, which had been increased to four members, presented a definition for a composite pavement structure which was considered, amended, and remanded to the task force with the instruction that they should send out a questionnaire which would include many and varied typical sections, and which would prove or disprove the ability of the definition to define clearly the types which would fall in the category of composite pavement structures. The task group prepared such a questionnaire which included 18 typical sections including all those of normal usage as well as some in the fringe areas of the future and the past. That the definition is adequate is attested to by the fact that there was perfect agreement on 14 of the 18 typical sections, a vote of 8 to 1 on 2 of the typical sections, the only areas of disagreement being on sections 15 and 18 which were bituminous-filled brick on an asphaltic-concrete base, and monolithic brick on a portland cement concrete base.

The response to the questionnaire demonstrates that the classic definition of a composite pavement structure as amended, completely separates it from the fields of the Flexible Pavement Committee and the Rigid Pavement Committee. As a matter of fact, the classic definition of a composite pavement structure would leave new fields for investigation by the Flexible Pavement Committee and the Rigid Pavement Committee, if it should be found desirable to make the general division of the work on sucl a basis.

That the term "composite pavement structure" as now defined would include all surfaced roads is fallacious as demonstrated by the committee's investigation of just such an objection which was made to their definition. The point was advanced that in the strictest sense all roadways except graded earth roads are composite; that as soon as crushed stone is placed on a clay subgrade, a pavement structure of two composite elements has been constructed having different engineering properties, and on this account such a road would be considered a composite pavement structure. The committee's investigation of this objection following the line of its adopted philosophy of delving into the literal definitions of words was as follows:

a. Soil - (HRB Abstracts Vol. 29, No. 6, June 1959) Stone, gravel, sand, silt, clay, or any combination thereof as defined by AASHO M145 and M146. (Note: Particle size, rather than origin of material, is the basis of the foregoing definition. Cinders, crushed stone, slag, chert, caliche, etc., are thus considered within the definition of soil.)

b. Pavement - (Corpus Juris Secundum) The meaning of the word "pavement" is not limited by the particular material used, for no particular material is necessary, and a pavement may be made of anything which will produce a hard, firm, smooth surface for travel, and, as a general rule, any substance which is spread upon a street to form a compact, hard, or level surface or floor may be properly designated a "pavement," although ordinarily the term is not applied to the gravel and stone coating placed on country roads.

c. Pavement - In the legend on almost any road map there will be found substantially this ascending scale: dirt; graded; improved (gravel); paved; etc.

Thus, from the standpoint of the committee, an ordinary earth road surfaced with cinders, crushed stone, slag, chert, caliche, etc., is not a composite pavement structure, in fact, it is not even a pavement. The definitions just given indicate that there is engineering authority, legal authority, and the authority of cartographers for considering such roads to be nothing more than "improved earth roads."

Discussion of Possible Designs Of Composite Pavements

WILLIAM VAN BREEMEN, Research Engineer, Engineering Research, New Jersey State Highway Department

•THE purpose of the following is to discuss very briefly a few of the numerous possible designs of composite pavement.

Actually, in principle, a pavement of this type is by no means of recent development. For example, over a period of many years, pavements involving a portland cement concrete base overlaid with such materials as bituminous concrete, granite blocks, or bricks, have been constructed in many areas. Moreover, in recent years, many deteriorated PCC pavements have, in effect, been converted into composite pavements by the application of a layer of bituminous resurfacing material.

A typical design that has been used extensively for street construction in many cities, and even on some rural highways, is shown in Figure 1. This design involves a bituminous concrete surfacing on a plain concrete base, which is in turn supported on a layer of subbase. The probabilities are that most of the pavements of this type which are in service on city streets were not constructed on subbase. But despite this omission, practically all of these pavements have given a very good account of themselves, even under very heavy traffic.

In almost all cases, the plain concrete base has been constructed without joints of any kind, other than plain butt joints between each day's work. In this type of construction, the base sooner or later cracks transversely at erratic intervals, and thus becomes divided into a series of slabs. During cold weather there is a contraction of

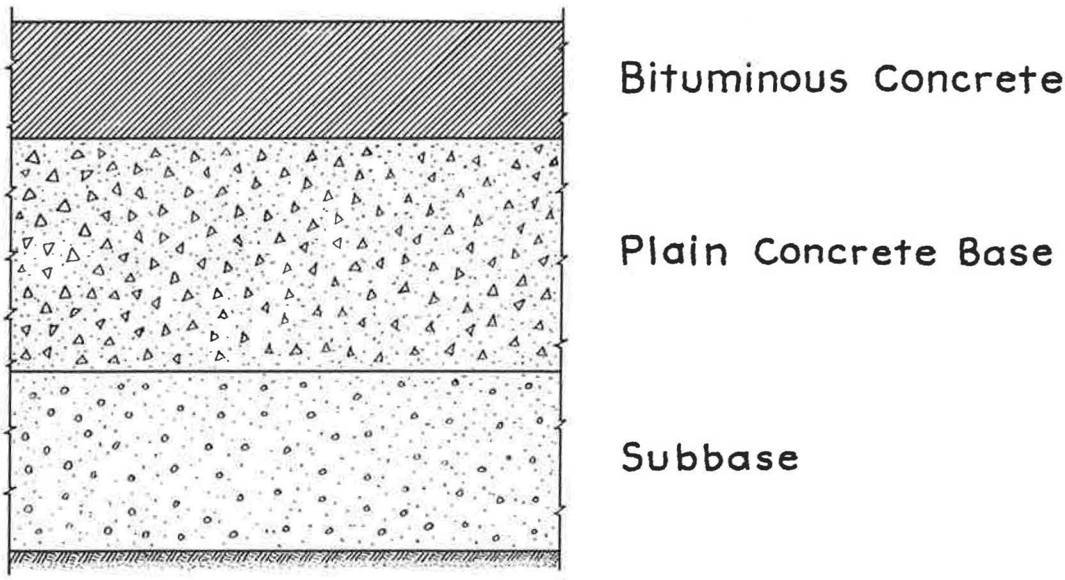


Figure 1.

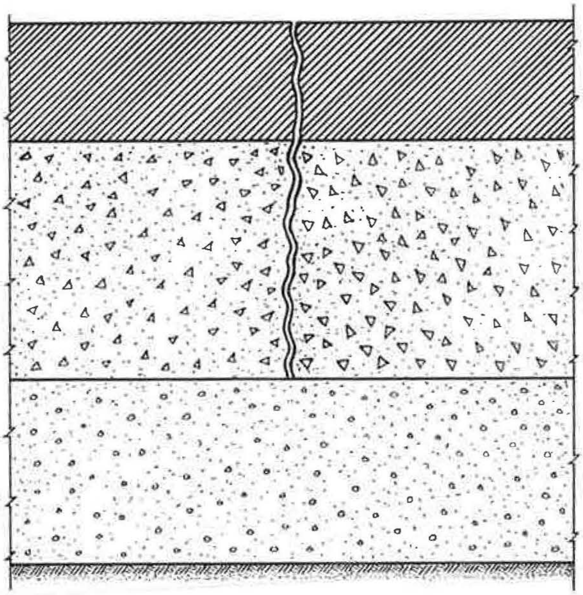


Figure 2.

the slabs, and a corresponding opening of the cracks. This, in turn, results in the development of so-called reflection cracks in the bituminous surfacing—these cracks being directly above the cracks in the base. A typical reflection crack is shown in Figure 2.

The reflection crack is coincident with the crack in the base, and of essentially the same width. These cracks are due mainly to the development of excessive tension in the bituminous surfacing, that is, immediately above the cracks in the base, with the result that the surfacing is pulled apart at these cracks. As is well known, the same sort of thing inevitably happens in connection with resurfaced concrete pavements, in which reflection cracking occurs over the joints and also over any cracks which undergo appreciable changes in width.

Reflection cracks are objectionable for several reasons, some of which are as follows:

1. They tend to undergo a progressive increase in width.
2. They permit the leakage of surface water to the subgrade. This water, especially if it contains de-icing agents, can also have a damaging effect on the base.
3. Inevitably, there is serious raveling and disintegration of the bituminous surfacing adjacent to the cracks. This, in fact, is their main objection.

For these reasons, one of the most important problems in connection with a pavement of this type is that of preventing the development of reflection cracks. Various designs which seem to offer promise of accomplishing this objective, at least to a very great extent follow.

Figure 3 shows a rather elaborate design which involves:

1. A surfacing of bituminous concrete containing reinforced steel (the steel may consist of either welded wire fabric or expanded metal).
2. A plain concrete base with contraction joints at close intervals and an underlying layer of subbase.

The function of the reinforcing steel, in effect, is to increase the tensile strength of the bituminous surfacing, to the extent that it will not be pulled apart at the contraction joints in the base.

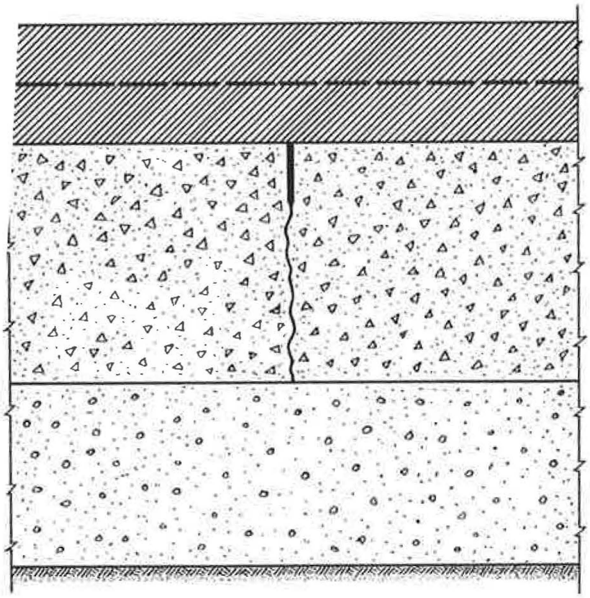


Figure 3. Typical reflection crack.

The purpose of installing the contraction joints at close intervals is mainly to restrict the amount of joint opening, and thus to avoid a situation wherein the reinforcing steel is called upon to do more than it is capable of doing.

Over a period of years, several test sections of pavement conforming essentially with this design have been constructed in New Jersey, and on the basis of the very satisfactory performance of these sections, it is now planned to utilize this design in connection with several miles of pavement soon to be constructed on an Interstate route.

Figure 4 shows a design involving a bituminous surfacing on either a lean concrete base or a cement-treated base. This design is based on the premise that, if the base has a low cement content, the cracks in the base may, for various reasons, be of such limited width as not to induce reflection cracks in the surfacing. Undoubtedly a number of pavements of essentially this design have already been constructed in various parts of the country.

Experience has indicated that relatively thick bituminous overlays are beneficial from the standpoint of at least minimizing the seriousness of reflection cracking. The design shown in Figure 5 has been developed with this in mind. This design involves a bituminous overlay of substantial thickness, in which the major portion of the overlay, for reasons of economy, consists of bituminous-stabilized base. Incidentally, it seems quite possible that one of the reasons why a thick overlay is beneficial is that it tends to reduce the over-all seasonal changes in temperature of the base, and thus to reduce the changes in width of the cracks in the base.

The design shown in Figure 6 appears to have considerable promise. It includes an intermediate layer of untreated granular material between the bituminous surfacing and the base. The purpose of this intermediate layer is, of course, to so separate the surfacing from the base that the cracks in the base will not induce reflection cracks in the surfacing. It appears, however, that there may be some risk involved in this design, especially if the surfacing is too thin, or if the materials in the intermediate layer are not of first-class quality. On the other hand, outstandingly satisfactory performance has been reported in connection with certain pavements of this design, notably in one of the Southwestern States.

To avoid such risks as may exist in connection with a relatively thin surfacing, Figure 7 goes a step farther and introduces a layer of bituminous-stabilized base be-

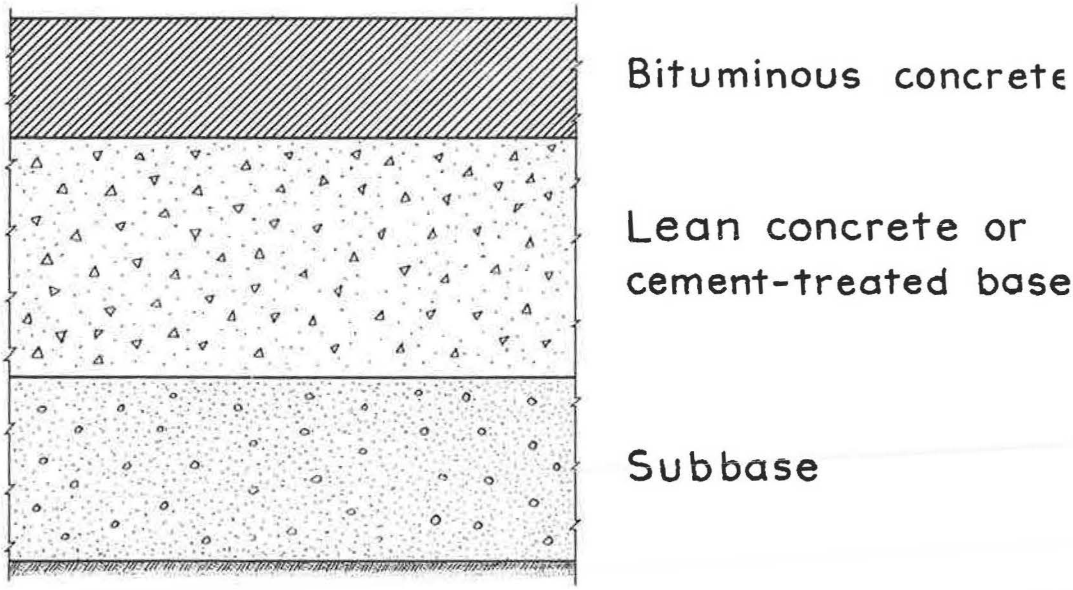


Figure 4.

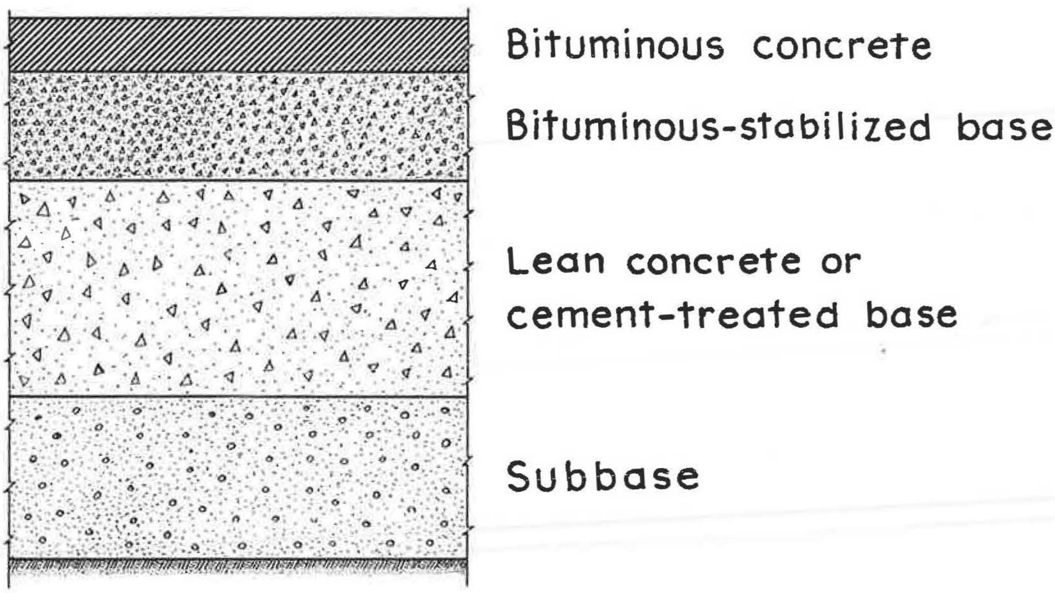


Figure 5.

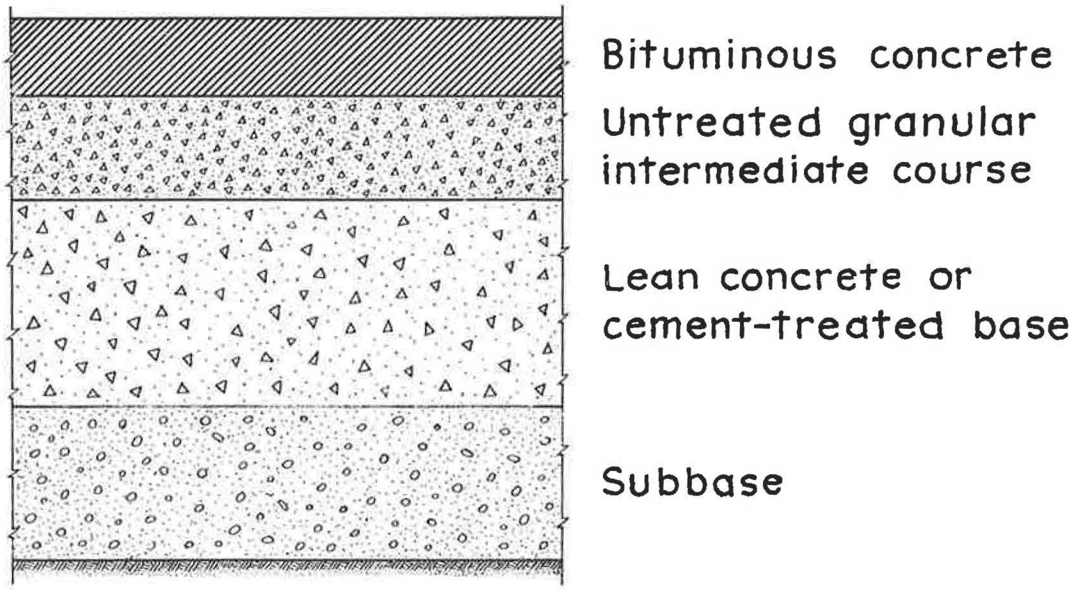


Figure 6.

