EVALUATION OF ASPHALT PAVEMENT PERFORMANCE

submitted to

Idaho Department of Highways
Project No. 42

IDH - RP042

December, 1966

by

MATERIALS RESEARCH & DEVELOPMENT, INC.
2811 Adeline Street
Oakland, California

2811 ADELINE STREET, OAKLAND, CALIFORNIA • 94608 • (415) 444-1256

MATERIALS
RESEARCH &
DEVELOPMENT, INC.

RD 1061

December 16, 1966

Mr. R. B. Christensen, P.E.
Assistant State Highway Engineer
(Engineering)
Idaho Department of Highways
P. O. Box 7129
Boise, Idaho 83707

Dear Sir:

Project No. 42 -
Evaluation of Asphalt
Pavement Performance

In accordance with the terms of our consulting agreement dated June 24, 1966, we are submitting herewith the final report for Phases I and II of the subject project.

The engineers participating on this project for Materials Research & Development, Inc., in addition to the undersigned, were Messrs. B. A. Vallerga, F. N. Hveem, and R. G. Hicks. The laboratory testing for Phase II was under the supervision of Mr. R. M. White, Chief Chemist.

The excellent cooperation of Department and Bureau of Public Roads personnel who participated in this project is gratefully acknowledged.

Very truly yours,

Fred N. Finn

Fred N. Finn, P.E. 1800
Vice President, Engineering

REGISTERED PROFESSIONAL ENGINEER 1800 STATE OF IDAHO FRED N. FINN

FNF/ljb

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I. SUMMARY

The basic approach in the execution of this project for the Idaho Department of Highways has involved seven major steps, as follows: (1) discussions with Department personnel, (2) formulation of a suitable condition survey procedure, (3) field condition survey of representative pavements, (4) development of hypotheses as to mechanism for the types of distress observed, (5) evaluation of hypotheses using Department data obtained from constructions included in the field survey, (6) suggestions for modifications in materials and construction requirements, and (7) recommendations for future research.

The conclusions and recommendations relate to considerations dealing with: (a) materials requirements and control, (b) structural design, (c) construction procedures, (d) maintenance procedures, and (e) future research and special studies.

Each step in the project will be discussed in the subsequent sections of this report. The essence of the findings and recommendations are as follows:

A. Findings

1. Distress of various types was exhibited on a portion of those pavements examined. The most prevalent type of distress is the load associated longitudinal cracking pattern occurring generally in the wheel tracks of the traveled lanes. Major emphasis has been given

to this item in terms of the analyses and recommendations. The other forms of distress encountered were variously described as: rutting, ravelling, spalling, transverse cracking due to shrinkage presumably caused by thermal stresses or absorptive aggregate, longitudinal construction cracks, alligator cracks and shrinkage cracks. The extent of these types of distress is indicated to be minor and not of sufficient seriousness to warrant special corrective programs beyond the normal actions currently underway in the Department to improve all phases of materials testing, pavement design, and construction.

2. Examination of construction and materials records has not provided any clear indication as to the causes of the longitudinal cracking. Examination of these cracks, which were present on some 65 percent of the projects surveyed, indicates that they are load associated. This conclusion leads to three possibilities of causative factors: (1) inadequate ballast (structural section), (2) materials deficient in tensile or fatigue resistance properties, or (3) construction procedures which do not produce necessary in-place strength characteristics.

3. Examination of the current pavement design procedure used by the Department indicates it to be in substantial agreement with modern practice in this area

of technology. In several respects, particularly as regards classification of loadometer studies, the application of the Department's design method represents excellent planning and interpretation.

4. Because of a lack of strong evidence, the major share of recommendations contained in this report are based on the subjective field observations, recent research developments in pavement technology and the engineering experience and judgment of the various individuals working on this project, including engineers of the Bureau of Public Roads and the Idaho Department of Highways. The one single exception to this type of deductive reasoning has been the disclosure that the asphalt is being stripped from the aggregate in the lower courses of the plant mix, particularly in the coarser graded mixes. This evidence was noted on projects in Districts 1, 2 and 3 in the field condition survey and subsequently in District 5 by a Department study. The net effect of this finding is that the ballast section is being substantially reduced by the loss of tensile strength in a portion of the plant mix. It is also believed that the stripping will reduce the fatigue life of the surfacing.

5. There is no conclusive evidence that construction is significantly influencing the performance of the pavements. Longitudinal construction joints have, in a few instances, resulted in cracks and occasionally spalling adjacent to the cracks. The more recently constructed pavements appear to have good, monolithic longitudinal joints. The transverse construction joints are evident on most jobs and usually create a slight discomfort to the user; however, because of their infrequent occurrence, pose no serious problem. There was some indication that the longitudinal construction joint in the first layer (course) of asphalt concrete was associated with a longitudinal crack observed in the surface layer. This item is discussed further in Section IV of the report.

6. There was some visual evidence, with at least some verification in terms of laboratory tests, to indicate that absorptive aggregates used in the plant mix are resulting in a higher frequency of transverse cracking and possibly block cracking (random shrinkage cracking).

7. Variations in asphalt content appear to be rather high on a significant number of projects for which this type of information was available. In some cases, it was possible to detect significant changes in the asphalt content by visual observations.

8. Evaluation of the durability of ten asphalts by the Rostler parameter and pellet abrasion test indicates that three asphalts may be of marginal quality. However, the asphalts tested are fairly representative of asphalts produced in most areas of the United States. Continued observation of these asphalts is recommended.

B. Recommendations for Future Considerations in Design, Construction and Research

Recommendations are largely designed to minimize stripping in the asphalt concrete and to implement research findings in asphalt pavement technology which tend to indicate an improvement in the reliability of performance. The following items briefly list the recommendations discussed in the body of this report.

1. Coarse aggregate gradations should not be used on projects with traffic classifications of "Heavy" or "Average" according to the Department's Plans and Survey Manual.

2. Commercial hydrated lime should be used as an anti-strip additive on all plant mixes manufactured with aggregates which have demonstrated a susceptibility to stripping.

3. Stripping characteristics should be studied extensively both in the field and the laboratory.

4. Absorptive aggregates should be identified by means of the centrifuge kerosene equivalent (CKE) test with special mix design considerations in cases when K_f or K_c exceed 1.5.

5. For heavy-duty highways, some special provisions should be added to the standard specifications which will aid in optimizing the performance. Of major concern are those items dealing with the surfacing. A new designation of "Asphalt Concrete" could be used for such constructions. Specific recommendations for this material are included in the report.

6. Projects, built in stages, left open (without surfacing) or unfinished through the winter, should be thoroughly tested for density and material properties prior to initiation of work in the subsequent construction season or construction contract. Any deficiencies should be corrected in accordance with basic design and construction requirements.

7. Implement a series of research projects and special studies with the following objectives.

   (a) Develop some means of identifying asphalt-aggregate combinations which tend to strip

   (b) Continue to inventory in-service pavements on a regular schedule

(c) Continue to undertake case studies of <u>both</u> damaged (distressed) and undamaged pavements

(d) Develop nondestructive tests to evaluate the life characteristics of in-service pavements and to identify the in-service performance of various materials used in pavement constructions on a state-wide basis

(e) Re-examine thickness design requirements with particular emphasis given to materials with high strength properties

(f) Re-examine the regional factor to determine if additional requirements are necessary for the Idaho environment

(g) Establish better quality control relative to materials and construction associated with highway contracts, and

(h) Continue to study the interaction between asphalt properties and pavement performance particularly to study cold temperature properties.

8. A table of minimum pavement thickness requirements is provided in Section VII, Recommendations.

These and other recommendations are discussed in the subsequent sections of this report. The above items have been specifically sorted out as being of the type which would be most beneficial to the performance problems considered to be of major importance and for which a reasonable effort and cost could be expected to produce some useful results.

The items for future study are also limited to major problems. It is emphasized that the confident execution of special studies and research referred to above will require a substantial effort by Department personnel. It is believed that, in terms of meeting the long-term needs of the Department, such an effort will be justified. Research in the area of highway materials, pavement design, and pavement construction is proceeding in this country and abroad at a rapid rate. To take advantage of this substantial effort by implementing research findings to Idaho's needs should be a prime objective of the Department.