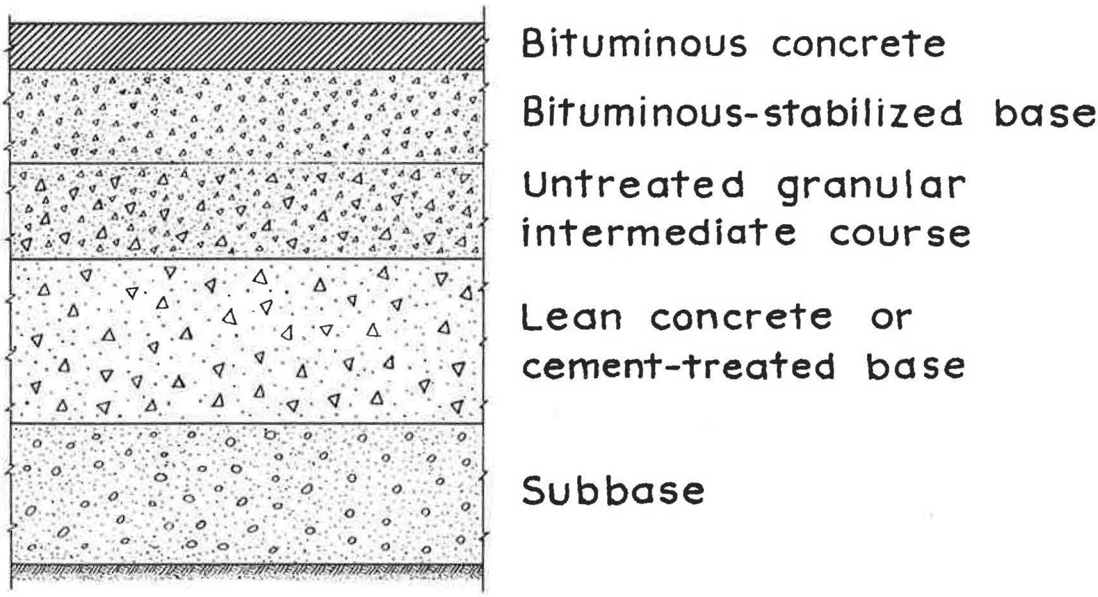


Figure 7.

tween the surfacing and the intermediate untreated granular layer. Whether or not this precaution is really necessary is something which is still to be determined. At any rate, because this design appears to offer considerable promise, it is intended to construct a test section in New Jersey on one of the major trucking routes.

Figure 8 shows a design in which the objective is to prevent reflection cracking by using a type of concrete base in which the opening of the cracks in the base is so slight as to have no adverse effect on the surfacing. As related to this type of base, those who are familiar with continuously-reinforced concrete pavements know certain basic things about them:

1. They contain a considerably greater amount of longitudinal reinforcing steel than installed in conventional concrete pavements.
2. Initially, the steel induces the occurrence of transverse cracks at very close intervals, and then subsequently prevents these cracks from opening to any significant extent.

A continuously-reinforced concrete pavement should, therefore, constitute an excellent base for a bituminous surfacing. Unfortunately, however, owing to its relatively high cost, this type of base would perhaps be warranted only under exceptional circumstances, such as in connection with a very heavily traveled pavement in an urban area, where its cost might very well be fully justified.

But be that as it may, the indications are that the required amount of reinforcing steel is more or less directly proportional to the tensile strength of the concrete. Consequently, if a lean concrete base were to be constructed having a tensile strength substantially lower than that of normal concrete, it would require proportionately less steel. Therefore, as a result of the reduction in the amount of cement and steel, it may be entirely possible to construct a composite pavement of this design at a cost which is very little if any higher than that of a high-grade pavement of conventional design, and the performance of which could prove to be notably outstanding.

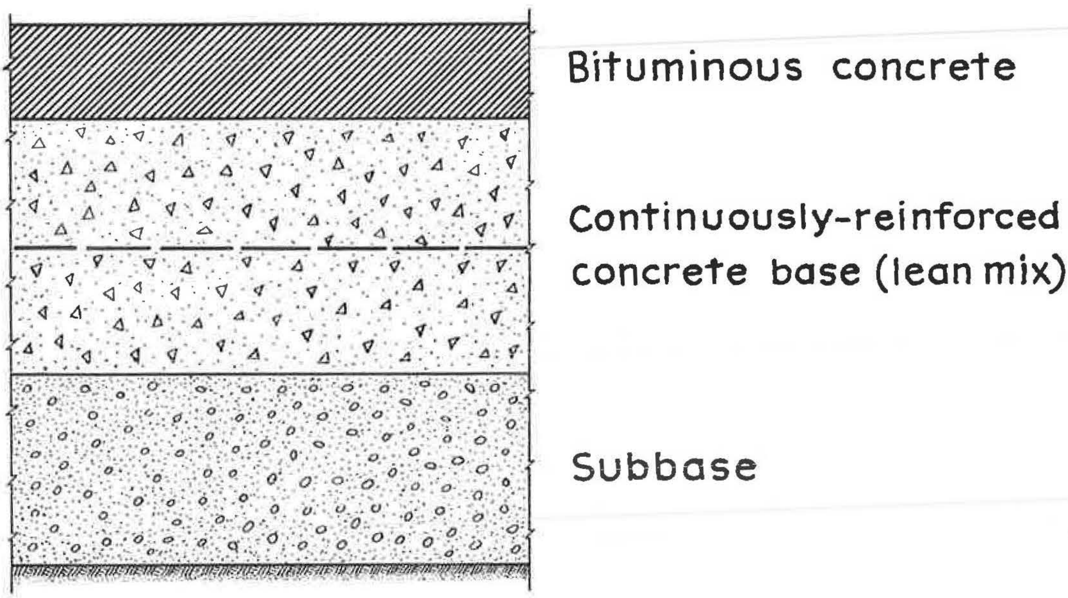


Figure 8.

Proposed Experimental Composite Pavements

HAROLD ALLEN, Chief, Division of Physical Research, Office of Research, U. S. Bureau of Public Roads

•THE Committee on Composite Pavement Design believes that it is impractical at this time to conduct experiments of a type that would produce design information pertaining to all types of composite pavement. It does appear to be appropriate, however, to perform experiments in which composite pavements as a class are compared with conventional pavements and from which a design of composite pavement whose performance is equivalent to that of a conventional pavement may be deduced. If these experiments show that there are advantages to composite pavements, further experimentation will be recommended in which other specific design variations may be studied.

The specific objective of the overall program recommended by the Committee at this time is as follows:

> To determine, through a series of experiments over different soils and in different environments, various designs for composite pavements that may be expected to exhibit the same performance as specified designs of conventional rigid and flexible pavements. When such information as to equivalent design is available, future choice of pavement type should be based on cost considerations.

The Committee proposes that the design variables in each experiment consist of (a) base thickness and (b) surface thickness. If more than one type of base or surface is desired, either two experiments may be conducted or a more elaborate single experiment may be set up.

The following recommendations are advanced for the design of the experimental sections:

1. Uniform foundations are essential for obtaining comparison of performance between the various composite and the control sections. Special care in the selection of sites and in the subsequent construction process will be necessary to assure uniformity of support.
2. The length of the test sections should be at least 600 ft and preferably less than 1,000 ft.
3. After selecting the type of base to be used, a design which will provide a pavement structure capable of performance equivalent to an adjacent conventional pavement should be made. Thickness of base and surface for an equivalent design is thus established.

In HRB Correlation Circular 473, five suggested schemes for arrangement of experimental pavements are illustrated. Choice of the experimental designs is based on two levels of each variable; that is, two base thicknesses and two base types. The choice of actual thicknesses, types, strength of materials, etc., should be determined by the agency that will conduct the experiment.

State highway departments interested in building experimental composite pavements can be assured of the full cooperation of the Bureau of Public Roads in planning and financing such projects.

Discussion of Experimental Composite Pavements

JOHN M. GRIFFITH, Director of Research and Development, The Asphalt Institute, College Park, Md.

•FOR many years, New York City has used a portland cement concrete base, an asphalt concrete binder course and a sheet asphalt surface. Other cities have used somewhat similar designs. Highway departments have constructed many thousands of miles of composite pavements through stage construction techniques. When a portland cement concrete pavement becomes unserviceable, it is normally resurfaced with asphalt concrete.

In this latter example, however, the asphalt surfacing was placed primarily as a rehabilitation measure, to restore a satisfactory riding surface. It was not until a research program was initiated by the Corps of Engineers on airfield pavements that engineers began to realize that the load-supporting capacity of a pavement was substantially increased by means of these asphalt pavement overlays. Special field tests conducted at Lockbourne AFB, at Sharonville, Ohio, and elsewhere amply demonstrated this increase in load-supporting capacity.

As all highway design engineers know, the basic concepts used in the design of flexible and rigid types of pavement vary in a marked degree. It is likely that neither design concept will be fully adequate for the design of composite pavements. Yet, there are many who consider a composite type of pavement to be the most suitable for many situations. To serve these needs, there is a definite need for the development of adequate design procedures through construction of experimental sections so that the composite type of pavement can be adequately and economically designed.

It seems likely that mix design for the experimental sections for both the asphalt concrete and portland cement concrete used in a composite pavement may require some modifications from conventional practice. In all probability, a somewhat leaner portland cement concrete mix placed under less restrictive controls would be adequate. A somewhat harder grade of asphalt cement might be used for the asphalt concrete surface because deflections would be somewhat less than for conventional flexible pavements. These factors all serve to underline a need for the development of design procedures and for field verification of these procedures if the composite pavement is to be designed on an engineering basis.

The Corps of Engineers has developed and verified design procedures for composite airfield pavements. Recently, the Corps has proposed a modification of these procedures for use in highway design. It must be recognized, however, that these procedures are based primarily on wheel loadings of far greater magnitude than those encountered on highways. The AASHO Road Test has indicated that there are interactions between load and design. Therefore, design procedures based on extremely heavy airfield type loadings quite likely will require some modification for use on highways. Nevertheless, the Corps procedures afford a good "basing point" for the development and verification of design procedures for composite pavements for highways. Requirements indicated by the Corps procedures should first be compared with existing highway composite pavements of known performance. Studies of composite pavements now under way in England and Germany, as well as composite pavement sections constructed in the Illinois rehabilitation of the AASHO Road Test sections, will be most helpful in this respect. These comparisons should provide indications as to modifications in design procedures that may be required. Following these comparisons, a limited number of field test sections may then be required to provide verification of such modifications as may be indicated.

Finally, there is the matter of economics which may have a substantial bearing upon the acceptability and general use of the composite type of pavement. Performance vs cost studies must be made for such an evaluation. These studies, however, must await the development and verification of adequate design procedures and the accumulation of a reasonable quantity of in-service performance data.

In summary, it would seem that current needs are to compare design procedures proposed by the Corps of Engineers with performance of existing composite pavements. Following these comparison studies, there probably will be a need for additional field tests of experimental sections to verify such modifications in design procedure as may be indicated. Finally, cost vs performance studies must be made as a basis for establishing the position of this type of pavement in the overall highway program.

Past Performance of Composite Pavements

PETER SMITH, SR., Materials Engineer, Materials and Research Division, Department of Highways, Ontario

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

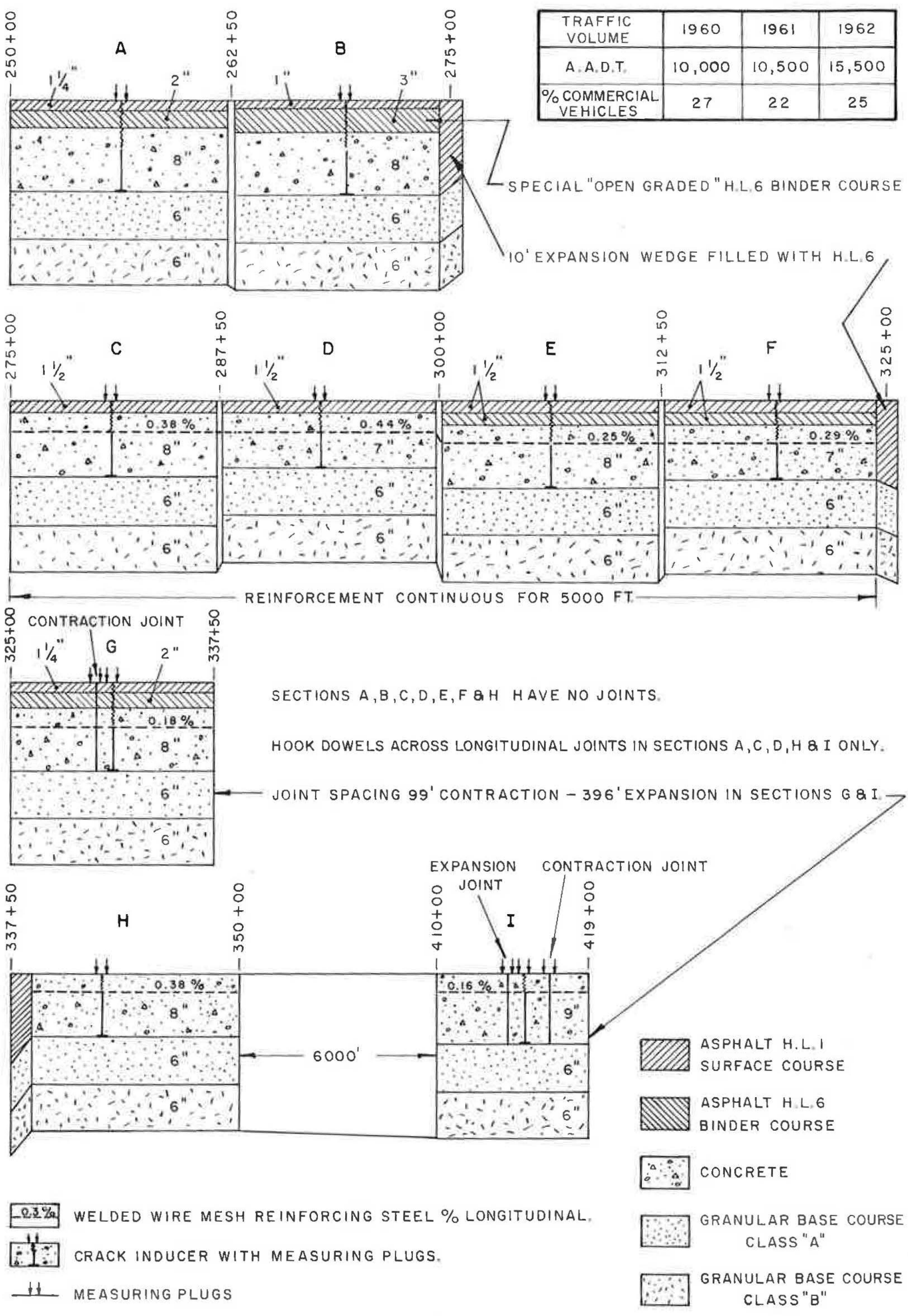


Figure 1. Design features of different sections, experimental composite pavement.

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.

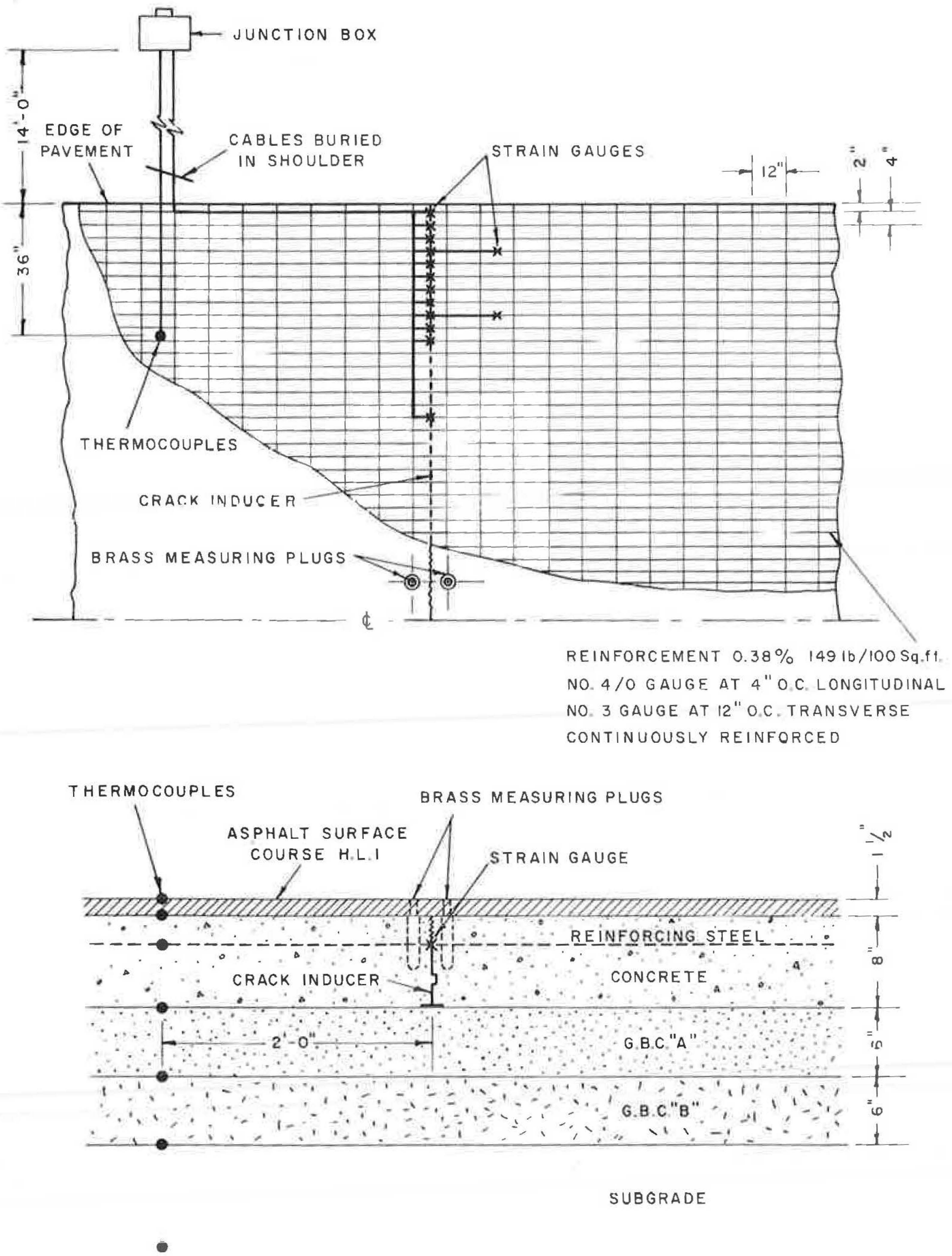


Figure 2. Instrumentation, test section C.

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

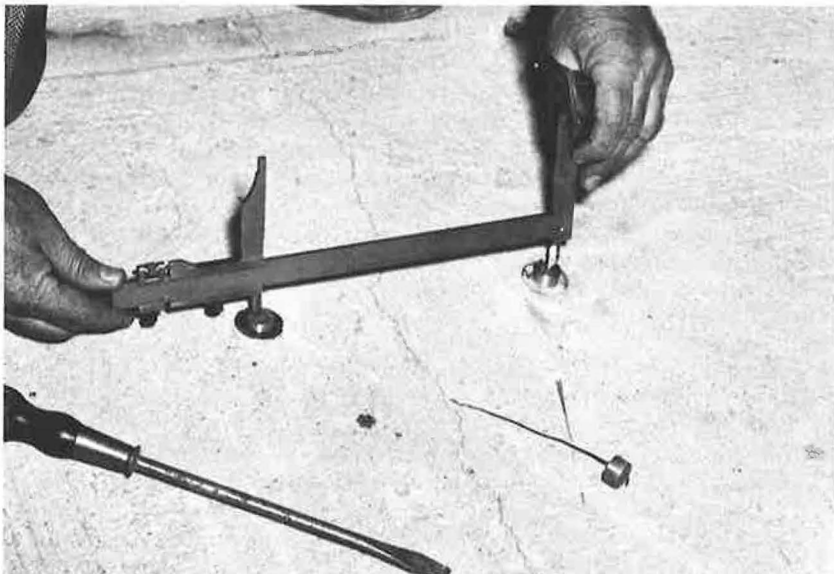


Figure 4. Width of induced crack being measured at brass plug gage points with vernier calipers.

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.

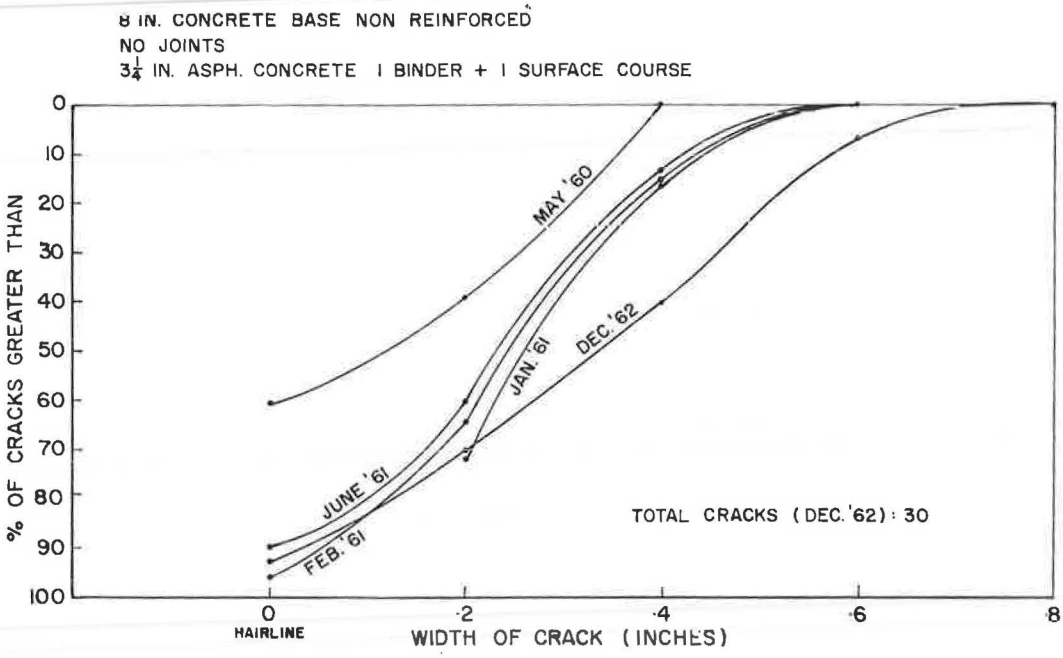


Figure 6. Section A crack width distributions.

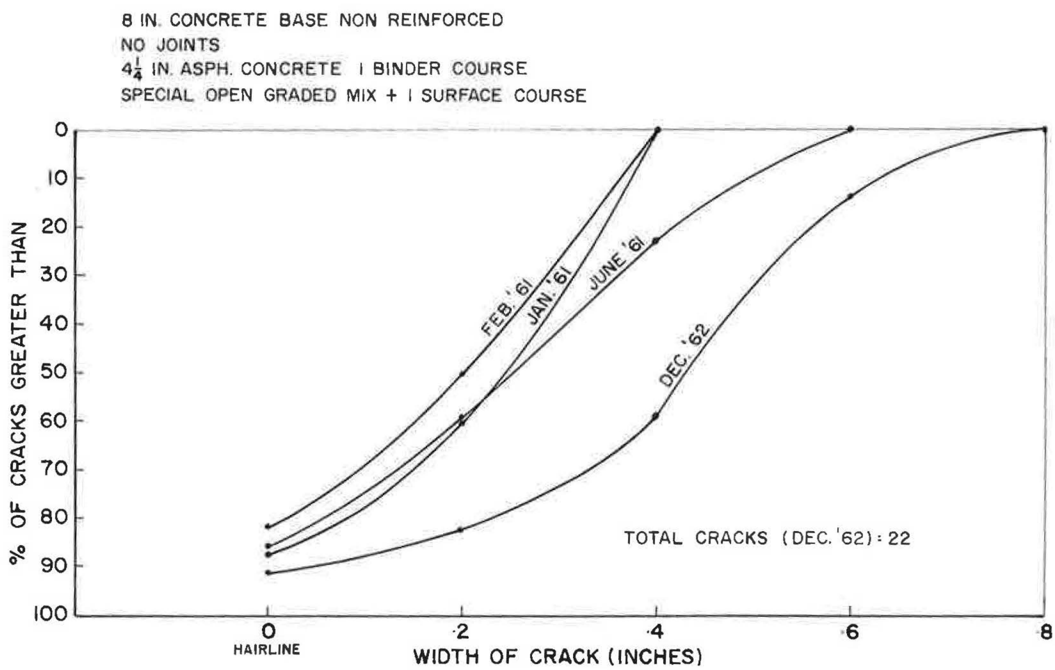


Figure 7. Section B crack width distributions.

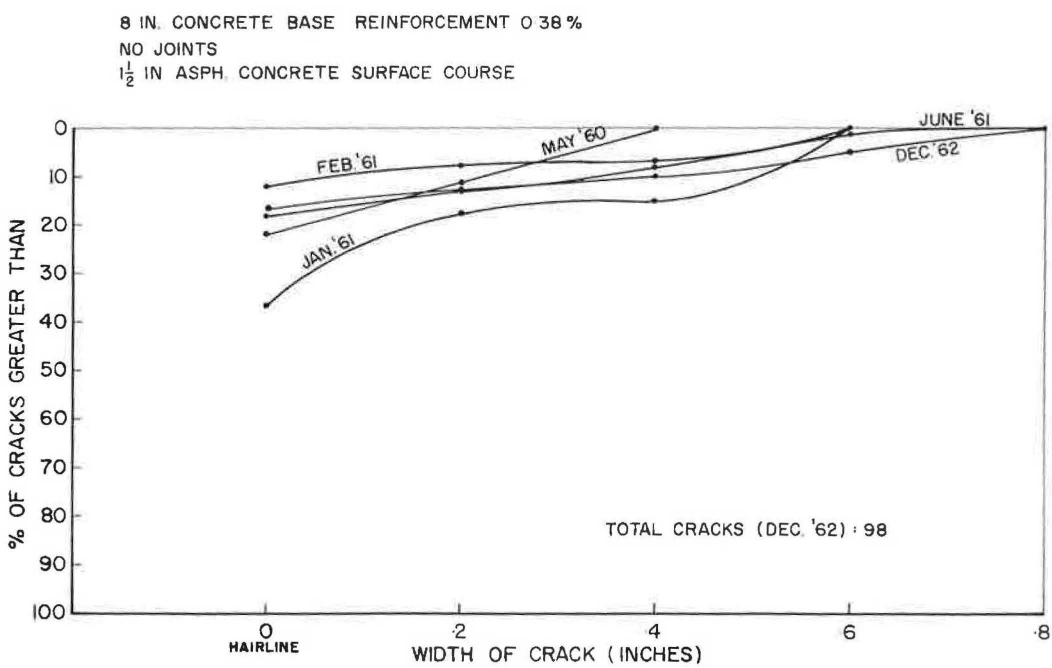


Figure 8. Section C crack width distributions.

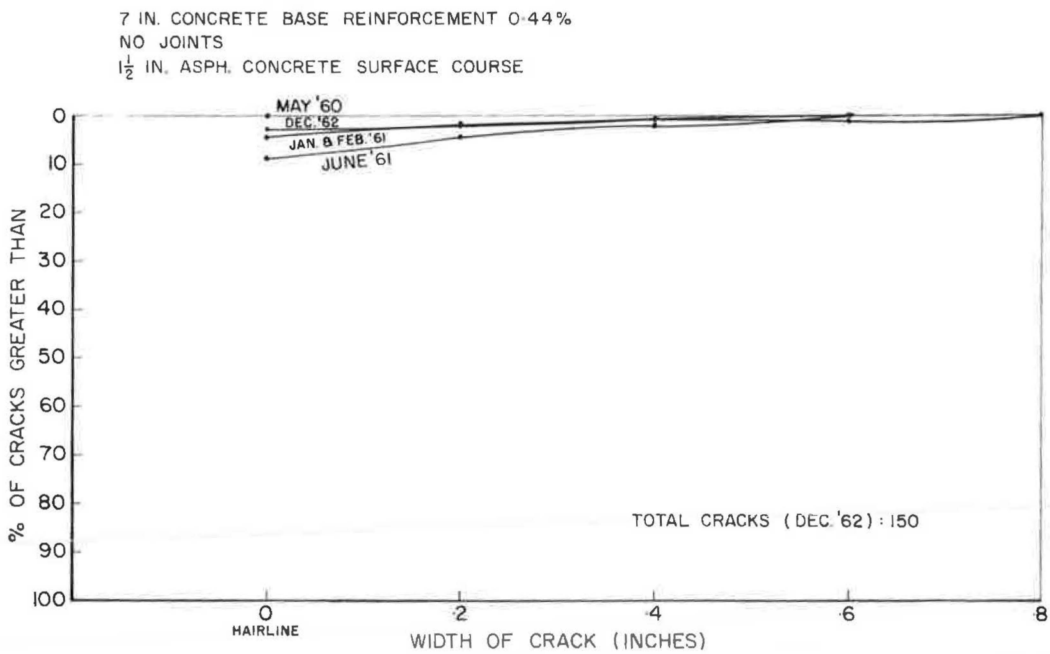


Figure 9. Section D crack width distributions.

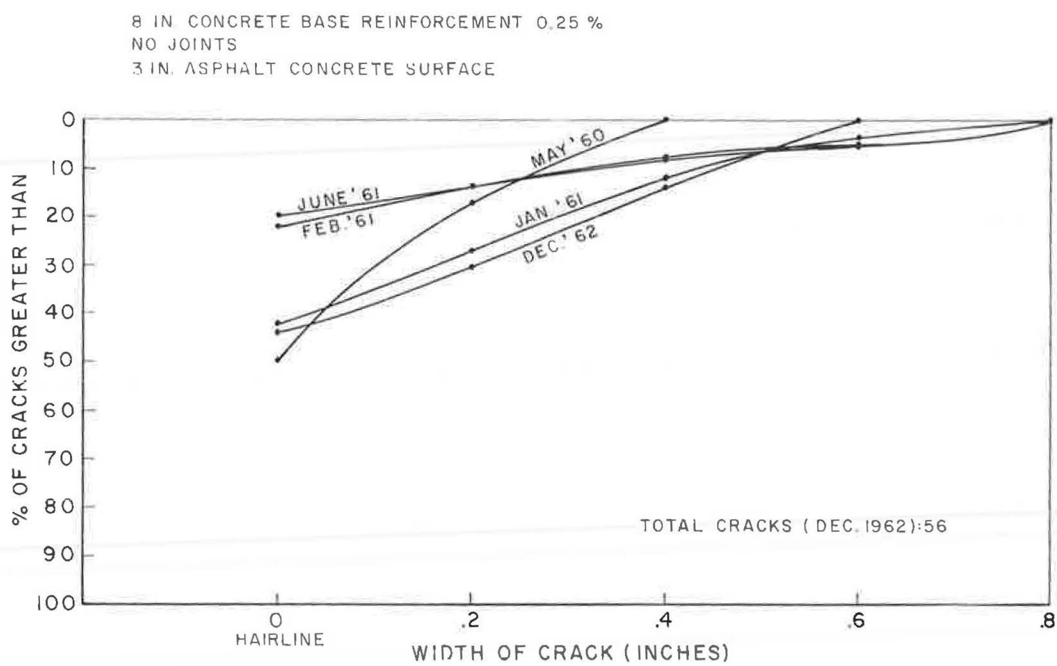


Figure 10. Section E crack width distributions.

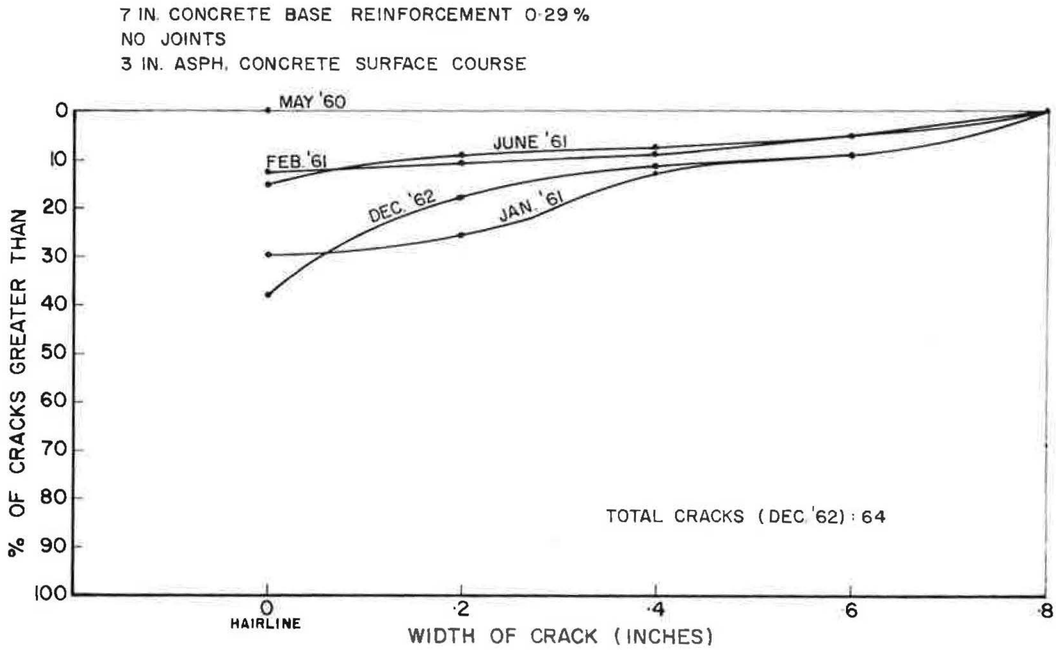


Figure 11. Section F crack width distributions.

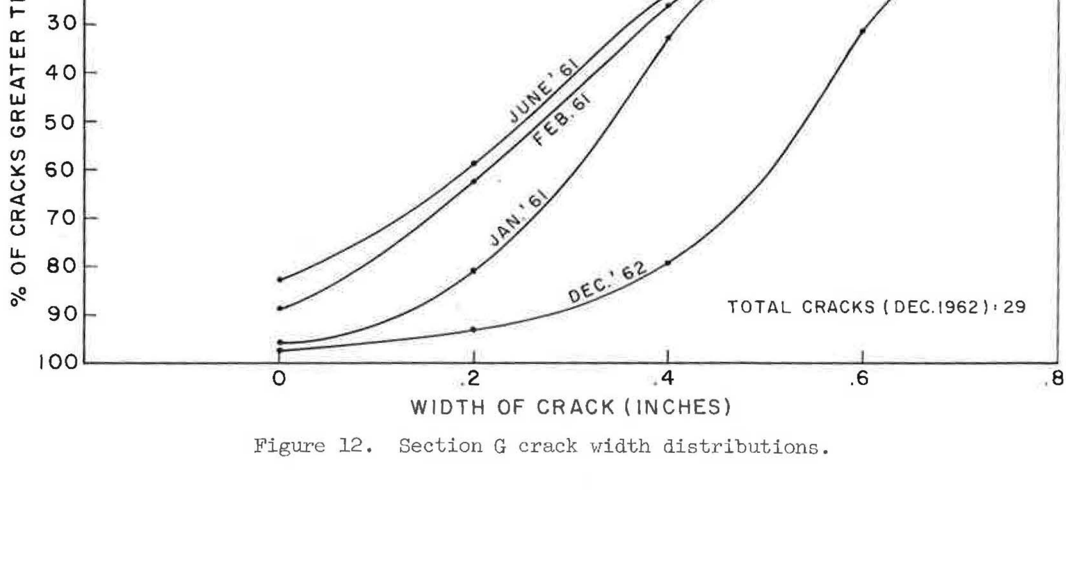


Figure 12. Section G crack width distributions.