II. INTRODUCTION AND SCOPE

The purpose of this project for the Idaho Department of Highways, as described in the basic contract, is to conduct a "comprehensive study of the level of performance achieved by selected in-service asphalt pavements with due consideration being given to all pertinent variables, a thorough review of similar studies in or for other agencies, and of systems developed for measuring serviceability of pavements, a comprehensive review of findings, recommendations as to correction of deficiencies for future use, and participation in an asphalt rating panel."

The subsequent sections of this report are the result of an effort by Materials Research & Development, Inc., (MR&D) to accomplish the study outlined above in accordance with the specific work outline prepared jointly by the Department of Highways and MR&D.

The responsibility for preparing this report, together with accumulation of supporting data and information, was assigned to MR&D. However, it is noted here that some substantial contributions were made by the members of the Pavement Review Panel and by representatives of the Idaho Department of Highways.

The Pavement Review Panel participated actively in the field inspection of selected pavements and has continued to act in an advisory capacity throughout the execution of the project. The members of this panel are:

Idaho Department of Highways:

James Clayton
William Sylvies

Bureau of Public Roads:

Raymond Swegler

Materials Research & Development, Inc.:

F. N. Finn
F. N. Hveem
B. A. Vallerga

In addition, the assistance of Mr. L. F. Erickson, Materials and Research Engineer for the Idaho Department of Highways, is gratefully acknowledged. Mr. Erickson provided excellent coordination between the Department and MR&D which resulted in a maximum of efficiency in the execution of those phases of the contract involving participation by both agencies. He also summarized the Department's construction records to provide background information used to analyze various aspects of materials properties, design, and construction for the selected projects included in the field survey.

Finally, a sincere note of appreciation to Department personnel, too numerous to identify individually, who cooperated to make the project possible. It is pertinent to note and comment on the level of enthusiasm of Department personnel. These engineers demonstrated a real interest in their desire to provide, for the State, the best possible highway system at the least possible cost. Their desire and capability are indeed commendable.

From discussions with Department personnel, it is apparent that they are well informed on recent developments in highway technology. In several areas, the Department has been able to provide recognized leadership.

The technician training program is commendable and every effort should be made to continually appraise the program to assure a source of well informed, well trained technicians.

The Department's development of special laboratory testing equipment is well known. The latest developments are the sand equivalent shaker, which has been copied by states and counties throughout the West, and the basin beam deflection device being studied to replace the Benkelman beam.

Research Project No. 20 of the Department represents one of the most complete studies of axle weight distributions available in the literature. While this information is only applicable to Idaho, it recognizes the variability in loading patterns as a function of environment, industry, farm to market, etc., and thus provides a realistic way of assigning funds according to need.

The project covered by this report is an effort by the Department of Highways to determine possible ways of improving the reliability of performance of heavy-duty pavements on the State and Federal System. Regardless of the findings, studies of this type are a sign of the continuing interest of the Department to maintain an awareness of potential problems and corrective procedures.

III. REVIEW OF PAVEMENT PERFORMANCE STUDIES AND DEVELOPMENT OF FIELD SURVEY PROCEDURES

A. Review of Pavement Performance Studies

The literature contains a number of useful publications describing the conduct and results of pavement performance or condition surveys. There have also been some studies of performance which have not been published. Specifically, California and Washington have, or are now conducting, evaluations with broad coverage in order to learn more of pavement performance. Discussions with representatives of these organizations and reports made to the Triaxial Institute have been helpful in examining these unpublished procedures. It has been found that in the majority of the performance surveys, the objectives of the particular project have controlled the procedure for the survey. For example, some investigators were specifically interested in obtaining information relative to the performance of the asphalt while others were attempting to evaluate a regional factor or thickness association to the AASHO Road Test results or the AASHO Interim Guides.

Reference (1) outlines various types of condition surveys designed to be helpful in evaluating the influence of various factors such as material properties, structural design procedures and construction methods on pavement life. Specific types of condition surveys were described as follows:

1. Reconnaissance surveys which are simply cursory observations generally made while driving over pavements.

2. Statistical or tally surveys wherein various features of pavement conditions are merely noted and summarized.

3. Intermediate or semi-detailed surveys in which identifiable features are noted for subsequent surveys; no sketches are made during survey.

4. Detailed sketch or strip map surveys in which features or pavement condition are sketched to scale and in detail for further analytical study.

5. Photographic surveys which produce an exact reproduction or photographic film of the pavement surface.

In addition to this effort, the Highway Research Board (2) has published a specific procedure for conducting pavement condition surveys aimed primarily at establishing maintenance requirements and priorities. The Washington Department of Highways has used an adaptation of reference (2) in their current field survey effort.

References (3) and (4) describe methods for rating pavements based on riding quality. Initially, these methods involved subjective evaluations or ratings by a selected panel of raters generally knowledgeable in evaluating overall pavement performance, but not necessarily knowledgeable in causes of distress. The subjective ratings, identified as "Present Serviceability Rating," were eventually computed in terms of objective measurements; such as, (1) longitudinal profile, (2) rut depth, and (3) cracking plus patching, then summarized into a present serviceability index (PSI). The

PSI procedure provides a present estimate of riding quality. In order to obtain some indication of performance, it is necessary to obtain a series of ratings over an interval of several years. The requirement to measure three identifiable features of pavement condition provides a means for supplemental analysis of the dominant type of distress.

Reference (5) describes a study which combines a pavement condition survey, as the dependent variable, correlated with measurements of surface Benkelman beam deflections, moisture-density factors, and physical properties of the subgrade and base as independent variables. For the purposes of this study, pavement performance was divided into six categories using both descriptive, e.g., Excellent, Good, etc., and scaler values, e.g., 95-100, 90-95, etc. The report deals with evaluating performance of 115 miles of flexible pavements, three to eight years of age, ranging from plant mix surface types to surface treatments. The main conclusion from this effort was to indicate that there were some anomalies in relating pavement deflection to pavement performance. This report suggests that radius of curvature, as obtained from the Helmer type recorder, would be a better predictor of performance than deflection.

Reference (5) is only one of many, (3), (6), (7), (8), (9), (10), (11), and (14), just to list a few, which deals with deflection and curvature as a means to predict performance.

Some of these efforts have met with success while others have not. For the most part, deflection has a high probability of being associated with pavement performance in terms of both riding quality (3) or cracking (6). For this reason, deflection measurements have generally been accepted by highway engineers as a measure of life expectancy or performance potential. There are some exceptions to this rule, and as will be discussed in more detail later in the report, performance on most of the newer pavements on the Idaho system for which deflection measurements are available do not correlate well with deflection. The level of deflection on these Idaho pavements is very low, and yet cracks apparently associated with load are developing. Reference (7) describes a similar situation on an Indiana test road. This report concludes that deflections accumulated within the pavement structure are potentially more damaging than deflections accumulated from the subgrade (basement) materials.

The essence of this work is the inference that deflections may be a simple and direct method for evaluating pavement performance, but that under certain conditions, possibly related to climate or to materials, such measurements may require special evaluation. It is pertinent to point out that difficulties of interpretation are related to low deflections only. High deflections, greater than approximately 0.060 inch, will almost certainly lead to poor performance under even low traffic loadings.

References (<u>12</u>) and (<u>13</u>) describe pavement performance evaluation programs in the State of Michigan. The primary objective of this study was to provide an accurate way for evaluating the adequacy of pavement design and detecting weaknesses in performance. The main dependent variable was the change, with time, of the longitudinal profile. The conclusions expressed by these reports indicate that: (a) the structural designs, as developed by the Michigan Highway Department, were adequate for all-season service without load restriction, (b) both rigid and flexible pavements lose serviceability (riding quality) in terms of roughness at a uniform rate due primarily to climate and environment, and (c) pavement construction should attempt to achieve a high level of initial riding quality since initial "built-in" roughness may be reflected directly in a reduced useful life of a pavement. Housel, in reference (<u>12</u>), takes exception to the use of static deflection measurements to predict performance since these measurements do not reflect environment. As tentatively reported, temporary pavement displacements due to frost action were high among factors contributing to changes in the longitudinal profile. Also, it is concluded that "frost displacement appears to originate in the freezing of moisture which accumulates in the subgrade and granular bases and subbases immediately beneath the pavement surface." Thus, frost depth per se would not appear as important as previously thought and that low (freezing) temperatures are sufficient to cause pavement displacement and pavement damage.