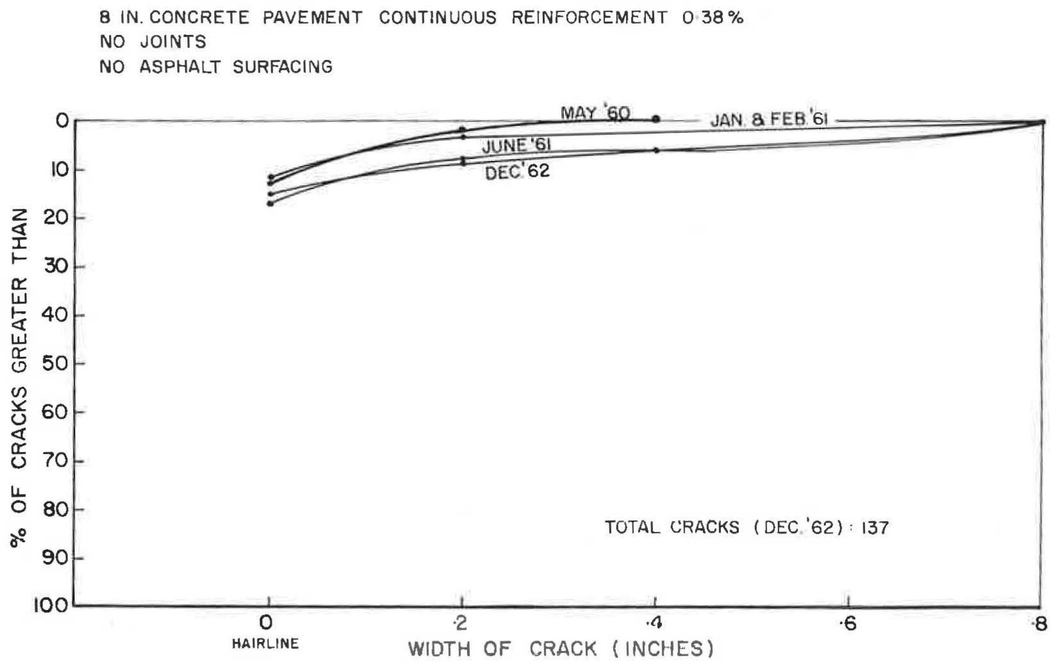


Figure 13. Section H crack width distributions.

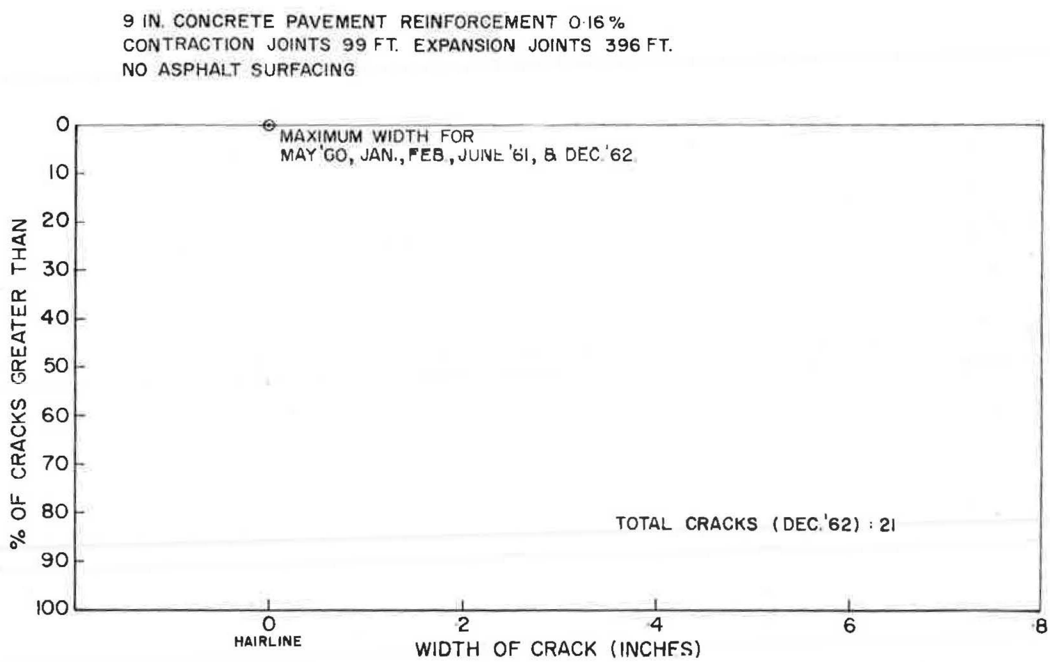
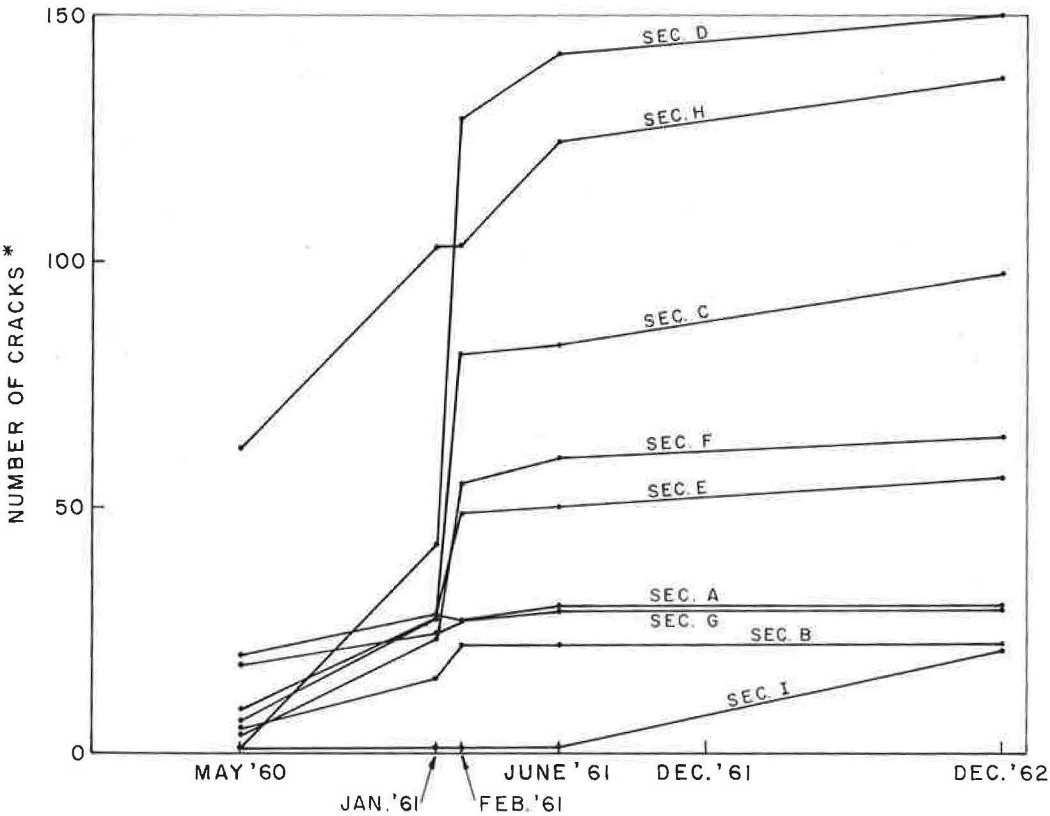


Figure 14. Section I crack width distributions.



* CRACKS FIGURED IN HALF WIDTH (12 FEET)

Figure 15. Incidence of cracking during first three years.

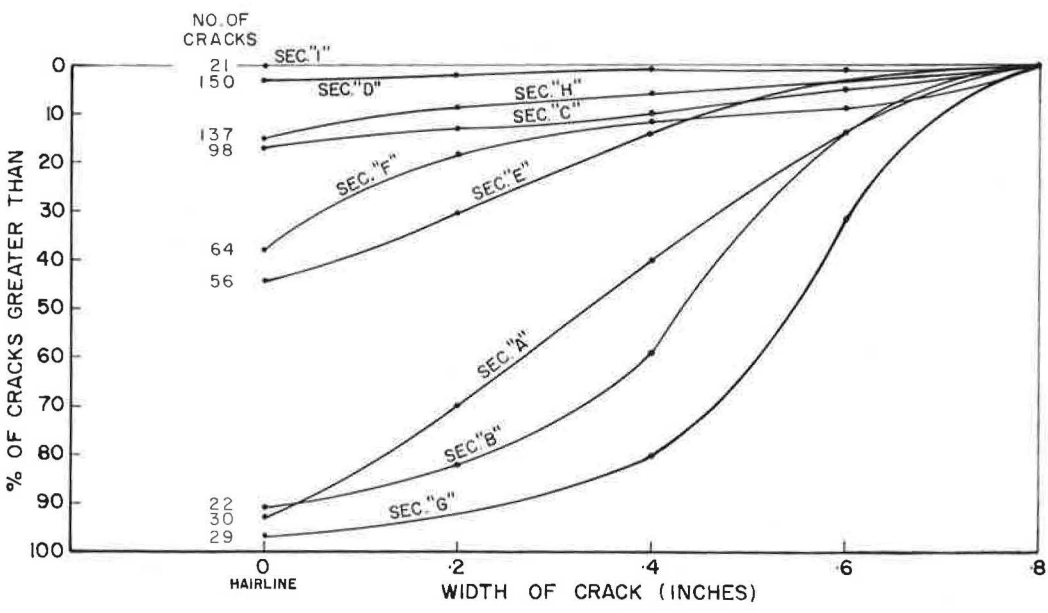


Figure 16. Crack width distributions after three years.

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 $1\frac{1}{2}$ 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 $1\frac{1}{2}$ 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 $3\frac{1}{4}$ 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.

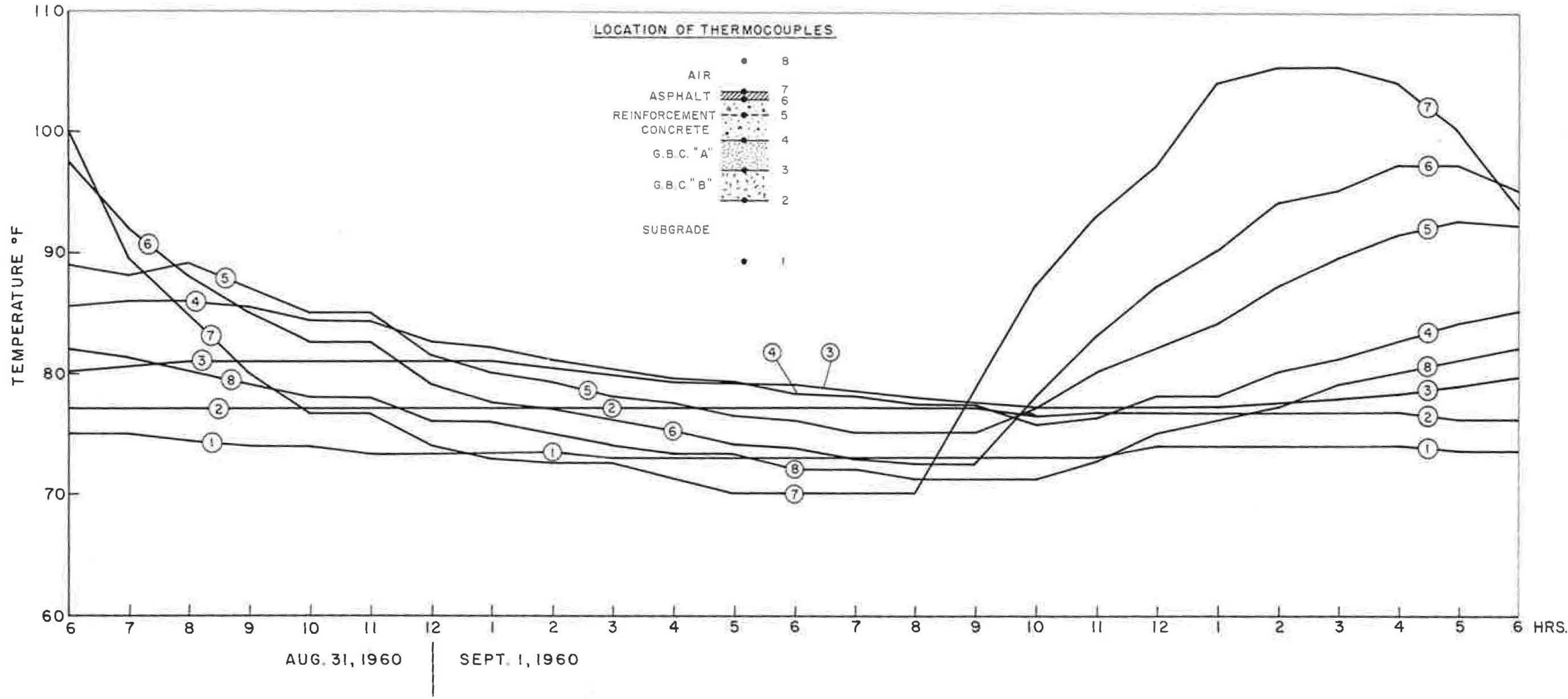


Figure 17. Typical 24-hr summer temperature cycle for concrete base, asphalt-surfaced composite pavement (Section C).

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

This report is based on design studies and observations of pavement performance made by the Materials and Research Division, Department of Highways, Ontario (A. Rutka, Materials and Research Engineer, H. W. Adcock, Assistant Deputy Minister, Engineering). Acknowledgment is made of the valuable contribution of The Steel Company of Canada through J. Hartley, of Avro Aircraft Ltd., through J. Wootten in the instrumentation of the experimental pavement at Milton, and of the main contractor, Peel Construction Ltd. and their subcontractor, Huron Construction for the concrete paving and the care taken in building experimental sections.

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.

REFERENCES

1. "Recommendations for Experimental Continuously Reinforced Concrete Pavements." HRB Circular 372 (Nov. 1958).
2. Skinner, J. A. and Martin, F. R., "Some Consideration of Airfield Pavement Design." Proc. Instit. of Civil Engineers, Part II, 4:1 (Feb. 1955).
3. Skinner, J. A., "Testing Runway Foundations and Pavements." Airport Paper No. 17, Instit. of Civil Engineers (1951).
4. Sharp, D. R., "Lean Concrete and Soil-Cement in Road and Airfield Bases." Roads and Road Construction, (Feb. -March 1960).
5. Williams and Williams, "The London-Birmingham Motorway, Luton-Dunchurch: Design and Execution." Proc. Instit. of Civil Engineers (April 1960).
6. Laing, Broadbent, and Fisher, "The London-Birmingham Motorway, Luton-Dunchurch: Construction." Proc. Instit. of Civil Engineers (April 1960).

Design Procedures for Flexible And Bituminous Overlays

WILLIAM A. GOODWIN, Highway Research Board

I. Federal Aviation Agency Procedures

•THIS design procedure is for bituminous overlays rather than composite pavements; however, it is pertinent to the design of composite pavements.

The design of airport pavement overlays currently in use by the Federal Aviation Agency is well illustrated in their brochure on "Airport Paving." This brochure, published in November 1962, is a reprint of the Civil Aeronautics Administration's design procedure for airport paving. The following comments relative to overlay design have been extracted for inclusion in this summary report. Design procedures are suggested for flexible, bituminous, and concrete overlays in the FAA manual, however, the concrete overlays are not herein discussed due to their limited use. The manual contains several design examples. Figure 1 contains typical sections of overlay pavements.

Preliminary design information is required before the actual design can be made, including the following:

1. Determination of the soil group and subgrade class of soil underlying the existing pavement based on the FAA classification procedure.
2. Determination of the actual thickness of each layer of the existing pavement.
3. Based on the type overlay contemplated, a determination must be made of the pavement thickness required for the wheel loading and subgrade class under consideration.

After the above information has been assembled, the design may take one of several forms depending on the conditions under consideration. If it is a flexible or bituminous overlay to be applied to either a flexible or rigid pavement, certain general criteria must be followed.

1. Subbase courses will not be used in pavement overlays.
2. Nonbituminous base courses shall consist of crushed material.
3. A portion of the thickness of a bituminous overlay may consist of penetration macadam.
4. Bituminous overlays shall have a minimum thickness of 3 in.
5. Bituminous overlays greater than 3 in. thick may be planned for stage construction.
6. All materials must comply with FAA's "Standard Specifications for Construction of Airports" for base course of surface course.

With a foreknowledge of the above six criteria, the design may be made by one of two procedures.

FLEXIBLE AND BITUMINOUS OVERLAYS ON EXISTING FLEXIBLE PAVEMENTS

To use the design procedure involving flexible and bituminous overlays on flexible pavements, the basic curves shown in Figures 2 and 3 are used to determine a total pavement thickness to accommodate the desired wheel load. The difference between the existing total pavement thickness and the required total pavement represents the unadjusted thickness of overlay. Adjustment to the overlay thickness is made on the basis of the character and condition of the existing surface and the type of overlay base, as follows:

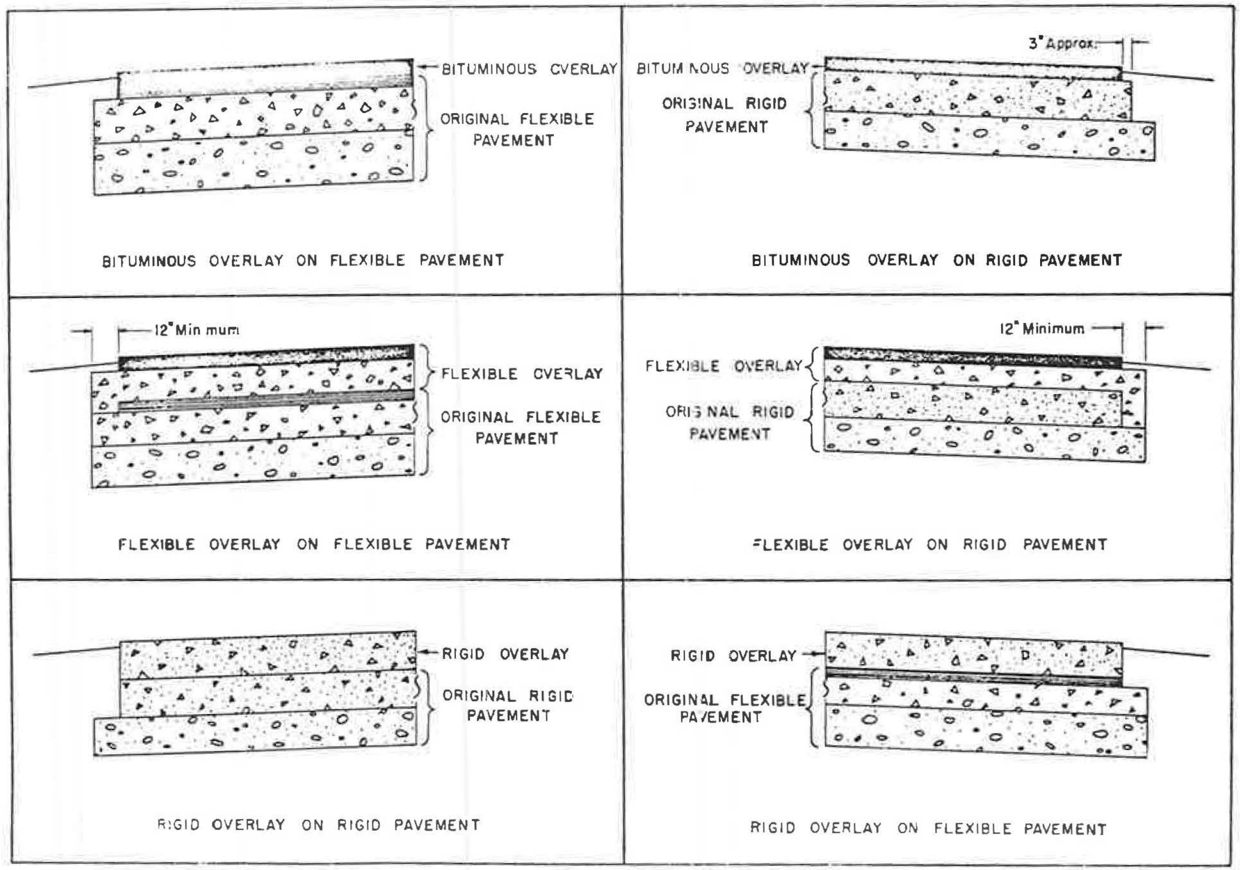


Figure 1. Typical sections of overlay pavements (FAA).

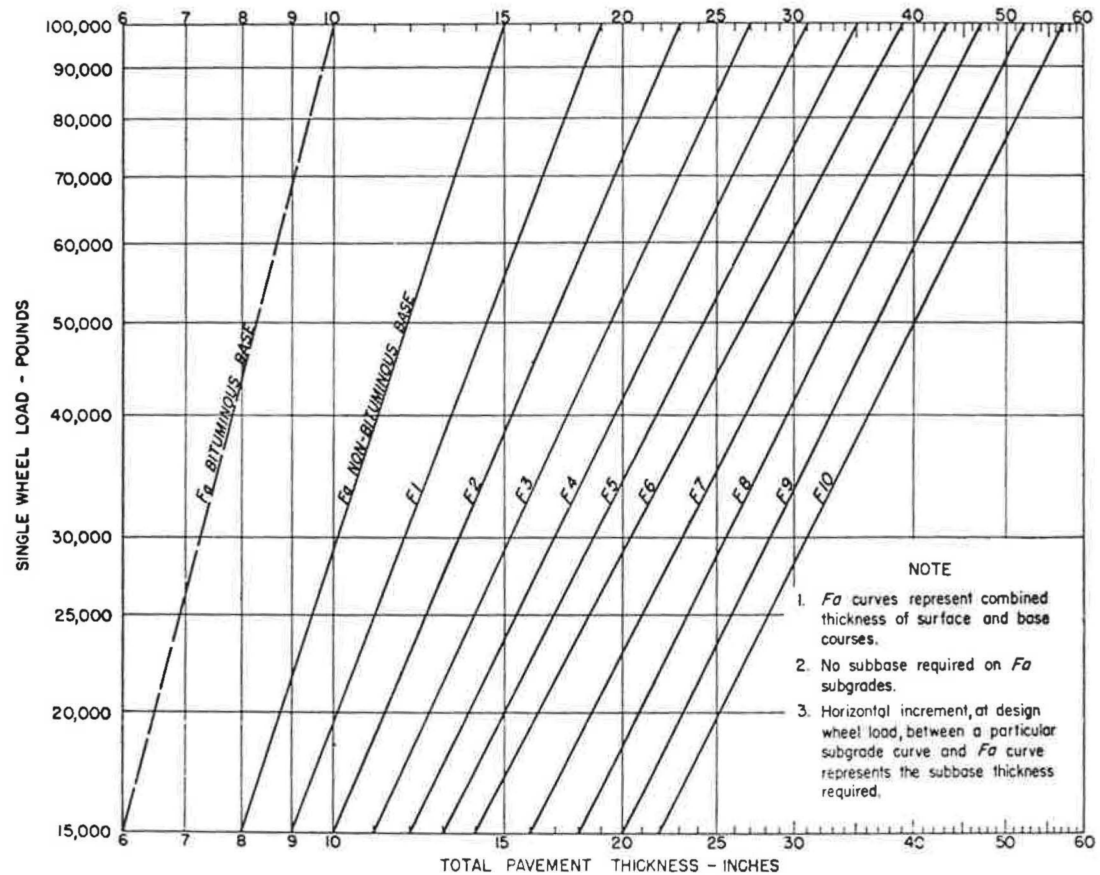


Figure 2. Design curves for flexible pavements, taxiways, aprons, and runway ends(FAA).

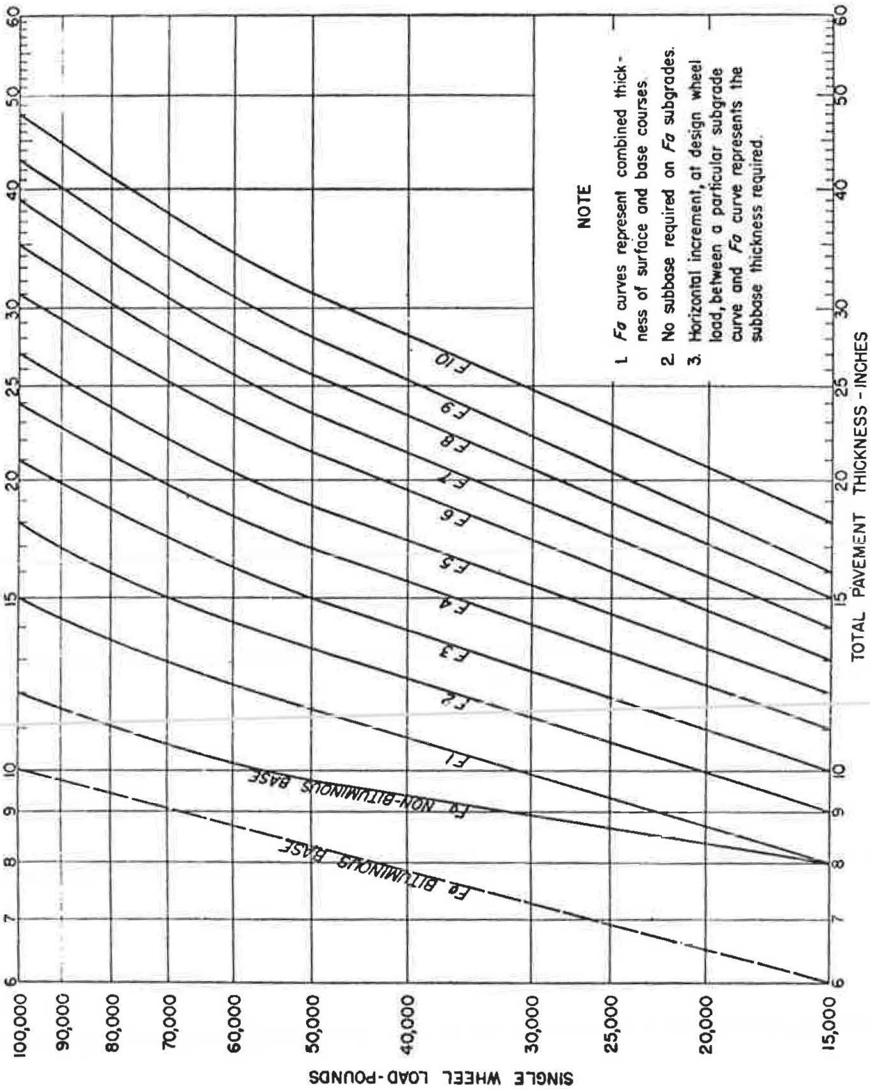


Figure 3. Design curves for flexible pavements—noncritical runway areas (FAA).

1. An existing dense-graded plant-mix bituminous surface, in sound condition, may be evaluated for base course purposes on the basis that each inch of such surface is equivalent to 1½ in. of base course provided the entire overlay will consist of bituminous concrete.

2. Under all other conditions, the existing pavement will be considered, inch for inch, as base course.

3. If a bituminous base of the type specified by FAA as Item P-201 is to be used, a thickness adjusted may be made on the basis of 1 in. of such base being equivalent to 1½ in. of nonbituminous base.

With regard to flexible overlays, the thickness of the nonbituminous base shall not be less than 4 in. unless the existing bituminous surface is broken to such an extent that it can be blended with the new base course material.

FLEXIBLE AND BITUMINOUS OVERLAYS ON RIGID PAVEMENTS

When a flexible or bituminous overlay is to be placed over an existing rigid pavement, the thickness of surface course is determined by the appropriate design curve. In all instances, the minimum thickness of base course is 6 in. To establish the required thickness of overlay, it is first necessary to determine the basic rigid pavement design thickness from the curves in Figure 4. This thickness is then modified by a factor F which represents the subgrade and subbase conditions under the existing concrete. Table 1 shows values for the factor F as related to the FAA subgrade classification system.

TABLE 1

FLEXIBLE AND BITUMINOUS OVERLAYS ON RIGID PAVEMENTS

Existing Subgrade Class	Value of F when Subbase Conforms to Requirements for Class of Subgrade:				
	Ra[1]	Rb	Rc	Rd	Re
Ra	0.80	–	–	–	–
Rb	0.90	0.80	–	–	–
Rc	0.94	0.90	0.80	–	–
Rd	0.98	0.94	0.90	0.80	–
Re	1.00	0.98	0.94	0.90	0.80

[1] Apply when no subbase has been provided.

The appropriate value for F is found in the column in Table 1 that represents the subgrade class that would have to prevail for the existing thickness of subbase to be adequate for the design wheel load. Having determined the value of F, the overlay thickness can be computed from one of the following formulas.

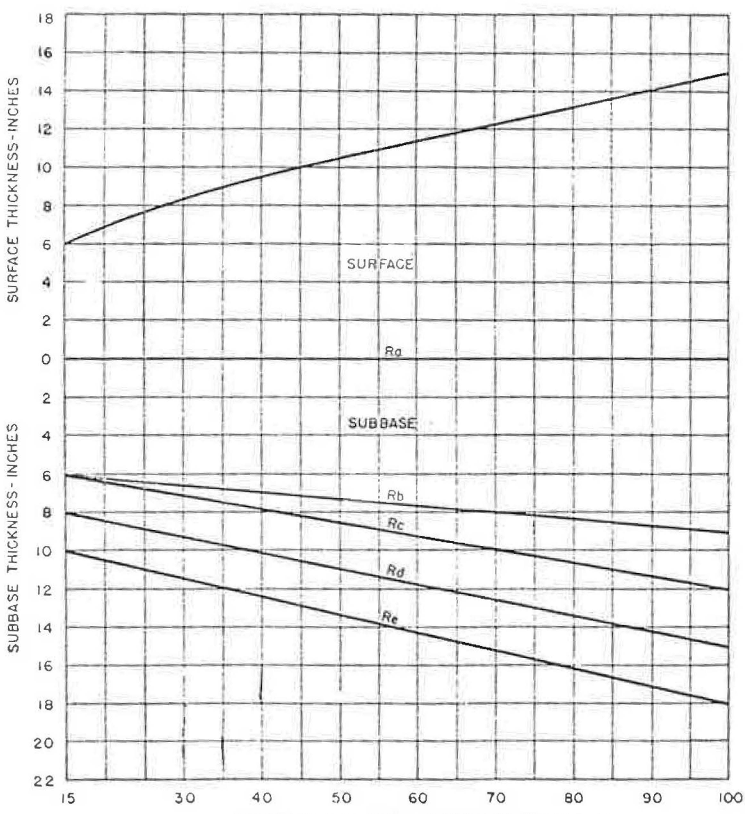


Figure 4. Design curves for rigid pavements, taxiways, aprons, and runway ends (FAA).

For flexible overlays:

$$t_f = 2.5\,(Fh - h_e) \tag{1}$$

in which

t_f = required thickness of flexible overlay;
F = factor which varies with subgrade class;
h = required thickness of an equivalent single slab placed on the subgrade or subbase; and
h_e = thickness of existing slab.

For bituminous overlays:

$$t_b = \frac{t_f}{1.5} \tag{2}$$

in which

t_b = required thickness of bituminous overlay; and
t_f = required thickness of flexible overlay.

In both the flexible and bituminous overlays, a minimum thickness of 6 in. is required for a nonbituminous base course and 3 in. for a bituminous overlay.

II. The Asphalt Institute Procedure

•THIS design procedure is for overlay pavements rather than composite pavements; however, it is pertinent to the design of composite pavements.

The following excerpts are from the manual on "Thickness Design of Asphalt Pavement Structures for Highways and Streets," as published by the Asphalt Institute in 1962.

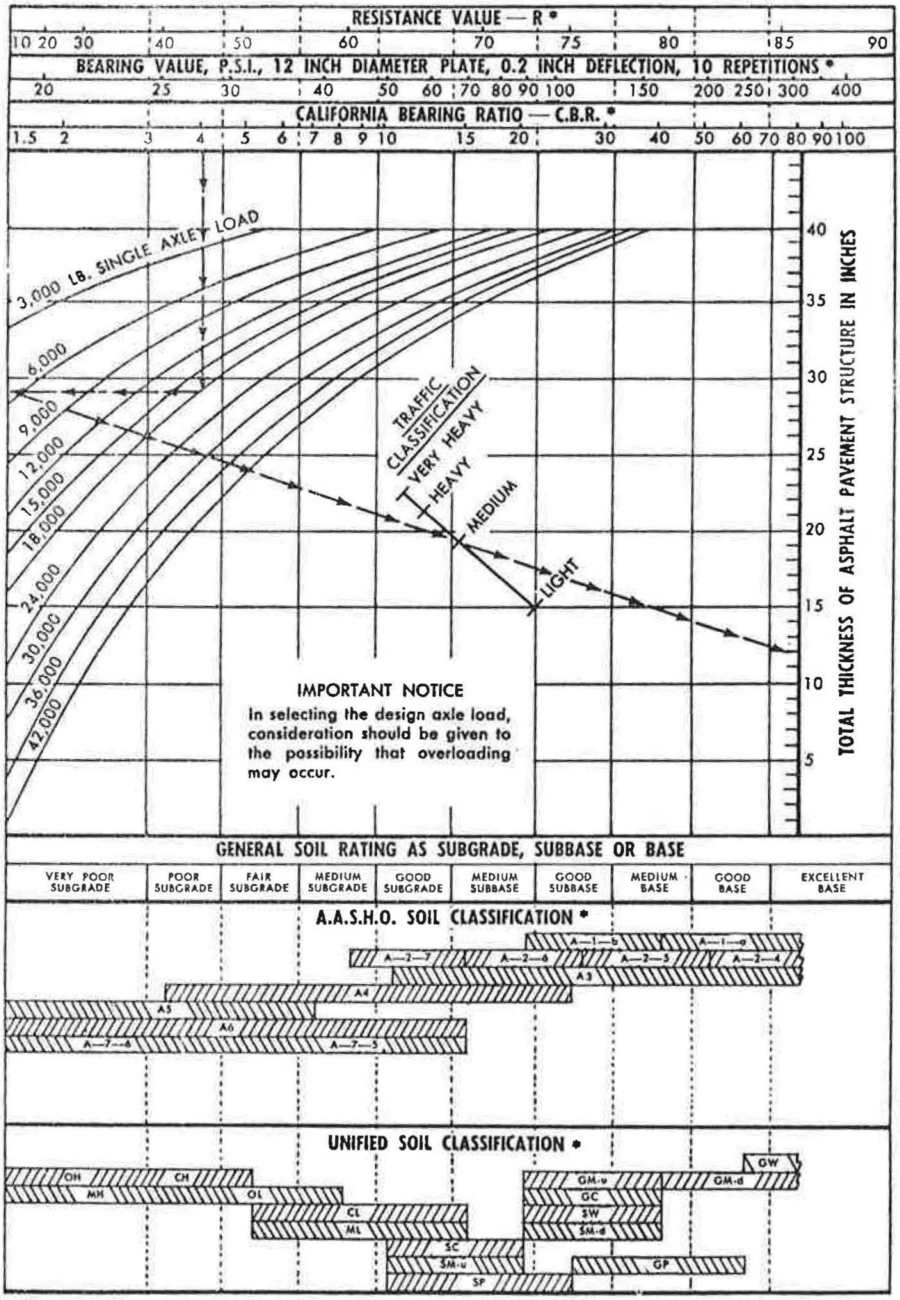
In this design procedure, existing pavements may be improved by overlaying with asphalt pavement or with a combination of asphalt pavement and asphalt base. Under certain conditions a high-quality, non-asphalt base may be included in the overlay. Such overlays may be considered in two categories:

1. Overlays to provide smooth, skid and water-resistant surfaces or to accomplish improvements in grade and cross-section.
2. Overlays to strengthen existing pavements to accommodate heavier loads or increased traffic.

In the first category, the overlay is usually constructed entirely of asphalt concrete, and the design thickness is determined by factors other than a required increase in pavement strength. When pavement strengthening is required, a thickness design procedure is warranted.

STRENGTHENING EXISTING ASPHALT PAVEMENTS

If an existing asphalt pavement structure requires strengthening to support heavier loads and higher volumes of traffic, the needed thickness of asphalt overlay may be arrived at by first establishing classification and strength characteristics of the existing pavement layers along with their thicknesses. After this information has been obtained, the new requirements of load and traffic are used to enter the design chart (Fig. 5) for the determination of required total thickness. The difference between existing thickness and required total thickness is the required thickness of overlay. In using the design chart, 4 in. of asphalt pavement surface is to be included in the total thickness. For any high-quality asphalt layer in the existing pavement, it may be considered that 1 in. of the asphalt layer is equivalent to 2 in. of non-asphalt base material.