Reference (14) describes extensive pavement evaluation work in Canada. This is one of the most complete studies of its type in the literature; and, because of similarity in climate, should be very useful to Idaho Department of Highways engineers. It is pertinent that one of the conclusions in this reference indicates that significant losses in riding quality of flexible pavements occurs each year regardless of pavement strength or traffic loading. This conclusion is in general agreement with Housel (12,13) and indicates a need to better understand climatic factors as they relate to pavement performance.

Reference (15) outlines an extensive program to extend the results of the AASHO Road Test to flexible pavements in Minnesota. This particular program is concentrating on physical measurements, e.g., plate bearing tests, Benkelman beam deflection, material properties, traffic, etc. This is a long-term program aimed at adjusting the various factors associated with pavement design in such a way as to be able to use AASHO Road Test type equations for pavement design in Minnesota. No specific conclusions were reached or expressed in the report. In a way, this project would also develop a Regional Factor for use with the AASHO Interim Design Guide For Flexible Pavements.

Reference (16) describes a study of cracking in asphalt surfacings attributable to thermal effects. This study was conducted in the province of Alberta, Canada. The field studies were conducted in the southeastern portion of the province (10 inches of precipitation with a freezing index of 1200-2000 degree-days) and the central portion of the province (10-15 degree-days). The investigators describe the suspected thermal cracks as occurring at regular intervals (5-15 feet) transversely along the length of certain projects. The field study included sections exhibiting both cracked and relatively uncracked pavements constructed contiguous to one another. Field evaluation consisted of mapping the cracking pattern and the subjective measure of riding qualities according to procedures developed by the Canadian Good Roads Association (14). Field cores were taken from the pavement for laboratory testing of the subgrade, base, and surfacing materials. Approximately 1500 miles were surveyed in the program. The results of this study were made inconclusive by the lack of strong correlations between the measured material properties and the presence or absence of transverse cracking. The authors appear to be suggesting that the source of the asphalt is related to cracking; however, this is confounded by the fact that changing asphalt also may be tied to a change of contractors. Asphalts were tested for penetration, ductility, and absolute viscosity with no conclusive relationship developing. Pavements which exhibited transverse cracking lost serviceability at an accelerated rate, according to the authors, largely due to swell in the underlying soil. This rather significant work points out the type of crack which can be associated with thermal stresses and volume change, but, as yet, is not definitive as to cause.

It is pertinent to discuss the report by O. A. White (17) dealing with a pavement survey conducted in the state of Oregon. The purpose of this study was to determine the effect of time and traffic on asphalt surfacing and to compare laboratory compacted specimens of asphalt concrete with specimens obtained from in-service pavements. The project was initiated in 1954 and included some 872 miles of asphalt surfacing. Measurements of in-service performance, including such factors as cracking, ravelling, shoving, flushing, and wheel track rutting, were used as performance variables. It is interesting to note that from the in-service evaluations, there was a measureable amount of cracking in over 40 percent of the sections evaluated after some six years of service. It was concluded from this project that an increased amount of asphalt cement would have reduced the amount of cracking. Only minor ravelling and flushing was reported. One of the most significant conclusions reached from this study was the need to increase density requirements in order to improve performance of the pavement sections.

Kenis (18) describes an experimental test road in Delaware. This project involved both asphalt and portland cement concrete types of construction. At the time the report was issued in 1962, no specific conclusions were possible due to the lack of pavement response to either time or loading. However, the author of this report does draw an interesting conclusion in the following statement:

> "At present no specific conclusions can be drawn, but the laboratory tests indicate that variances in behavior of the same asphalt at different locations in the road may be as great or greater than variances in the asphalts from the different crude sources used in the study."

This statement is significant in that it would indicate that methods of design and construction will need to be developed which can minimize the performance dependency on asphalt properties.

In summary, the pavement evaluation programs which have been conducted by various research agencies, as briefly reviewed herein, cover a broad spectrum of environments, materials, and construction methods. While these surveys are not word-for-word relatable to the Idaho project, they do provide background information for recommendations which will be made hereafter in the various sections of this report. Additional projects are described in references (19) through (24). It had been hoped that specific information relative to strength properties of materials, durability properties, and construction requirements could be forthcoming from such a survey. It would now appear that specific quantitative values cannot be reasonably extrapolated from one set of results to fit the specific needs of the Idaho Department of Highways. However, in a qualitative sense, these results are helpful.

B. Development of Pavement Condition Survey Procedure

1. Objective

The objective of the pavement condition survey is to evaluate the performance of a selected sampling of Idaho highways for the purpose of: (a) identifying the types and causes of distress currently being manifested by plant mix type pavements, (b) associating, on the basis of subjective evaluations made during the field survey, those factors which appear to be prime contributors to poor pavement performance, and (c) making recommendations for modified procedures related to materials testing, structural design, and construction which may be effective in reducing the incidence of distress.

2. Procedures for Conducting Condition Survey

In planning for the field condition survey, several different procedures of evaluation were available:

a. Reconnaissance survey - Cursory observation made by driving over a pavement section and noting the general performance and appearance. This procedure has the advantage of being able to cover a relatively large number of pavement sections per unit of time. The prime disadvantage in this method is that it does not provide the reviewer with the necessary time to examine the pavement in sufficient detail to identify the types and causes of failure. This method does not appear adequate to the objectives of this project.

b. Tally or statistical survey - This method requires that various types of distress be noted and eventually summarized. For example, in this type survey, a pre-selected listing of factors such as (1) rutting, (2) slippage, (3) ravelling, and (4) cracking would be considered as important performance parameters. The rater would simply note the presence or absence of these performance items on a particular project. This method will reduce the number of sections that can be rated per day as compared to the reconnaissance survey. It has the advantage of being able to identify pavement distress by types. If a sufficient number of pavement sections can be observed, a statistical evaluation should begin to identify the most prevalent types of distress.

c. Analytical-subjective method - In this method, some effort is made to assign numerical values to the present performance or condition of the pavement. The procedure involves subjective estimates of negative values to assign various forms of pavement distress. This appears to be relatively slow as compared with the reconnaissance or tally procedure; however, it provides considerably more information. This method would identify the type of distress and the weighted degree of severity or criticality. Per se, it does not attempt to identify the cause of distress. The summation of all negative values provides a numerical evaluation of performance. Washington has used the procedure primarily for the purpose of establishing maintenance priorities on the basis of need.

d. Objective evaluation - Several states (Texas, Minnesota, Arkansas, Missouri, and Michigan) have initiated extensive programs of evaluation involving measurements of riding quality (Present Serviceability Index) and strength (deflection, plate bearing tests, etc.) to evaluate material properties and strength potential. For the Idaho evaluation project, a measurements program of this type is considered outside the present scope of planned investigation. However, future studies may require this type effort.

e. Strip maps - This method requires a detailed measuring and locating procedure by which each type of distress is located and identified on a continuous strip map. This is a very detailed condition survey and applicable for special problems.

After examining the variety of survey procedures which have been used by other agencies, it was generally agreed that some combination of the so-called tally method and the analytical method would be best suited to the objectives of the Idaho project.

3. Developing Pavement Condition Survey Procedures for Idaho

The general procedure followed in developing the condition survey technique was as follows: First, two trial runs were made in cooperation with representatives of the Idaho Department of Highways. In these initial efforts, the primary objectives, as previously discussed, were to identify the types of distress and, wherever possible, to estimate, on the basis of engineering judgment, the causative factors related to the identifiable distress.