In this chart the "R-Value" is only a part of the full "R-Value" design method as originated and used in California. The latter integrates an elaborate traffic census and analysis with the R-Value bearing test for materials proper, in which a traffic growth factor as well as types of vehicles and frequency is incorporated. It also includes a "cohesiometer" factor for the bituminous layer or layers, which is essentially a beam or panel action factor. For use in other areas as an integrated design method, the traffic growth factor should be modified to fit local conditions. For soils, an "exudation pressure" measurement and an "expansion pressure" measurement are used in the analysis of thickness requirements.*

Figure 5. Thickness requirements for asphalt pavement structures (Asphalt Institute).

STRENGTHENING EXISTING RIGID PAVEMENTS

The Asphalt Institute recognizes the design procedures established by the Corps of Engineers for strengthening rigid airfield pavements with asphalt overlays, but expresses caution for such use for highway pavements; that is, until the extent to which the design procedures are applicable to highways has been established. As an interim procedure, the design method currently in use by the Institute for the design of asphalt pavement structures is recommended.

To use the interim design procedure, it is necessary to assign certain equivalency factors (n) for evaluating the strength contribution of the existing rigid pavement. Based on Corps of Engineers tests, n factors have been selected as follows:

1. n = 1.5 to 2.0 for stable, non-pumping rigid pavement with some cracks but with no pieces smaller than about 1 sq yd in area.
2. n = 2.0 for nonreinforced, stable, non-pumping rigid pavements.
3. n = 2.5 for reinforced, stable, non-pumping rigid pavements.

Preliminary to the design, the following factors pertinent to the existing rigid pavement must be evaluated:

1. Strength of subgrade in terms of one of the systems shown on the design chart (Fig. 4).
2. Depth and strength of in-place granular base, if any.
3. Condition and construction features of existing rigid pavement for establishment of n factor.

With the preliminary design information, including the new loading and traffic volume for which the overlay is to be designed, an equivalent thickness of asphalt pavement structure may be determined from Figure 4. If a granular base is beneath the existing rigid pavement, its strength characteristics will determine if it is suitable to function as a subbase. The strength equivalency of the existing rigid pavement is then computed, and the required thickness of asphalt overlay may be established.

In case of an asphalt overlay over an existing rigid pavement having rocking or unstable slabs, slab support should be restored by undersealing.

In either of the above design procedures an overlay thickness of 6 in. or less should be constructed entirely of asphalt base and surface. If overlay thicknesses greater than 6 in. are required, a non-asphalt granular base course may be included provided it can be adequately drained. When drainage of the layer is questionable, the full depth of overlay should be asphalt construction.

California Division of Highways Procedure For Designing Composite Pavements

ERNEST ZUBE, Supervising Materials and Research Engineer, California Division of Highways

•IN the California method for determining the design thickness for flexible and composite type pavements, the cohesion C or tensile strength of the various layers making up the structural section is evaluated. The design values are established from a large number of cohesiometer tests, correlation with test track data and correlation with experience on highways.

The basic design formula is

$$T = \frac{0.095\,(TI)\,(90 - R)}{\sqrt[5]{C}} \qquad (1)$$

in which

- T = cover required over soil in question, in in.;
- TI = traffic index = $1.35\,(EWL)^{0.11}$;
- R = resistance value of the soil in question as determined by the stabilometer;
- C = combined cohesiometer values of the proposed overlying layers; and
- EWL = 5,000-lb equivalent wheel loads in one direction for a 10-yr design period.

Of the three variables (traffic, R-value and C-value) embodied in current design method for layer thickness determination, only the C-value, or cohesiometer value, relates to the beam strength of the surfacing and base.

Figure 1 shows the deformation of a structural section under an excessive load. Resistance to such deformation is supplied by interparticle friction (measured in terms of R-value), the tensile strength of the structural section, and the confining force due to the weight of material surrounding the loaded area. If the deforming forces overcome interparticle friction, the tensile strength must be sufficient to prevent the indicated lateral displacement. Since cohesion measures the ability of a material to resist tensile stress, its evaluation is included in the design process.

The cohesiometer test is performed on specimens 4 in. in diameter by 2½ or 3 in. high clamped to a hinged plate with a test load applied to a lever arm attached to one side of the hinged plate (Fig. 2). The cohesiometer value (C) is expressed as the breaking load in grams per inch diameter for a 3-in. specimen.

From original test track studies, it was found that the thickness of cover is proportional to $\frac{1}{\sqrt[5]{C}}$. It is very often convenient to express the total thickness of cover required in terms of gravel equivalent. The gravel equivalent is the thickness of gravel (sand, crushed stone or other granular material) required to protect the underlying material from a given load and is based on an assumed cohesion value of 100 for the granular untreated cover material. Therefore, the equation for unit gravel equivalent may be derived as follows:

Let

- Tg = thickness of gravel;
- Cg = cohesiometer value of gravel;
- Tx = thickness of other material; and
- Cx = cohesiometer value of other material.

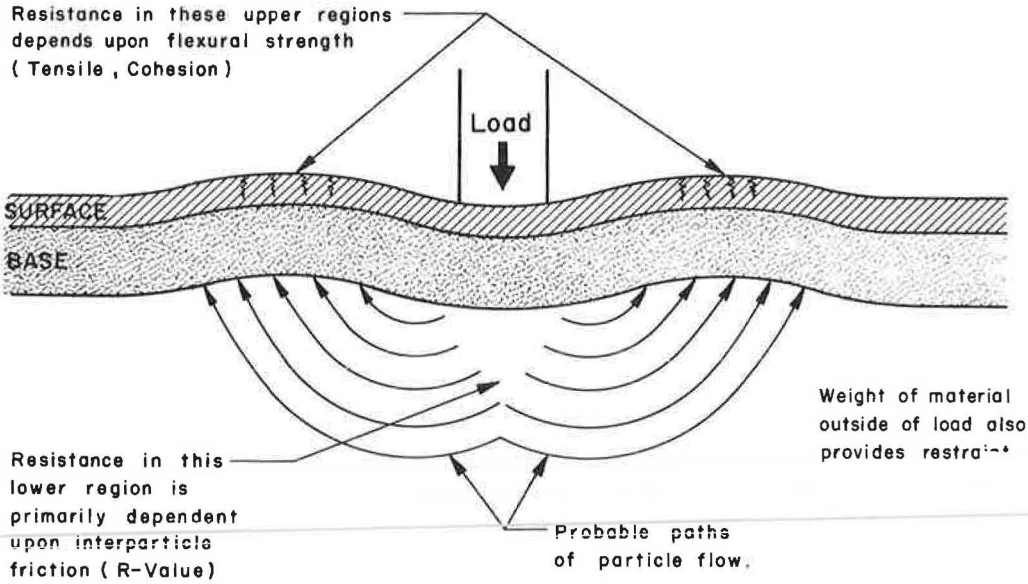


Figure 1. Schematic representation of plastic flow phenomena.

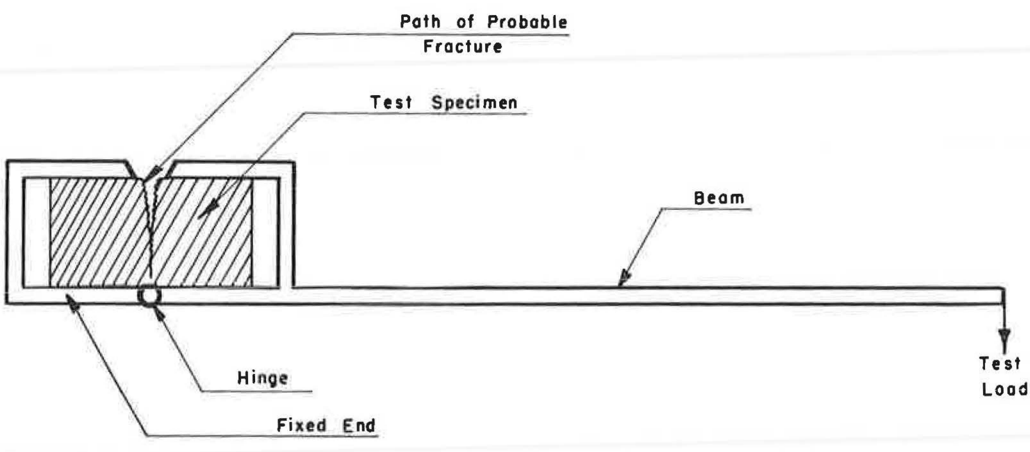


Figure 2. Schematic arrangement of apparatus used in cohesiometer test.

Then

$$\frac{Tg}{Tx} = \frac{\frac{1}{\sqrt[5]{Cg}}}{\frac{1}{\sqrt[5]{Cx}}} = \frac{\sqrt[5]{Cx}}{\sqrt[5]{Cg}} = \sqrt[5]{\frac{Cx}{Cg}} \tag{2}$$

If Tx = 1 in. and Cg = 100 (cohesiometer for untreated soils or gravel), then

$$Tg = \sqrt[5]{\frac{Cx}{100}} \tag{3}$$

Application of Eq. 3 to Class A CTB which has a cohesiometer value of 1,500 gives

$$Tg = \sqrt[5]{\frac{1,500}{100}} = 1.72 \text{ in. of gravel per inch of CTB}$$

Table 1 shows the cohesiometer values and unit gravel equivalents assigned to various layers of the structural section.

A reduction in base thickness from that required for uncemented-aggregate bases is, therefore, made when Class A or B CTB in composite pavements is used.

1. Class A CTB reduces the thickness of untreated base by 42 percent.
2. Class B CTB reduces the thickness of untreated base by 33 percent.

It should be pointed out, however, that if the reduction in thickness from an untreated base layer results in a CTB thickness of less than 6 in., it is advisable, from the construction standpoint and due to variations encountered in the construction of

TABLE 1

Layer	Cohesiometer Value	Unit Gravel Equivalent (in./in.)
Surfacing:		
Asphalt concrete (plant mixed)	400	1.32
Road-mixed asphalt surfacing	150	1.08
Base:		
Aggregate base (untreated)	100	1.32
Cement-treated base, Class A (750 psi at 7 days)	1,500	1.72
Cement-treated base, Class B (400 psi at 7 days)	750	1.50
Cement-treated base, Class C (80+ R-value)	100	1.00
Lime-treated base (80+ R-value)	100	1.00
Asphalt-treated base (plant mixed)	400	1.32
Asphalt-treated base (road mixed)	150	1.08
Subbase:		
Aggregate subbase (untreated)	100	1.00
Cement-treated subbase	100	1.00
Lime-treated subbase	100	1.00

any base, that the CTB layer be built at least 6 in. thick and preferably not less than 8 in. when used under asphalt-concrete surfacing carrying heavy traffic. The California test track indicated that cement-treated bases of less than 5 in. thickness over a saturated subgrade are subject to early breakup after exposure to a comparatively small number of truck repetitions.

A more detailed description of the California method of design is presented in Test Method No. Calif. 301.

An example of the application of cohesiometer value in the design formula follows:

A. Multilayered systems require the combining of individual cohesions to obtain an equivalent value for use in the design equation, shown as follows:

1. Assume the following structural section over a basement soil:
   (a) 4 in. of asphaltic concrete (AC),
   (b) 8 in. of Class A cement-treated base (CTB), and
   (c) 4 in. of imported subbase material (ISM).
2. Refer to Table 1 for the unit gravel equivalents (GE).
   (a) GE of 4-in. AC = 1.32 × 4 in. = 5.28 in.
   (b) GE of 8-in. CTB = 1.72 × 8 in. = 13.76 in.
   (c) GE of 4-in. ISM = 4.00 in.
   (d) Total GE for the three layers = 23.04 or 23 in.
3. Knowing the actual assumed thickness (16 in.) of the system and having calculated its gravel equivalent, the cohesiometer value is determined by the following formula:

$$\text{Coh} = \left(\frac{\text{GE}}{\text{T}}\right)^5 \times 100 = \left(\frac{23}{16}\right)^5 \times 100 = 620$$

B. For the purpose of this design problem, assume a traffic index of 10.0 (about 50 million EWL) and a design R-value for the basement soil of 20.

C. Then, using the design equation and above values for the variables, the required thickness of cover over the basement soil may be calculated from

$$\text{T} = \frac{0.095\,(\text{TI})\,(90-\text{R})}{\sqrt[5]{\text{Coh}}} = \frac{0.095\,(10)\,(90-20)}{\sqrt[5]{620}} = 18.4 \text{ in.}$$

D. Since the required thickness of 18.4 in. is greater than the assumed thickness of 16.0 in., the proposed design is not adequate.

E. Therefore, assuming 8 in. of ISM (instead of 4 in.) and retaining the 4 in. of AC and 8 in. of Class A CTB (total cover = 20 in.), the design requirement is recalculated.

1. Combined cohesiometer value is recalculated using the preceding method; the value now becomes 450.
2. By substituting this value in the design equation, a required thickness of 19.7 in. is determined.
3. Since the assumed cover thickness is 20 in., the design calling for 4-in. AC, 8-in. CTB Class A and 8-in. ISM will be satisfactory over the existing basement soil.
4. In actual practice there are charts, tables and special slide rules that greatly facilitate these calculations.

Composite Pavement Design for Roads and Streets

T. B. PRINGLE, Chief, Civil Engineering Branch, Engineering Division, Military Construction Directorate, Office, Chief of Engineers, U. S. Army; and
F. M. MELLINGER, Director, Ohio River Division Laboratories, U. S. Army, Corps of Engineers, Cincinnati

This paper contains a brief review of the Corps of Engineers' development of one type of composite pavement design procedure for airfield pavements. This development is used as a basis for presenting a procedure for the design of composite pavement for roads and streets. Sufficient information is given for the direct application of this procedure. Adequate references are provided for more detailed study of this development and design procedure.

•THE method of composite pavement design given in this paper was originally developed for strengthening plain concrete airfield pavements with non-rigid overlays. The non-rigid overlays consist either of bituminous concrete only or of a high-quality base course surfaced with bituminous concrete. The development of this design method for composite airfield pavements (1) extended over the 10-yr period from 1945 to 1955. This method in its present form has been in use by the Corps of Engineers for the design and construction of military airfield pavements since 1955. The method was adapted in June 1961 to the design of composite pavements for roads and streets (2).

DESIGN DEVELOPMENT

Composite Pavements for Airfields

The background of the full-scale traffic testing and the analysis of the results leading to the design method for composite pavements is given by Mellinger and Sale (1). A knowledge of the manner in which failure occurs and the interaction of the components leading to failure is basic to this or any other method of design. Two conditions are involved in the design of composite pavements: (a) that the non-rigid overlay be of sufficient quality that the bituminous concrete or base materials, if present, will not fail providing the concrete base pavement gives adequate support; and (b) that failure starts in the concrete base pavement and progresses to the surface of the overlay. Failure, with this type of interaction of the overlay and base pavement, is defined as the condition where visible transient deflection of the overlay surface under the design traffic loading is in the order of 0.75 to 1.00 in., and permanent deflection or rutting of the surface is in the order of 1.00 to 1.50 in.

Therefore, the composite pavement design is related to the design procedure for plain concrete pavement for various degrees of failure. The design procedure for plain concrete pavements is given by Mellinger (3). Two advantages are obtained by placing a non-rigid overlay on a plain concrete pavement: (a) the rate of cracking or break-up of the base pavement is reduced; and (b) a greater degree of break-up or cracking of the base pavement can be permitted than would be feasible without the overlay.

Figure 1 shows three conditions of the concrete base pavement that resulted from traffic loading. The traffic was continued in the first case (a) after failure of the overlay surface, and stopped in the second case (b) at incipient failure; that is, when transient deflections of the overlay surface were in the order of 0.75 in. or less. The last case (c) shows the condition of the base pavement when traffic was not sufficient to pro-

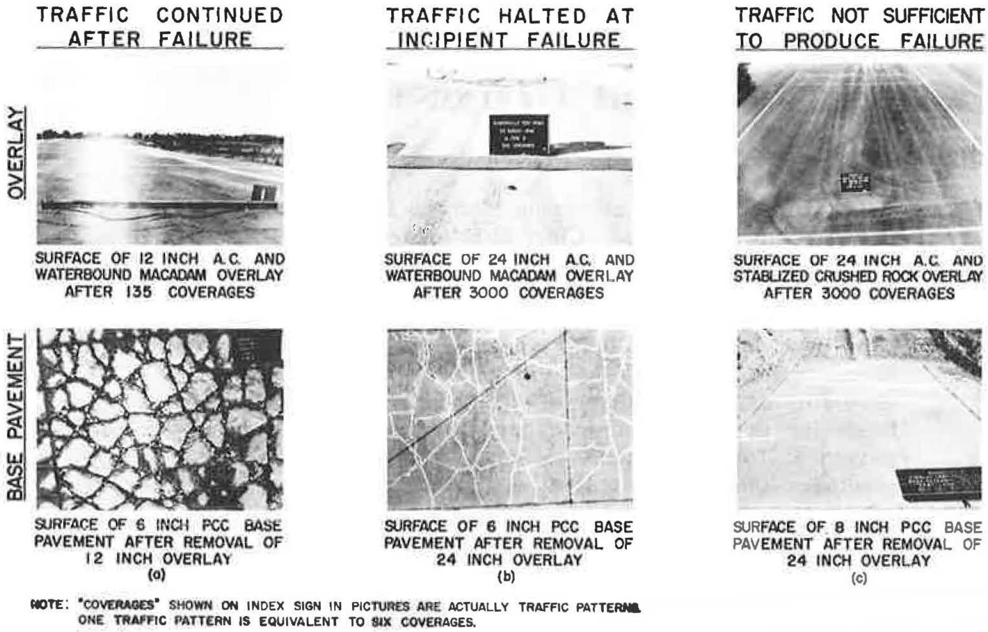


Figure 1. Typical after traffic overlay and base pavement conditions, Sharonville overlay test tracks.

duce failure. The base pavement was broken into pieces having an area of from 5 to 7 sq ft for the case where traffic was stopped at incipient failure, which was typical of the behavior of all 51 full-scale overlay test items which were subjected to traffic test loading. For these items, the thickness of the base pavements ranged from 6 to 12 in. and the thickness of the non-rigid overlays varied from 3 to 42 in. The thickness of the overlays composed completely of bituminous concrete varied from 3 to 20 in. Where base courses were used in the overlays, the base thickness varied from 5 to 38 in. with the thickness of the bituminous concrete surfacing being 4 in. in all cases.