A number of forms were developed, and, in fact, during the process of conducting the field surveys, two forms were actually used. The final pavement condition survey form is attached as Appendix A. This form generally outlines the procedure followed in making the survey and provides a systematic method for data accumulation.

The procedure followed in the actual conduct of the pavement condition survey is divided into three steps:

a. The first step was the identification of the project together with examination and tabulation of information relative to traffic, construction, age, ballast design, basement soil type, asphalt cement grade, asphalt cement supplier, asphalt content, and such other miscellaneous information relative to materials, construction, and design as seemed appropriate to the condition survey. Typically, such information as the identification of a seal coat would be noted at this time.

b. The second step in the survey was to drive through the project at a speed of approximately 50 to 60 miles per hour. The purpose of this step was to obtain a present serviceability rating (PSR) for the project. In some cases, it was possible to obtain a serviceability rating in both directions or on both lanes in the case of a two-lane roadway; however, this was not possible in all instances. In those cases where riding quality was evaluated in both directions, it was generally found that no apparent difference could be identified in this particular characteristic of the pavement. Hence, tables summarizing the riding quality have been reduced to a single number which, for purposes of this study, is considered to be associated with the travel lane in each direction.

It is pertinent to point out that the present serviceability rating concept as used on this project. is identical to the riding quality concept used on the AASHO Road Test. In this way, it is possible to compare results of the Idaho survey to similar studies made by other highway departments. For example, reference (24) describes a national survey made by the Highway Research Board to identify the serviceability index or riding quality at which pavements are generally overlaid. For Region 4, which includes Idaho, and for primary flexible pavements, this survey indicates that Idaho pavements are most commonly overlaid when the riding quality reaches a level of 2.3. This is in good agreement with the findings from other states. As a guide to the Review Panel, it was suggested that a serviceability index of 2.5 be considered as the lower limit for heavy-duty, high-speed traffic.

c. The third step in the pavement condition survey procedure was to examine at close range the pavement condition. This close range examination consisted of driving the project at very slow speeds and by a walking examination. Continuous observations were made at slow speeds (15 mph) to identify the presence of distress. As a requirement, the Review Panel stopped along the road at several locations in order to make a walking inspection of a limited portion of the project. In some cases, this close examination resulted in a discovery of the presence of some form of distress not observable in a moving vehicle at 10 to 15 miles per hour. At each location at which the Review Panel made a walking inspection, they also noted on the survey form, by lane and wheel path, the presence of the various types of distress listed. At the same time, special remarks could be noted on page 2 of the pavement condition survey form indicating the suggested causes of distress as evaluated by the particular surveyor. Levels of distress were noted as 0 through 3, indicating no distress, to major distress. Major distress was defined as requiring immediate maintenance. At the conclusion of the visual examination for distress, the Review Panel stopped for purposes of summarization. This summary required each member of the panel to indicate his best judgment as to the level of distress, that is, its degree of seriousness, and the extent of distress, that is, the percent of the total area affected. At the same time, the Panel members were requested to indicate their general impression as to the surface appearance and particularly to indicate whether in their judgment the amount of asphalt was satisfactory or not.

The major types of distress noted by the panel are listed on the pavement condition survey form. In order to provide some consistency in identifying these types of distress, the following definitions were used:

Rutting - Rutting was generally evaluated by means of a string line stretched across the wheel tracks of the travelled lane. The degree of seriousness of rutting was a matter of subjective evaluation by the various panel members. However, in general, it was agreed that rutting less than 0.04 feet would be classified as either minor or moderate distress.

Ravelling - Ravelling was considered synonomous with pitting in this particular survey. This type of distress manifests itself by a loss of aggregate in the surface texture of the pavement and is an areal effect.

Spalling - Spalling was a progressive breaking away or cracking of the asphalt surfacing in the vicinity of a crack.

Corrugation - For purposes of this pavement condition survey, corrugations encompass two types of distress, namely, transverse undulations at regular intervals in the surface of a pavement or shoving as normally occurs in an unstable plant mix.

Cracking - Cracking was subdivided into six types of distress. For the most part, the types of cracking are sufficiently descriptive not to require further definition. A more complete description and discussion of types of cracks is contained in Section IV. Longitudinal cracking was divided into two types; namely, load associated and construction associated. Alligator cracks are generally associated with fatigue of asphalt concrete pavement. Ladder cracks are generally associated with cement treated bases. Shrinkage cracks can be associated with volume change in the subgrade in the structural elements of the pavement or in the asphalt concrete surfacing per se. Figure 1 illustrates the typical pattern of the various types of cracking associated with the pavement condition survey. This figure was included in the notebook provided each rater during the second rating period, together with the pavement condition forms.

The information from these forms was eventually collated in the offices of Materials Research & Development, Inc., and is included herein as Tables 1 and 2. The information tabulated in these tables is not necessarily representative of a majority expression of the members of the panel. Rather, it represents the observations of the panel. In most instances, if one or two members noted a particular type of distress, it has been recorded on the table. The amount of the distress noted on the tables is reduced arbitrarily in accordance with

TABLE 1 - SUMMARY OF PAVEMENT CONDITION SURVEY

PROJECT	Age	TRAFFIC		Riding Quality	TYPES OF DISTRESS										REMARKS
									CRACKING						
		ADT	Percent Commercial		Rutting	Ravelling	Spalling	Corrugations	Transverse	Longitudinal (Load)	Longitudinal (Construction)	Alligator	Shrinkage	Ladder	
District 1															
1. FI-1031(1) Malad So. - Utah Line	12	1700	14	1.9	0	0	1	0	2/B	2/A	P	2/C	1/C	0	See Case Study No. 1 of Research Project 24, Oct. 1963. Concluded ballast was inadequate, asphalt absorption measured at 1.9 to 2.4%.
2. FI-1031(3) Malad No.-Deep Creek	11	1700	14	1.8	0	0	0	0	2/B	2/C	0	2/B	2/C	0	See Case Study No. 1 of Research Project 24.
3. I-15-1(5)17 Deep Creek-Colton Lane	3	1750	14	3.3	0	1	0	0	0	1/B	0	2/B	0	0	Absorptive aggregate reported by Department personnel in this District.
5. I-15-1(18)70 Pocatello Cr. G.S. - Jct. 15W	4	3900	-	2.7	not examined for cracking, etc.										Absorptive aggregate reported by Department personnel in this District.
6. I-15-2(3)71 Jct. 15W-Chubbuck G.S.	4	4500	-	2.7	0	0	0	0	1/A	2/C	0	2/B	0	0	Test pits - stripping noted in lower course of asphalt plant mix in area of longitudinal crack.
7. I-15-2(6)71 Sec A Chubbuck G.S. - So. Blackfoot I.C.	5	4550	14-16	3.1	0	0	0	0	1/A	2/C	0	1/B	0	0	Test pits - stripping noted in lower course of asphalt plant mix in area of longitudinal crack.
8. I-15-2(6)71 Sec B Chubbuck G.S. - So. Blackfoot I.C.	5	4550	-	3.1	0	1	1	0	1/A	2/C	P	1/A	1/A	0	Test pits - stripping noted in lower course of asphalt plant mix in area of longitudinal crack.
10. I-IG-15-2(9)88 Sec B West Blackfoot I.C. - Porter Bridge G.S.	4	4200	-	3.6	0	0	0	0	0	0	0	0	0	0	Flushing in travel lane, northbound.
11. I-15-2(11)96 Sec A Porter Bridge G.S. - Gr. Western Canal	4	4200	-	3.4	0	0	0	0	0	1/C	0	0	0	0	Longitudinal cracks in both cut and fill.
12. I-15-2(11)96 Sec B Gr. Western Canal - Bonneville Co. Line	4	4200	-	2.8	1	0	0	0	2/B	2/C	0	0	0	0	Flushing in travel lane. Reported no stripping.

TABLE 1 - SUMMARY OF PAVEMENT CONDITION SURVEY

PROJECT	Age	TRAFFIC: ADT	TRAFFIC: Percent Commercial	Riding Quality	TYPES OF DISTRESS: Rutting	Ravelling	Spalling	Corrugations	CRACKING: Transverse	Longitudinal (Load)	Longitudinal (Construction)	Alligator	Shrinkage	Ladder	REMARKS
14. I-IG-15W-4(9)88 Rockland Jct. - Igo O.H.	3	2900	18.5	3.6	2	0	0	0	0	1/A	0	0	1/A	0	Intermittent flushing associated with tack coat - BST for 4/5 years.
15. I-15W-4(13)97 Rockland Jct. - Massacre Rocks	4	2300	18.5	3.3	0	1	0	0	1/B	1/A	P	0	0	0	BST for 4 years.
19. S-1721(4) American Falls Bingham Co. Line	7	910	15	3.2	0	0	0	0	2/C	0	0	0	0	0	Spacing of transverse cracks appears to be associated with ballast section.
20. S-1721(6) Power County Line - Aberdeen	3	800	15	3.1	1	0	0	0	1/A	1/C	P	0	0	0	Flushing - basalt aggregate
District 2															
47. I-80N-4(1)220 Jct. I-15W-80N - Cotterell	3	1200	-	3.1	0	1	1	See Remarks	1/B	2/C	P	0	0	0	Minor corrugations associated with construction. More cracking in southbound lane - Overlaid 1965 with 0.1 ft. plant mix containing lime for filler. Cracking reduced or eliminated in heavier ballast section on south end of project. Test pit - stripping in lower 0.2/0.3 ft. of plant mix in area of longitudinal crack.
49. F-2361(4) and S-2862(1) Burley Streets	8	--	-	3.2	0	0	0	0	2/A	0	0	0	0	0	Seal coat in good condition.
50. F-2371(2) Wendell - Gooding	7	1150	-	3.6	0	1	0	0	0	0	0	0	0	0	Fog seal in outer 10 ft. of each lane. Ravelling minimized in this area.
51. F-2441(7) Rupert Streets	5	4200	-	3.7	0	0	0	0	0	0	0	0	0	0	Seal coat in good condition.

TABLE 1 - SUMMARY OF PAVEMENT CONDITION SURVEY

PROJECT	Age	TRAFFIC: ADT	TRAFFIC: Percent Commercial	Riding Quality	Rutting	Ravelling	Spalling	Corrugations	CRACKING: Transverse	CRACKING: Longitudinal (Load)	CRACKING: Longitudinal (Construction)	CRACKING: Alligator	CRACKING: Shrinkage	CRACKING: Ladder	REMARKS
52. F-2441(8) Rupert - Heyburn	2	3800	-	3.8	0	0	0	0	0	0	0	0	0	0	
53. F-2441(15) 5 mi. East of Jct. U.S. 93 - Jct. SH50	4	2150	-	3.0	0	1	2	0	2/C	2/C	P	2/C	2/C	0	General reduction in cracking from north to southerly end; primarily longitudinal cracking at south end of project. Spacing of transverse cracks approximately 5 to 15 ft.
54. F-2441(20) Jerome East - U.S. 93	1	3900	-	2.8	1	0	0	See Remarks	0	0	0	0	0	0	Corrugations primarily at stop sign, are due to unstable mix for loads applied.
55. S-2741(1) Addison East	12	--	-	2.3	1	0	0	0	2/C	1/A	0	0	0	0	Pavement has received several seal coats including a slurry seal, absorptive aggregate in surfacing.
E. Cemetery to Kimberly (Not a part of programmed survey)	2	--	-	3.7	0	0	0	0	0	See Remarks	0	0	0	0	Seal coat placed as part of construction, except at easterly connection, some longitudinal cracking now evident in area without seal.
District 3															
57. I-80N-1(19)25 West of Caldwell - Jct. SE44	2	5340	8.6	3.3/3.1(1)	0	1	1	0	0	0	0	0	0	0	
58. I-80N-1(30)14 Sand Hollow-U.S. 30	5	3500	9.6	2.8/2.8(1)	0	0	0	0	0	1/B	0	0	0	0	Test pit - stripping in lower 0.2 ft. of plant mix in area of longitudinal crack.
59. I-80N-1(31)18 Jct. SH44 - Sand Hollow	4	3600	9.6	3.2/3.1(1)	0	1	0	0	0	2/C	0	0	0	0	Pumping of fines from underlying layers noticed on the surface. Test pit with stripping noted in lower 0.2 ft.
60. F-3022(2) Glenns Ferry - King Hill	9	3300	17.1	3.1	0	0	0	0	2/B	0	0	0	0	0	BST from 1952 to 1957, east end of project exhibits highest percentage of transverse cracks.

(1) Replicate rating

TABLE 1 - SUMMARY OF PAVEMENT CONDITION SURVEY

PROJECT	Age	TRAFFIC: ADT	TRAFFIC: Percent Commercial	Riding Quality	TYPES OF DISTRESS: Rutting	Ravelling	Spalling	Corrugations	CRACKING: Transverse	Longitudinal (Load)	Longitudinal (Construction)	Alligator	Shrinkage	Ladder	REMARKS
61. St-3111(513) Nyssa Jct. on U.S. 95 north to 80N	2	1200	14.9	2.8	0	1	1	0	2/B	See Remarks	0	0	0	0	T.I. = 8.5. Longitudinal cracking probably due to settlement in the fill.
62. F-3271(1)-CTB Round Valley-Cascade	11	980	14.5	2.2	1	0	0	0	2/C	2/C	0	2/C	0	0	Cement treated base - Transverse cracks 15 to 60 ft. apart, distinct increase in amount of cracking in southerly 4 miles of this project, also southbound lanes exhibit more cracking.
62. F-3271(1)-Aggr. Base Round Valley-Cascade	11	980	14.5	2.7	1	0	0	0	1/A	0	0	0	0	0	Seal coat on present surface.
District 6															
38. I-15-3(2)194 Montana Line So.	6	510	-	3.0	0	0	0	0	2/B	1/A	0	0	0	0	Part of Research Project 38 using styrafoam as insulation layer, transverse cracks spaced at 25 to 100 ft. Cracks in cut and fill.
39. I-15-3(8)142 Sage Jct.-Hamer	6	920	-	3.1	0	0	0	0	2/B	1/A	0	0	0	0	No cracks in cut section.
40. I-15-3(9)150 Hamer-So. of Dubois	3	820	-	3.3	0	0	0	1	2/A	1/A	0	0	0	0	Good looking mix, has lime filler.
41. I-15-3(21)163 Dubois-China Point	1	630	-	3.4	0	0	0	1	0	0	0	0	0	0	Corrugations measured at approximately 1 per 8 inches.
42. I-6033(4) Roberts-Sage Jct.	7/8	1700	-	3.3	0	0	0	0	1/A	1/B	P	0	0	0	Seal coat on present surface, transverse cracks in both cut and fill spaced from 25 to 1500 ft. apart.
44. F-6033(8) So. connection to Idaho Falls	3	4720	-	3.6	0	0	0	0	0	0	0	0	0	0	