Figure 2 shows a set of curves developed from full-scale traffic tests of plain concrete pavements. The percent of design thickness is plotted on semi-log paper against the number of coverages at which the pavement broke into pieces having an average area of roughly 7 sq ft. This is the condition of the base pavement at which a surface failure of the overlay is imminent. The following procedure was used to correlate the results of the traffic tests on the 51 full-scale overlay test items.

Using the procedures for the design of plain concrete pavements (3), the thickness of concrete h_d necessary to carry the wheel loading of the traffic for 5,000 coverages on each item was computed. This thickness was reduced by a percentage (percent standard design thickness) F, from Figure 2, depending on the number of coverages at which the test item failed. The design thickness (Fig. 2) is for the 5,000-coverage level. For example, Item No. 11 of the Sharonville No. 1 test track had a plain concrete base pavement with a thickness h of 6 in. and an all-bituminous-concrete overlay with a thickness t of 5.6 in. This item failed after 70 coverages of a 100,000-lb twin-wheel loading. The full concrete design thickness h_d required to carry this wheel load for 5,000 coverages is 16 in. The thickness of concrete required to carry this loading for 70 coverages and result in the condition of failure defined for the base pavement would be F h_d. Figure 2 establishes F as 51 percent in this case, the subgrade modulus k for this item being 100 pci. The dimension of concrete deficiency for the base pavement is given by $Fh_d - h$. In this case $Fh_d - h = 0.51 \times 16 - 6 = 2.2$ in.; that is, if

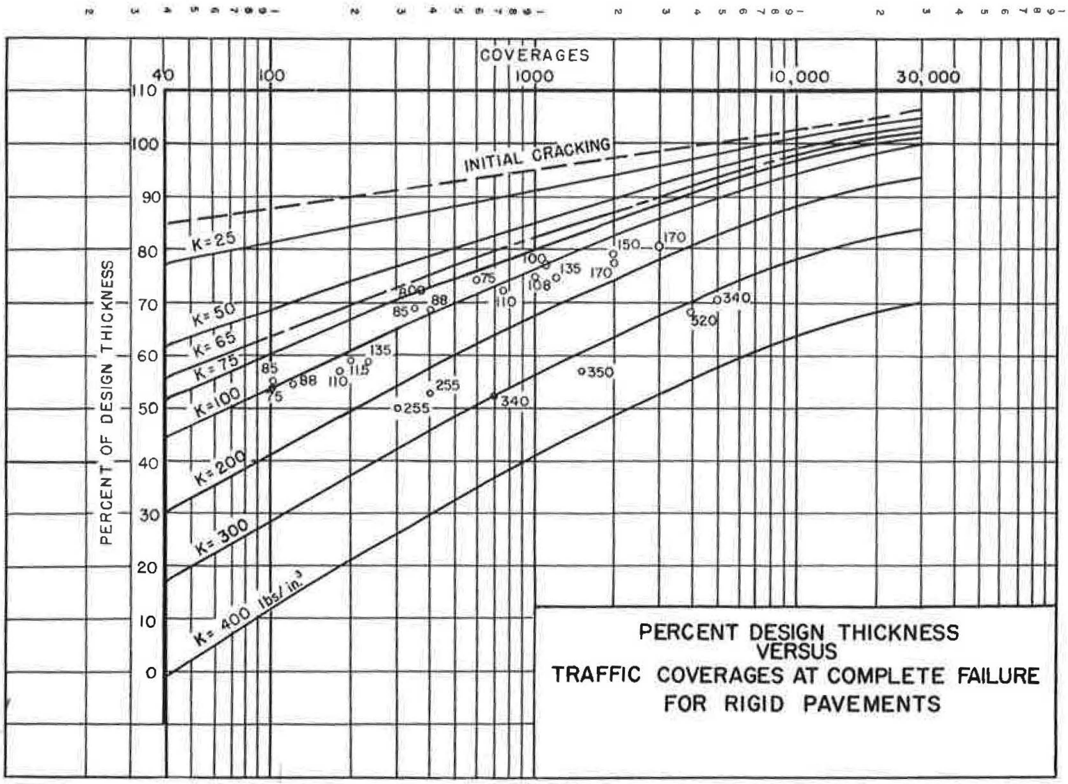


Figure 2.

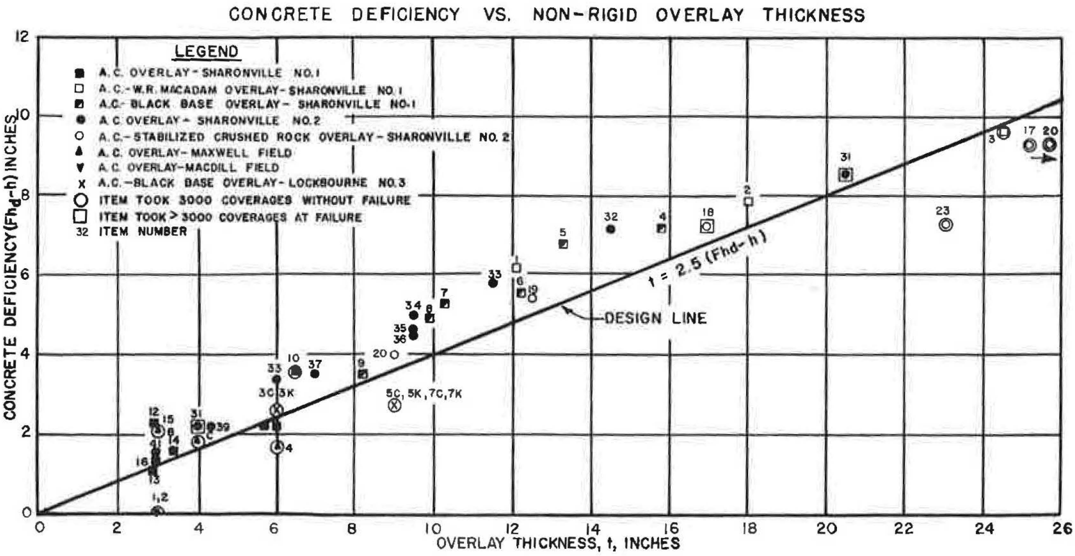


Figure 3. Concrete deficiency vs non-rigid overlay thickness.

the concrete base pavement had been 2.2 in. thicker, or 8.2 in. thick, it would have been reduced to the same condition after 70 coverages of the 100,000-lb twin-wheel traffic loading as it was after this same loading but having a 5.6-in. all-bituminous overlay. The dimensions, number of coverages at failure, subgrade modulus, and flexural strength of the concrete of the base pavement were known for each of the 51 overlay test items. Using this information, the concrete deficiency $Fh_d - h$ was computed for each item and plotted against the overlay thickness (Fig. 3). A straight line drawn through the origin and on the conservative side of the plotted values is also shown in Figure 3. This line is called the design line and is expressed by

$$t = 2.5\ (Fh_d - h) \qquad (1)$$

in which t is the thickness of the non-rigid overlay; F is a modification factor which varies with the subgrade modulus k and coverages. Values of F can be obtained directly from Figure 2, or curves of different coverage levels for k vs F can be prepared (1, Fig. 6) from the information in Figure 2. Eq. 1 is used to design non-rigid overlays for airfield pavements. To design such overlays, the flexural strength and the thickness h of the existing concrete pavement and the subgrade modulus are used. With these values, the design thickness, h_d of a plain concrete pavement is determined for the new wheel loading at the desired coverage level. The design life of a plain concrete pavement is determined by the design coverage level. The number of aircraft operations per coverage will depend on the gear configuration of the aircraft and on whether the pavement is to function as a runway, taxiway or apron. The airfield pavement is designed to fail at the end of its design life. Failure is considered to have occurred in a plain concrete pavement when 20 to 30 percent of the slabs have 1 or 2 cracks in them and spalling starting at the joints and cracks. When h_d is modified by the factor F, a pavement thickness is obtained such that the pavement slabs would be broken up into pieces 5 to 7 sq ft in area at the end of the design life. This design life or coverage level is the same as that for the full thickness h_d. The modification factor, F, is always less than one.

RIGID PAVEMENTS FOR ROADS AND STREETS

The adaption of the foregoing procedures to the design of composite pavements for roads and streets requires a plain concrete pavement design procedure that provides for a pavement life definable in terms of load repetition or coverages. Such a procedure for designing concrete pavements has been given (2) and details of its development covered (4). Pavements for roads and streets are subjected to a much greater number of repetitions of traffic loading than are airfield pavements. Also, road and street pavements are subjected to a greater variety of mixed traffic than airfield pavements. Therefore, two modifications of the airfield pavement design procedure were required.

1. The curve for load repetition or coverage vs percent design thickness had to be extended to include the greater frequency of traffic on road pavements that would be encountered in a 25-yr life. This curve (Fig. 4) was developed from full-scale accelerated traffic tests of concrete airfield test pavements for a range of 40 to 30,000 coverages. The dotted portion of the curve indicates its extension beyond this range. This extrapolation is based on judgment as well as on a limited amount of data from laboratory research studies of the fatigue characteristics of concrete in flexure.
2. The second modification was necessary to provide a means of taking into account the effect of mixed traffic on coverage values. This modification was made by selecting a basic 18,000-lb single-axle loading with dual wheels and deriving an equivalent-coverage factor for selected vehicles using standard wheel configurations (4). Table 1 gives this conversion with respect to vehicle type, loading, and vehicle-operations per coverage, with the equivalent-coverage factor being shown in the last column.

A coverage is defined as a sufficient number of vehicle-operations to produce one application of the design stress over the entire width of the traffic area. The relation-

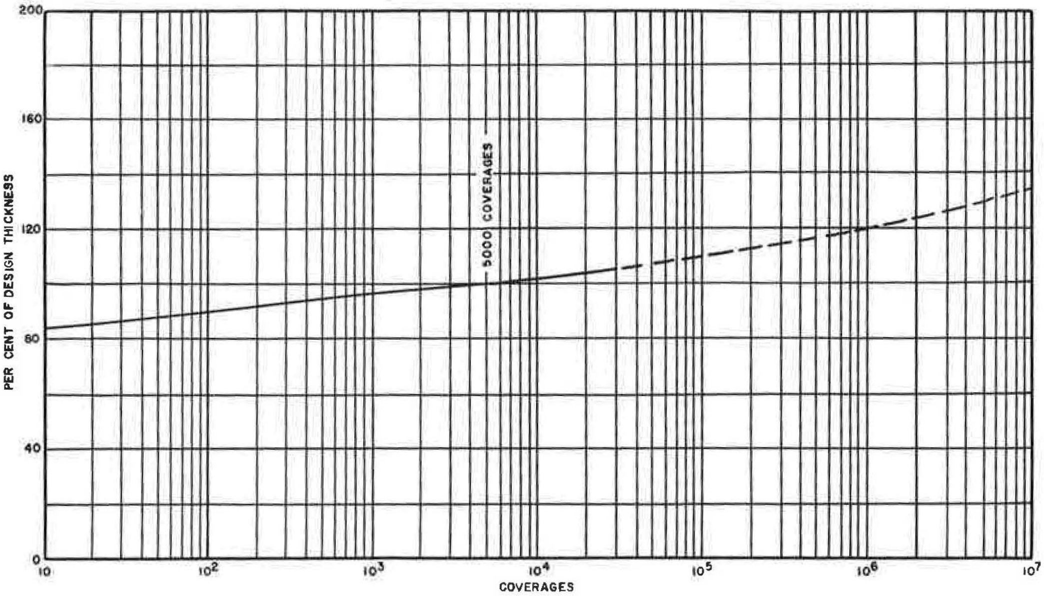


Figure 4. Rigid pavements coverages vs percent of design pavement thickness.

ship between operations (or applications) and coverages for each of the axle loadings (Table 1) is a function of the pavement lane width (11 ft), the width of the tire contact area, the number of wheels on the axle, the spacing of the wheels, and the degree of wander (or lateral distribution) of the traffic. The development of the values relating axle operations to coverages for each axle of the configurations of Table 1 has been described in detail (5).

To illustrate the use of Table 1, the case of the 3-axle truck is taken where 1.13 operations of this vehicle would be equivalent to 1 coverage of the standard 18,000-lb single-axle loading. The equivalent-coverage factor for this loading is 0.0288. On this basis, 1 coverage of the 18,000-lb single-axle basic loading would have the same effect stress-wise as approximately 1/0.0288 or 35 operations of the 35,500-lb 3-axle truck (4).

As an example of the further application, it is assumed that a pavement is being designed for the type of traffic distribution shown in the first three columns of Table 2. The number of operations for each vehicle type for a 25-yr (column 3) were converted to the number of equivalent coverages of an 18,000-lb single-axle loading by the equivalent-coverage factors (Table 1). For example, in the case of the 3-axle truck, the number of operations for 25 years is 9,125,000. The equivalent-coverage factors are for 1 operation of the various vehicles and not for 1 coverage. The equivalent number of coverages of the 18,000-lb single-axle load is obtained by multiplying the total number of operations of the vehicle by the appropriate coverage factor. The equivalent number of coverages, that is the number of coverages of the 3-axle loading that will have the same effect stress-wise as the standard 18,000-lb single-axle loading is obtained by multiplying 9,125,000 by 0.0288, the equivalent-coverage factor, which gives 263,000 equivalent coverages. The number of equivalent coverages computed for each vehicle type and total of the results are

TABLE 1

EQUIVALENT-COVERAGE FACTORS

Vehicle Type	Design Loading (lb)	Maximum Loading (lb)	Vehicle-Operations per Coverage	Equivalent-Coverage Factor
Passenger cars	3,900	4,500	4.79	1.4×10^{-10}
Panel and pick-up trucks	5,500	6,000	4.63	1.6×10^{-9}
2-axle trucks and buses	15,000	26,000	2.10	1.45×10^{-4}
3-axle trucks	35,500	44,000	1.13	0.0288
4-axle trucks	50,200	58,000	0.841	0.0444
5-axle trucks	62,400	68,000	0.677	0.0290

TABLE 2

EXAMPLE OF TRAFFIC SUMMATION FOR OBTAINING NUMBER OF EQUIVALENT COVERAGES FOR MIXED TRAFFIC

Vehicle Type	Avg. Operations per Day per Lane	25-Yr Operation per Lane	No. of Equivalent Coverages
Passenger cars	4,000	36,500,000	51.20×10^{-4}
Panel and pickup trucks	500	4,560,000	73.00×10^{-4}
2-axle trucks and buses	1,000	9,125,000	13.20×10^{2}
3-axle trucks	1,000	9,125,000	26.30×10^{4}
4-axle trucks	200	1,820,000	7.90×10^{4}
5-axle trucks	50	456,250	1.32×10^{4}
Total			356,520

TABLE 3

RELATIONSHIP BETWEEN RIGID PAVEMENT DESIGN INDEX AND EQUIVALENT COVERAGES OF THE BASIC LOADING

Rigid Pavement Design Index	Percent Thickness for 5,000 Coverages	Range of Equivalent Coverages	
		Minimum	Maximum
1	82.0	1	45
2	90.5	45	600
3	99.0	600	13,000
4	107.5	13,000	130,000
5	116.0	130,000	800,000
6	124.5	800,000	3,500,000
7	133.0	3,500,000	14,000,000
8	141.5	14,000,000	40,000,000
9	150.0	40,000,000	110,000,000
10	158.5	110,000,000	300,000,000

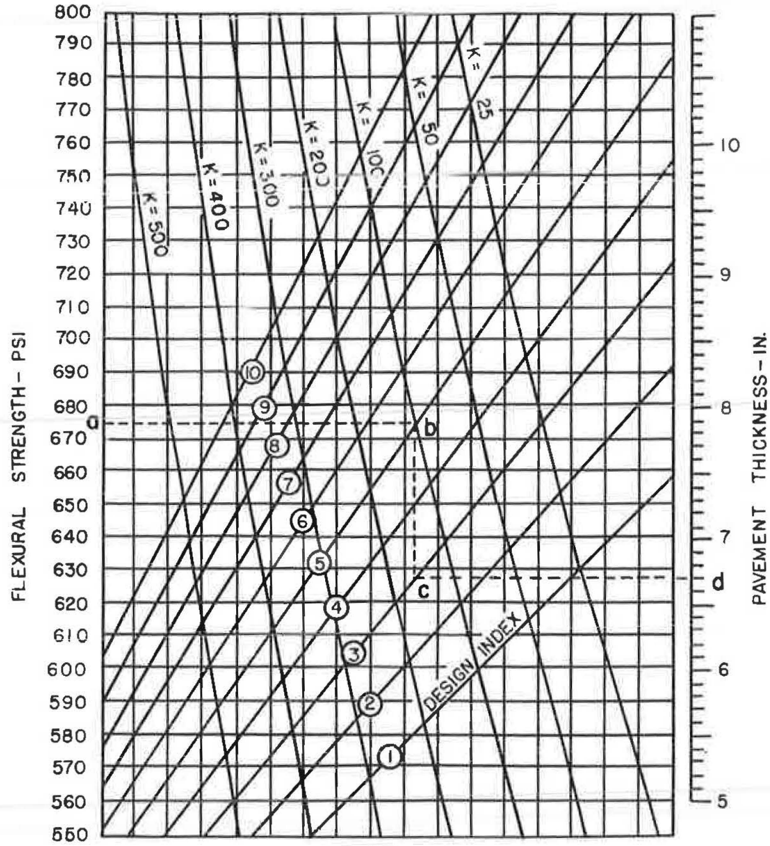


Figure 5. Design curves for concrete pavements roads, streets, and open storage areas.

given in the last column of Table 2. This gives about 356, 520 equivalent coverages for the total volume of mixed traffic over the 25-yr period for the number of operations per day per lane selected. A pavement for the vehicle types and traffic volumes in Table 2 can now be designed on the basis of 356, 520 coverages of the standard 18, 000-lb single-axle loading. The design method consists of computing the critical stress at a free edge for the 18, 000-lb single-axle loading by means of the Westergaard analysis (6). A uniform impact factor of 0. 25 and the appropriate coverage design factor from Figure 4 is included in the stress computation. To simplify the procedure, Table 3 defines pavement design indices for coverage levels of the basic loading.