TABLE 2 - SUMMARY OF BALLAST (STRUCTURAL) SECTIONS

PROJECT	SUBGRADE CLASSIFICATION (AASHO)	BALLAST (STRUCTURAL SECTION) SURFACE (FT)	BASE, (FT) ¾ inch	BASE, (FT) (1)	SELECT (FT)	TOTAL (FT)	Deflection	Stage Construction	Longitudinal Cracking Index	REMARKS
District 1										
1. FI-1031(1) Malad So. - Utah Line	A-1(a) A-2(6) A-4(7)	0.2	0.4	0.6	-	1.2	.029	-	2	Case Study No. 1 - Research Project 24, absorptive aggregate reported
2. FI-1031(3) Malad No.-Deep Creek	A-1(a) A-2(4)	0.2 0.2 0.2	0.4 0.4 0.4		1.05 .60 -	1.65 1.20 0.6	.033	Yes	6	
3. I-15-1(5)17 Deep Creek-Colton Lane	A-6 A-7(6)	0.3	0.4		1.2	1.9	-	Yes	2	
5. I-15-1(18)70 Pocatello Cr. G.S. - Jct. 15W	A-4(8)	0.3	0.4	0.5	-	1.2	.018	No	-	
6. I-15-2(3)71 Jct. 15W-Chubbuck G.S.	A-4(8)	0.3	0.4	0.5	-	1.2	.017	-	6	
7. I-15-2(6)71 Sec A Chubbuck G.S. - So. Blackfoot I.C.	A-4(8)	0.3 0.3	0.4 0.4	0.7 0.5	-	1.4 1.2	.012	Yes	6	Deflections reported from "Field Investigation"
8. I-15-2(6)71 Sec B Chubbuck G.S. - So. Blackfoot I.C.	A-1 A-4(1)	0.3 0.3	0.7 0.4	0.5	-	1.0 1.2	.011	-	6	Deflections reported from "Field Investigation"
10. I-IG-15-2(9)88 Sec B West Blackfoot I.C. - Porter Bridge G.S.	Gravel Borrow, 3'	0.3 0.3	0.4	- 1.2	(3.0)	0.7 1.5	.009	No	0	
11. I-15-2(9)88 Sec B Porter Bridge G.S. - Gr. Western Canal	A-1-a	0.3	0.4	0.8	-	1.5	-	Yes	3	

(1) Various maximum sizes ranging from 1 inch to 2 inches.

TABLE 2 - SUMMARY OF BALLAST (STRUCTURAL) SECTIONS

PROJECT	SUBGRADE CLASSIFICATION (AASHO)	BALLAST (STRUCTURAL SECTION) SURFACE (FT)	BASE, (FT) ¾ inch	BASE, (FT) (1)	SELECT (FT)	TOTAL (FT)	Deflection	Stage Construction	Longitudinal Cracking Index	REMARKS
12. I-15-2(11)96 Sec B Gr. Western Canal - Bonneville Co. Line	A-4(8)	0.3	0.4	0.4	0.8	1.9	.012	Yes	6	
14. I-IG-15W-4(9)88 Rockland Jct. - Igo O.H.	A-4(8)	0.4	0.4	0.5	-	1.3	-	-	1	
15. I-15W-4(13)97 Rockland Jct. - Massacre Rocks	A-4(8)	0.3	0.4	0.6	0.6	1.9	-	No	1	
19. S-1721(4) American Falls Bingham Co. Line	A-2-4	0.2 0.2	0.35 0.35	-	1.45 0.45	2.0 1.0	-	Yes	0	
20. S-1721(6) Power County Line - Aberdeen	A-2-4	0.2	0.4	-	0.9	1.5	-	Yes	3	
District 2										
47. I-80N-4(1)220 Jct. I-15W-80N - Cotterell	A-4(3) to A-6(8) (NP-PI 11)	0.4	0.4 0.8	0.8 1.3	-	1.6 2.5	-	-	6 -	Original thickness of plant mix was 0.3 ft. overlaid after 2 years of service.
49. F-2361(4) and S-2862(1) Burley Streets	A-1 to A-6(9) (NP-PI 12)	0.3	0.4	0.7	-	1.4	-	-	0	
50. F-2371(2) Wendell - Gooding	A-2-4 to A-4(8) (NP-PI 8)	0.25	0.4	0.35 0.85 1.15	-	1.00 1.50 1.70	-	-	0 0 0	Stopped at locations believed to representative of various thicknesses and found no identifiable variation in performance.

(1) Various maximum sizes ranging from 1 inch to 2 inches

TABLE 2 - SUMMARY OF BALLAST (STRUCTURAL) SECTIONS

PROJECT	SUBGRADE CLASSIFICATION (AASHO)	BALLAST (STRUCTURAL SECTION) SURFACE (FT)	BASE, (FT) ¾ inch	BASE, (FT) (1)	SELECT (FT)	TOTAL (FT)	Deflection	Stage Construction	Longitudinal Cracking Index	REMARKS
51. F-2441(7) Rupert Streets	A-1-b to A-4(0) (NP-PI 5)	0.2	0.4	0.2	-	0.8	-	-	0	Seal coat placed after 5-6 yrs. of service.
52. F-2441(8) Rupert - Heyburn	A-2-4	0.3	0.4	0.3 0.6	-	1.0 1.3	-	--	0	
53. F-2441(15) 5 mi. East of Jct. U.S. 93 - Jct. SH50	A-1-a to A-4(8) (NP-PI 7)	0.2	0.8		-	1.0	-	Yes	6	Base placed in 1954. Highly absorptive aggregate (vesicular) used in plant mix.
54. F-2441(20) Jerome East - U.S. 93	A-4(2) to A-4(8) (NP-PI 9)	0.2	0.3	0.3 0.5 0.6	-	0.8 1.0 1.1	-	Yes, on West end of project.	0	
55. S-2741(1) Addison East	A-4(5) to A-4(8) (NP-PI 6)	0.2	0.2	0.6	-	1.0	-	-	1	Extensive transverse cracking.
District 3										
57. I-80N-1(19)25 West of Caldwell - Jct. SE44	A-1-b to A-4(6)	0.3	1.1	1.0	-	2.4	-	No	0	T.I. of 9.0.
58. I-80N-1(30)14 Sand Hollow-U.S. 30	A-1-b to A-4(6)	0.3	0.8	0.8	-	1.9	-	No	2	T.I. of 9.0.
59. I-80N-1(31)18 Jct. SH44 - Sand Hollow	A-1-b to A-4(6)	0.3	0.8	-	0.8	1.9	-	-	6	
60. F-3022(2) Glenns Ferry - King Hill	-	0.3	0.8	-	1.1	2.2	-	-	0	

(1) Various maximum sizes ranging from 1 inch to 2 inches

TABLE 2 - SUMMARY OF BALLAST (STRUCTURAL) SECTIONS

PROJECT	SUBGRADE CLASSIFICATION (AASHO)	BALLAST (STRUCTURAL SECTION) SURFACE (FT)	BASE, (FT) ¾ inch	BASE, (FT) (1)	SELECT (FT)	TOTAL (FT)	Deflection	Stage Construction	Longitudinal Cracking Index	REMARKS
61. St-3111(513) Nyssa Jct. on U.S. 95 north to 80N	-	0.2 0.4 See Remarks	-	-	-	-	-	-	0	Existing roadway with approx. 0.4 ft. of asphalt road mix overlaid with 0.2 ft. of plant mix.
62. F-3271(1)-CTB Round Valley-Cascade	A-1-b to A-2-4	0.2	0.5 See Remarks	-	1.0	1.7	0.036	-	6	Cement treated base.
62. F-3271(1)-Aggr. Base Round Valley-Cascade	A-1-b to A-2-4	0.2	0.8	-	1.0	2.0	0.018	-	0	Aggregate base.
District 6										
38. I-15-3(2)194 Montana Line So.	A-1-a to A-7	0.3	0.4	0.6 0.9	-	1.3 1.6	-	No	1	
39. I-15-3(8)142 Sage Jct.-Hamer	A-3	0.3	0.3 0.4	-	0.4 -	1.0 0.7	-	No	1	
40. I-15-3(9)150 Hamer-So. of Dubois	A-1-a, A-6(7) and A-3 (dominant)	0.3	0.4 0.6 0.4	-	0	0.7 0.9 1.6	-	No	1	
41. I-15-3(21)163 Dubois - China Point	A-4 to A-7	0.3	0.4 (AB) to 0.4 (ATB) See Remarks		0.8 to 2.0	1.5 to 2.7	-	No	0	Six different ballast sections on this project including asphalt treated base, no noticeable difference recorded. AB - aggregate base ATB - asphalt treated base
42. I-6033(8) Roberts-Sage Jct.	-	0.3	0.5	-	-	0.8	-	-	0	
44. F-6033(8) So. connection to Idaho Falls	A-4	0.3	0.4	0.6 1.1	-	1.3 1.8	-	Yes	0	

(1) Various maximum sizes ranging from 1 inch to 2 inches.