For example, the design index for the equivalent-coverage level computed in Table 2 is 5 since the range of equivalent coverages which this factor represents is between 130, 000 and 800, 000 (Table 3). The average coverage design factor for this range from Figure 4 is 1. 16. With these factors established, the design chart (Fig. 5) can be prepared. The pavement thickness is determined by entering the chart with the flexural strength, as a, proceeding to the appropriate subgrade modulus line, as point b, then to the appropriate design index line, as c, and finally to the indicated pavement thickness, as point d. For example, to design a pavement when the 90-day flexural strength of the concrete will be 650 psi, the subgrade modulus 100 pci, and the traffic volume has a design index of 5, the thickness from Figure 5 is 8 in. If it were 8. 25 in. or greater, a 9-in. thickness would be used.

COMPOSITE PAVEMENTS FOR ROADS AND STREETS

The foregoing method of plain concrete pavement design provides the basis for composite pavement design by making available a means of determining h_d for roads and streets for use in Eq. 1. However, one further modification of the information developed for airfield pavement design is necessary. The curves of Figure 2 were extended from the 30, 000-coverage level to a 300, 000, 000-coverage level and curves for determining F, as given in Figure 6, for the design indices were obtained. Each curve represents a different equivalent-coverage level or design index, as defined in Table 3, and is numbered accordingly.

The formula for determining the non-rigid overlay thickness t of a composite pavement as developed from Eq. 1 for airfield pavements is

$$t = 2.5\ (Fh_d - Ch) \tag{2}$$

in which h_d is the exact design thickness (to the nearest 0. 1 in.) determined from Figure 5. Using the flexural strength of the existing rigid base pavement, the measured subgrade modulus k and the appropriate rigid pavement design index, the factor F is obtained from Figure 6. F is determined from the curve labeled with the same number as the rigid pavement design index. The C-factor is a coefficient depending on the structural condition of the rigid base pavement. Numerical values for C are determined as follows:

C = 1. 00 when the rigid base pavement is in good condition or contains only nominal initial cracking,
C = 0. 75 when the rigid base pavement slabs contain multiple cracks and numerous corner breaks.

The non-rigid overlay thickness t used in design should be determined to the nearest 0. 5 in.

For example, there is an existing concrete highway pavement 6 in. thick that is to be strengthened by means of a non-rigid overlay to carry the volume of traffic defined in Table 2 for a 25-yr period. The existing pavement is in good condition so that a C-factor of 1. 00 is used. The concrete has a flexural strength of 650 psi and the subgrade modulus is 100 pci. The rigid pavement design index for the increased traffic is 5. The full thickness of concrete h_d required to carry this increased traffic volume is 8 in. The factor F in Eq. 2 for a rigid pavement design index of 5 and a subgrade

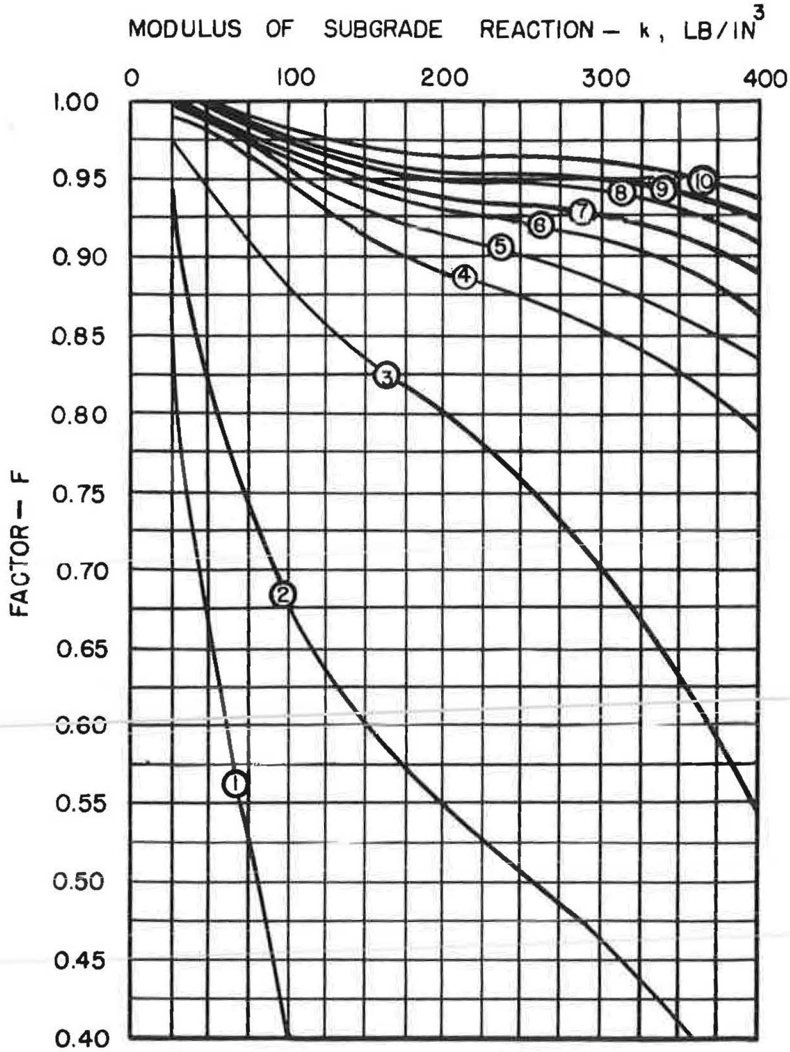


Figure 6. Composite pavement design factors.

modulus of 100 pci, as obtained from Figure 6, is 0.96. The non-rigid overlay thickness obtained using Eq. 2 is t = 2.5 (0.96 × 8 - 1 × 6) = 4.2 in. If the existing 6-in. concrete pavement has slabs containing multiple cracks and numerous corner breaks, a C-factor of 0.75 is used and the thickness t of the non-rigid overlay would be 8.0 in.

The two types of non-rigid overlays used in the actual construction of composite pavement are the overlay consisting entirely of bituminous concrete and the overlay consisting of a bituminous-concrete surface course over a granular base course (flexible overlay). Regardless of the type of overlay used, the thickness will be the same. The full-scale testing (1) indicated no difference in the performance of equal thicknesses of the two overlay types.

The all-bituminous-concrete overlay is used only when the combined thickness of a minimum 4-in. compacted base course and the required thickness of bituminous concrete surface course exceeds the design thickness t. There is no limitation, other than the economics of construction, on the maximum thickness of all-bituminous-concrete overlay that can be used. The bituminous concrete of the overlay is designed and constructed in accordance with previously reported requirements (7). A tack coat is used between the rigid base and the all-bituminous-concrete overlay.

A minimum thickness of 4 in. is required for an all-bituminous-concrete overlay where it is used to increase the structural capacity of an existing concrete base pavement. The purpose of this limitation is to reduce to a minimum the reflection cracking resulting from movements occurring in the rigid base pavement.

When the overlay design thickness t is large enough to permit a 4-in. or more compacted base course plus the required thickness of bituminous concrete surface course, a flexible overlay may be used. The bituminous surface course will vary from 1.5 to 5.0 in., depending on the type of traffic. The base course should be a crushed aggregate material with a CBR of 100 for the full depth. Gradation and compaction requirements of the base course material are given elsewhere (7).

The flexible pavement design method may indicate a lesser thickness of overlay required than that given by Eq. 2, and this possibility must be considered when designing non-rigid overlays for rigid base pavements. This condition may occur when the existing rigid base pavement has a flexural strength of 400 psi or less, or if the modulus of subgrade reaction k exceeds 200 pci. Where such conditions prevail, the existing rigid base pavement is assumed to have a CBR of 100, and the required total thickness of pavement above the subgrade CBR is determined using the flexible pavement design procedure (7). The overlay design is then based on the method which requires the lesser thickness over the existing rigid base pavement.

SUMMARY AND CONCLUSION

The method for designing composite pavements is dependent on a suitable design method for plain concrete pavements. To present this design method for composite pavement for roads and streets wherein the effects of load repetition and the physical properties of the various components of the pavement are considered, it was necessary to devote a considerable portion of the paper to outlining the Corps of Engineers' design method for rigid pavement for roads and streets. This method of designing rigid pavement for roads and streets is important to the composite pavement design because it sets up the basis for evaluating the effects of mixed traffic by means of a standard axle loading.

The formula for determining the thickness of the non-rigid overlay is empirical; however, it is based on the definite trend indicated by the results of carefully conducted full-scale traffic tests (Fig. 3). The full-scale tests included a variety of overlay and base pavement thicknesses and a range of subgrade modulus from 50 to 370 pci, the predominate range being 50 to 100 pci. It is in the range of weak subgrade support that composite pavements have their greatest application.

In addition to taking into account the physical properties of the various pavement components and the effect of load repetition that can be translated into terms of pavement life, the design method is based on a limiting failure concept that is defined with respect to the interaction of the base pavement and overlay.

ACKNOWLEDGMENT

The material presented in this paper was developed in connection with the U. S. Army Corps of Engineers pavement investigation program. The program is conducted by the staffs of the Rigid Pavement Laboratory, Ohio River Division Laboratories and the Flexible Pavement Laboratory, Waterways Experiment Station, Vicksburg, Mississippi. The program is sponsored by the Directorate of Civil Engineering, HQ, U. S. Air Force and is coordinated and directed by the Civil Engineering Branch, Engineering Division, Military Construction Directorate, Office of the Chief of Engineers, U. S. Army.

REFERENCES

1. Mellinger, F. M., and Sale, J. P., "The Design of Non-Rigid Overlays for Concrete Airfield Pavements." ASCE Proc., No. AT2 (May 1956).

2. U. S. Army, Corps of Engineers, "Rigid Pavements for Roads, Streets, Walks and Open Storage Areas." EM 1110-345-292 (June 30, 1961).
3. Mellinger, F. M., "Evaluation of Rigid Pavement Performance." HRB Bull. 187, 58-66 (1958).
4. U. S. Army, Corps of Engineers, Ohio River Division Laboratories, "Development of Rigid Pavement Thickness Requirements for Military Roads and Streets." Tech. Report No. 4-18 (July 1961).
5. U. S. Army, Corps of Engineers, Waterways Experiment Station, "Revised Method of Thickness Design of Flexible Highway Pavements at Military Installations." Tech. Report No. 4-582 (Aug. 1961).
6. Westergaard, H. M., "New Formulas for Stresses in Concrete Pavements of Airfields." ASCE Trans., Vol. 113 (1947).
7. U. S. Army, Corps of Engineers, "Flexible Pavements for Roads, Streets, Walks, and Open Storage Areas." EM 1110-345-291.

Proposed Research Relating to Composite Pavements

J. H. HAVENS, Assistant Director of Research, Kentucky Department of Highways

•THE intent here is to enumerate specific areas in which knowledge is deficient and in which "scientific break-through" would be most welcome and fruitful. There are implications from the use of laminated industrial products that laminated structures would be more capable of preventing stress concentrations or stress-risers, of redistributing stresses, of mobilizing more uniform fiber-stresses and of relieving thermal stresses (1) than a solid mono-layer would be. The more practical aspects of these ideas have been summarized in HRB Circular 473, "Suggestions for the Study of Composite Pavements," July 1962. The suggestions offered here are intended to supplement Circular 473 and the preceding proposals regarding experimental pavements.

SOME IMPLICATIONS IN BASIC THEORY

Recourse is made to Baker's treatment of elastic theory (2).

1. "...the influence of changing a perfectly rigid system to a flexible one is that of varying the vertical stress distribution beneath the pavement."
2. "...the importance of stiffness ratio is in the control which...relative rigidity exercises over the vertical stress distribution."
3. "Vertical deflections are...essentially defined by the load, the stiffness ratio, and the subgrade modulus...."
4. "...changing the subgrade modulus necessarily changes the stiffness ratio and thus the entire deflection curve. If the subgrade modulus k is altered, but simultaneous changes in pavement modulus or pavement thickness are also made in order to keep the stiffness ratio constant, the deflection curve is again changed.... However, if together with a change in k the stiffness ratio is also altered so as to recreate the original deflection curve, these different stresses will be produced for the same deflection curve."
5. "...Increasing or decreasing the thickness, keeping all other conditions constant, produces a change in stiffness ratio and...in the maximum movement due to the change in stiffness ratio and...in flexural stress due to changes in moment and in the thickness. However, if a change in thickness is balanced by a corresponding change in Young's modulus for the pavement so as to keep the stiffness ratio and... the maximum moment constant, then the variations in stress will be due to thickness changes only. Thus, for constant subgrade conditions, pavement flexural stresses are not a direct function of thickness unless a change in pavement modulus is also involved."

Translating the foregoing discussion into its significance or application to composite pavements, for a given value of k, the three dominating parameters are thickness, pavement modulus, and allowable flexural stresses. The concept of composite pavements inherently involves the possibility of being able to control the modulus of the pavement as well as the allowable flexural stresses; that is, through reinforcement of the extreme fibers and perhaps reducing the pavement modulus. For instance, a relatively thin but heavily reinforced-concrete mat overlaid by a relatively thicker course of bituminous concrete might greatly extend the range of allowable flexural stress than might otherwise be achievable with much greater thicknesses of bituminous concrete alone and remain so throughout all seasonal temperatures. Although this might well enhance the maximum allowable stresses at the bottom of the pavement, it would not greatly enhance the tensile strength at the top fibers to resist reversals of stresses or otherwise enhance the corner condition. Thus, the corner or edge condition might necessitate a similar type of reinforcement at the top of the pavement. At least, it is apparent that the modulus of the pavement as well as its flexural strength might be altered in this or some other way.

The elastic theory thus evokes some challenging opportunities for conjecture, insofar as concepts of composite pavements are concerned. The theory offers no clue as to how the pavement itself might best be built up, except that it should have high-tensile-strength layers at both the top and bottom, i.e., envisioned as a filled-in truss or sandwich. Of course, composite pavements, as presently defined by the Committee, are restricted to bituminous overlays or "open-sandwich" construction. However, the "full-sandwich" concept should not be ignored or rejected by the Committee without due investigative processes.

FATIGUE

As in the design of conventional types of pavements, the elements of fatigue are of concern. The EWL-concept of mixed traffic offers several interesting possibilities for analysis.

$$EWL = n_1 f_1 + n_2 f_2 + n_3 f_3 + \ldots = N f = N (r)^{P-b} \tag{1}$$

in which

n = number of applications;
f = severity factor for respective loads;
N = equivalent number of applications of load P;
r = ratio of successive factors;
P = axle load in tons or wheel load, in kips;
b = basic axle load in tons or wheel load, in kips; and
$f = (r)^{P-b}$.

The original California factors f were based on Bradbury's earlier work. Table 1 gives these factors in comparison to AASHO Test Road factors.

The smoothed ratio between successive AASHO factors is approximately 1.5; whereas, the ratio between the original California factor is 2.0. However, the EWL's are easily converted from one system to the other.

TABLE 1

P	California Factors		AASHO Factors (9-ton basis), (P_t = 2.0)	
	Original (5-ton axle)	Converted (9-ton axle)	Rigid D_2 = 9	Flexible SN = 5
5	1	0.0625	0.08	0.08
6	2	0.125	0.18	0.17
7	4	0.250	0.34	0.34
8	8	0.500	0.60	0.60
9	16	1.000	1.00	1.00
10	32	2.000	1.58	1.57
11	64	4.00	2.38	2.35
12	128	8.000	3.47	3.40

The EWL-concept of mixed traffic seems to be wholly in accord with Miner's law of fatigue in metals (3). This is so, provided that load-equivalency factors are reliably determined and provided that the loads imposed are within the fatigue range.

There is a need for research and experimentation in the region of loading which will produce failure near the upper limit of the fatigue range. This is as true perhaps in regard to rigid and flexible pavements as it is to so-called composite pavements. For instance, assume that:

$$\text{EWL (9-ton axle)} = n_1 f_1 + n_2 f_2 + \ldots = 1,100 \text{ per day}$$
$$1,100 \times 7,300 \text{ (days in 20 yr)} = 8.03 \times 10^6$$
$$8.03 \times 10^6 = N (1.5)^{P-9}$$
$$P = \text{axle load in tons}$$
$$\text{Let } N = 1$$
$$P = 48.4 \text{ tons}$$

This implies that a 48.4-ton axle would cause failure, in one application, of a pavement designed to carry 8.03×10^6 equivalent, 18-kip axles. In the same way, a load P of 40.6 tons would produce failure in 2 applications, etc.

This task group suggests that research and experimentation in this region of loading would be worthwhile in substantiating fatigue concepts of pavement behavior.

DYNAMICS OF CARGO HAULERS

The principal mass, i.e., cargo and body of a vehicle, supported on springs and tires and undergoing translational motion will undulate and thus transmit variable forces to the pavement. Dynamic forces may then be alternately greater and less than the weighed or static force. Measurements on pavements have indicated that for a given load, deflections and strains decrease as the translational velocity of the vehicle increases. To infer from this that the dynamic forces exerted on the pavement are continuously less than the static force would be wholly irrational. The true inference here is that the response of the pavement varies as the speed of the vehicle increases. The nature of this response remains obscure and proper explanation for this phenomenon should be sought in viscoelastic, time-dependent deformation studies on pavements and in the effect of forward thrust and traction upon elastic stress distribution with the pavement.

Extraneous disturbances and pavement excitations tend to force the principal mass into sinusoided oscillation having one or more modes—depending on the number of spring elements and their stiffnesses. The ratio of the peak dynamic force of the principal mass to the static force or weight of the mass is customarily termed the impact factor. Values ranging between 1.2 and 1.3 are in common usage. It is suggested that reliable impact factors are essential to the stress-analysis problem as well as to the fatigue problem. However, the effects of dynamic forces remain hidden in the EWL-parameter when the relationship between pavement performance and traffic is established empirically.

REFERENCES

1. Wiseman, J. F., Harr, M. E., and Leonards, G. A., "Warping Stresses and Deflections in Concrete Pavements: Part II." HRB Proc., 39:157-172 (1960).
2. Baker, R. F., and Papazion, H. S., "The Effect of Stiffness Ratio on Pavement Stress Analysis." HRB Proc., 39:61-85 (1960).
3. Monismith, C., Secor, K., and Blackmer, E., "Asphalt Mixture Behavior in Repeated Flexure." AAPT Proc. (1961).