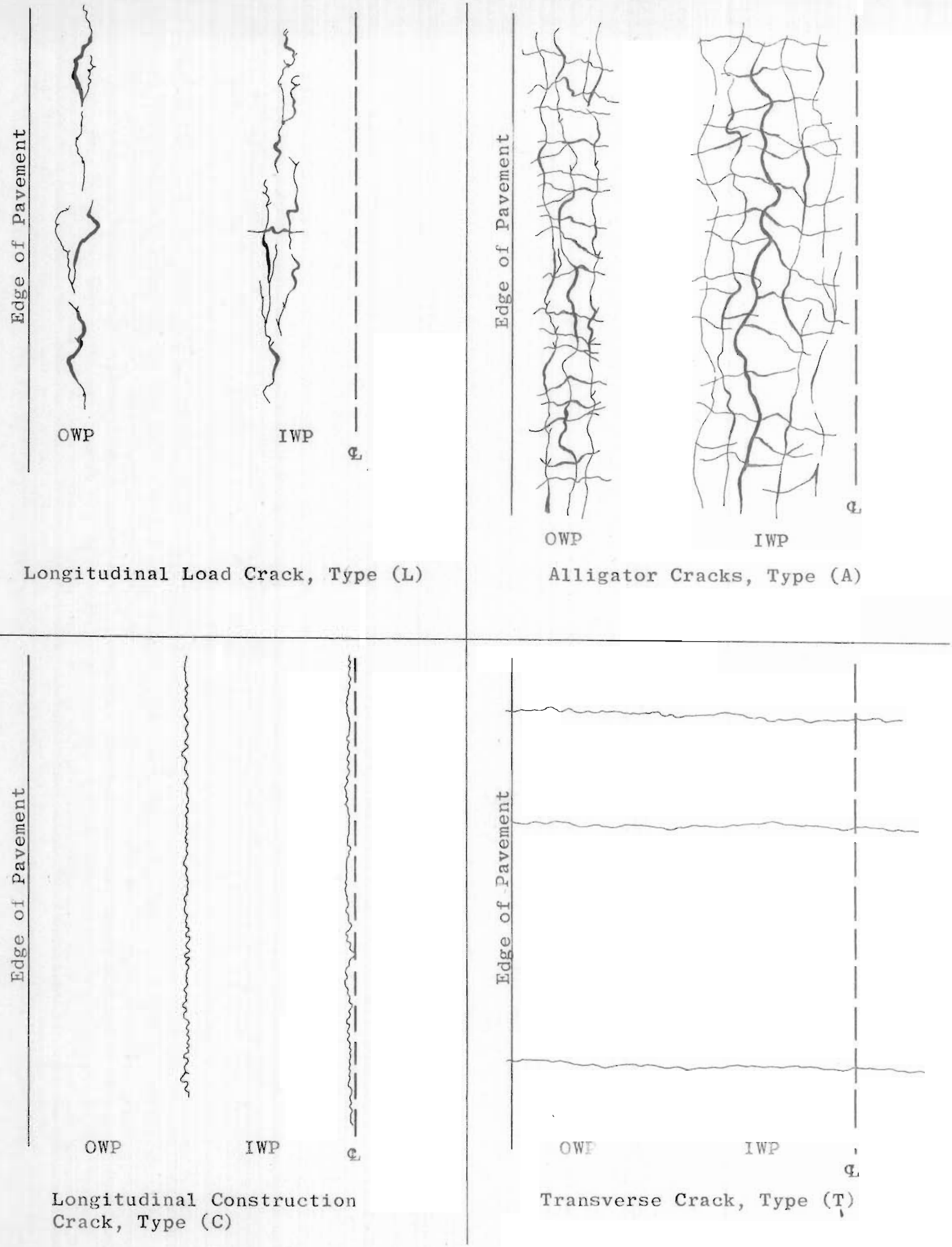
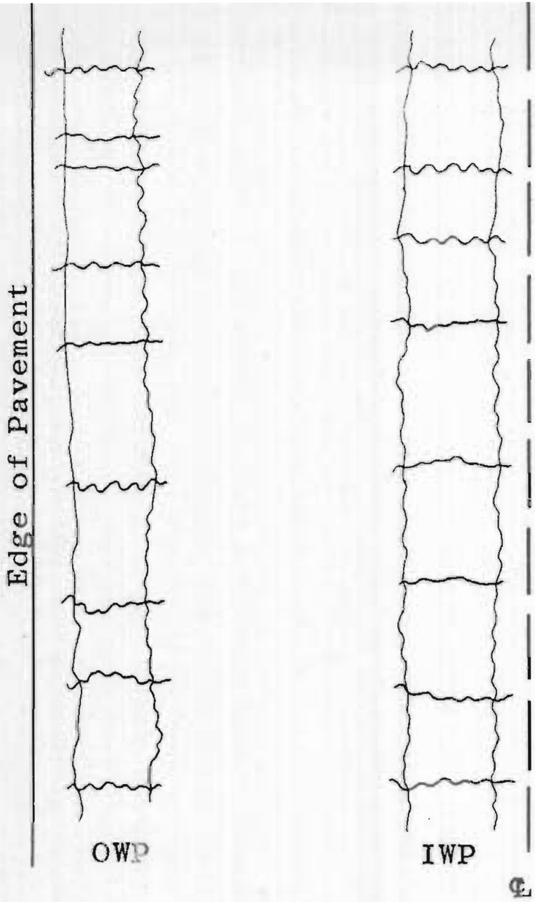
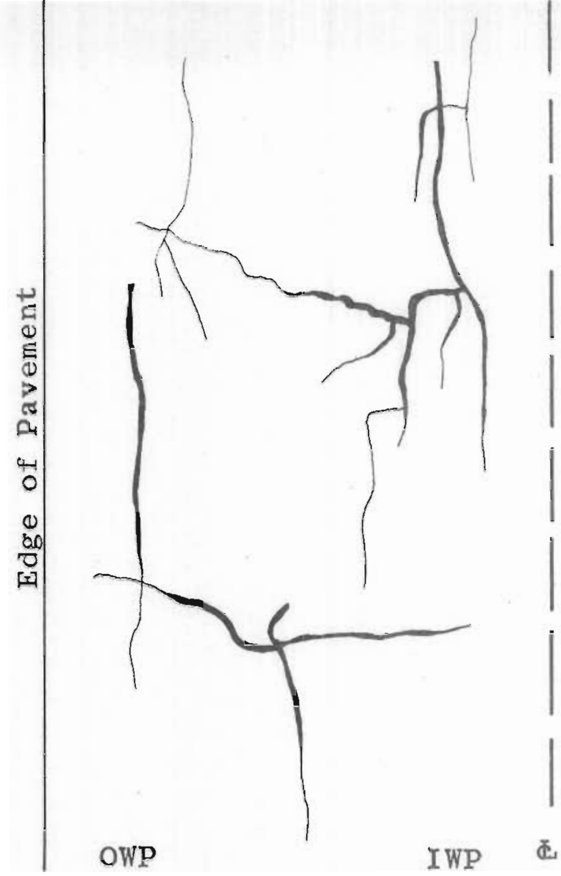


FIGURE 1a - TYPICAL PATTERNS USED TO IDENTIFY TYPES OF CRACKING



Ladder Cracks, Type (D)



Shrinkage Cracks, Type (S)

FIGURE 1b - TYPICAL PATTERNS USED TO IDENTIFY TYPES OF CRACKING the number of panel members noting a particular type of distress. Admittedly, no serious effort has been made in this regard to be precise. Rather, it was considered more important to report the opinions rather than to assess their particular value.

In summary, the pavement condition survey is designed to identify the types of pavement distress which are occurring on the selected pavements included in this field survey. Hopefully, this information would also provide some indication as to the causative factors relating to the various types of distress encountered. This information was collated and attempts were made to correlate it with physical measurements of material properties. The results of this correlation will be discussed in a subsequent section. It is pertinent to point out that single observations of a pavement condition does not, in itself, provide a good indication of pavement performance. To do this with confidence requires a series of observations or measurements.

IV. RESULTS OF FIELD INSPECTION OF SELECTED PROJECTS

The primary objective of the field condition survey was to obtain an understanding or appreciation of the types of distress most prevalent on a representative portion of the State highway system. It was hypothesized in planning the project that the information from the field inspection would serve as the basis for an analytical evaluation of design and construction factors most critically associated with performance and would help eliminate certain factors as being of only minor importance.

As will be shown, the field inspection did tend to isolate the major problems relative to performance in such a way as to put the analysis and interpretation in perspective. However, attempts to correlate laboratory test data with field performance were not particularly successful, and, instead, point up an area which may require some additional effort if pavement inventory programs are to be useful for studies of the type described herein. The procedure used to conduct the field inspection has been described in the previous section under Development of Condition Survey Procedure.

A second, and more difficult, objective of the field inspection and condition survey was an attempt to obtain some indication as to the causative factors associated with the various types of distress. It was hoped that subjective estimates of the cause of distress, based on individual experience would be helpful to the analysis and interpretation phase of the project. Two procedures were used to obtain information relative to the cause of distress. Initially, each type of distress was subdivided into the typical causes commonly assigned by highway engineers. For example, the presence of alligator cracking would usually lead to the conclusion that (a) the ballast (structural) section was inadequate, (b) the aggregate layers of base or borrow were too resilient, (c) the pavement was exhibiting excessive deflection, etc. Each factor was numbered and a tabulation furnished to the members of the Review Panel in their Condition Survey notebook. During the close examination phase of the survey, each rater would enter a number or numbers indicating his best estimate of the cause of distress. Estimates were usually made at several locations along the project. This method did not prove to be effective. In lieu of this approach, the procedure was modified to provide for extensive remarks by the rater. This proved to be a better way of obtaining the benefit of the experience of the various panel members as to the mechanism involved in the various types of distress.

The field condition surveys were made during the weeks of August 8-13 and August 29-September 3, 1966. The condition surveys were made by the permanent members of the Review Panel (Swagler, Clayton, Sylvies and Finn) with F. N. Hveem and B. A. Vallerga participating in alternate weeks.

A total of 35 separate projects in Districts 1, 2, 3, and 6 were included in the survey. The location of these projects is shown on Figure 2. Some additional projects in these districts were to have been examined, and some were planned in Districts 4 and 5. However, examination was not feasible due either to the remoteness of their location or, as was the case for most, recent maintenance in the form of overlays or seal coats made it impractical to conduct an inspection. The coverage obtained would appear to include the range of climatic conditions encountered in the State of Idaho. Reference (25) indicates that projects examined in Districts 3 and 6 are in areas in which the assigned Regional Factor is 1.15, the maximum multiplier assigned for climatic conditions.

Some constructions, not specifically included in the selected listing, were examined by the Panel as the opportunities presented themselves. For example, a reconstruction job in the vicinity of Mountain Home was examined as was a one-year-old project at Hagerman. This latter project probably had the smoothest ride of any pavement examined by the Panel. It appeared to have a slight excess of asphalt and possibly a gap grading in the mix. Two construction projects were observed in the course of the field inspection.

The remaining portion of this section will summarize the results of the condition survey including some subjective indication as to the cause of the various types of distress observed. Pertinent information associated with these observations is shown on Tables 1 and 2. The numbering system used for each project is in two parts: (1) a simple numerical identification used by MR&D for convenience in scheduling and tabulating and (2) an extended system of letters and numbers used by the Department. The MR&D numbers are not continuous due to changes made during the progress of the work. The summarization has been divided into subsections, each dealing with a particular type of distress.

Riding Quality - The evaluation of riding quality for each project was the average value of the Review Panel members. As was previously indicated, a general guideline suggested to the Panel for grading the riding quality was that a value of 2.5 or less would not be suitable for high-speed, heavy-duty traffic. Four of the selected projects (11%) had average values of less than 2.5. The overall average rating was 3.1 and ranged from 1.8 to 3.8. Three sections were rated twice (replicated) with intervals between ratings of approximately one month. These three sections in District 3 were: I-80N-1(19)25, I-80N-1(30)14 and I-80N-1(31)18, all west of Caldwell. The replicate ratings are noted on Table 1 and show a very close comparison by the Review Panel. The information contained in Table 1 will be analyzed further in Section V of this report.

Rutting - An observable amount of rutting was reported on seven projects or 20 percent. Of these seven, only one or 3 percent was categorized as "moderate." Moderate rutting was approximately 0.02 feet in the wheel path. Based on these observations, rutting, per se, does not appear to be a serious problem to the performance of the pavements examined. Conceivably, increased volumes of heavy traffic could develop more rutting, and hence, continued efforts should be made to minimize this factor; however, for the present, it poses no need for revisions in design or construction procedures.

Ravelling - Minor amounts of surface ravelling were noted on nine projects or 25 percent. For the present, it should be possible to take care of this problem with various types of seal coats. For example, the project from Wendell to Gooding, F-2371(2), showed minor ravelling which appeared to have been arrested by a fog seal.

Of some concern should be the rather high frequency of uncoated aggregates noted on the surface of the plant mix projects. Poor aggregate coating, possibly associated with stripping, and low asphalt content are suggested as the major causes for ravelling. This manifestation of performance could lead to serious ravelling and the need for early maintenance in the form of seal coats. Better long-term coating of plant mix aggregates was indicated to be necessary if ravelling is not to be a problem.

Spalling - Spalling was noted if the plant mix was exhibiting sympathy cracks in conjunction with the longitudinal and transverse cracks present on a particular project. Six of the projects (17 percent) were noted to have some observable amount of spalling. Of these, only one (3 percent) was noted to have progressed to the point where maintenance should be considered.

In general, spalling does not appear to be a serious problem, and no particular trends were noted between spalling and age, asphalt content, or asphalt source. To some extent, this lack of correlation may be due to the relatively small amount of spalling noted on the selected projects. No particular causative factors are suggested with regard to spalling.

Corrugations - This type of distress is usually associated with transverse undulations at regular intervals. Corrugations are often thought to be a result of some construction deficiency. For purposes of this condition survey, corrugations were defined to include transverse undulations associated with construction and plastic deformation due to instability of the plant mix. Precise identification has been included in the remarks column of Table 1.

Only one project (3 percent) (District 2, Jerome East - U.S. 93) was noted to exhibit plastic deformation. This occurred mostly at the stop sign on the east end of the project where stress conditions are most severe. It is interesting that some five projects exhibited an excess of asphalt, as noted by "flushing," in the remarks of the raters. No plastic deformation was noted on these projects. Increased volumes of heavy traffic could, conceivably, result in plastic deformation depending on the shear strength available from the plant mix and the untreated base. It is axiomatic in plant mix designs that a delicate balance exists between: (a) asphalt content and stability, (b) asphalt content and durability (ravelling, spalling, etc.) and (c) adequate asphalt for good fatigue properties. Hence, it becomes extremely important that the asphalt content for plant mixes be considered of critical importance to quality control. There was some indication on a number of projects examined that radical changes in asphalt content were being made at the asphalt plant. This was particularly true on the Jerome East project. Further discussion of this item will be made in the recommendations dealing with construction control.

Three projects (8 percent) were noted to have corrugations of the type generally associated with laydown or placing of plant mix with a paving machine. These corrugations were of little or no consequence to the riding quality.

Cracking has been subdivided into six categories. Table 1 summarizes cracking in two ways. The numerical value indicates the progression of the crack; 0 - not observed, 1 - minor (observable), 2 - moderate (estimated to need future maintenance), and 3 - major (maintenance required). The letter indicates the areal extent of the cracking as follows: A - 0 to 5 percent of area, B - 5 to 25 percent of area, and C - over 25 percent of area. This rather imprecise evaluation is due to the basic procedures adapted for the survey; however, it does provide some useful information for analysis purposes. In the case of longitudinal cracks associated with construction, a notation of P (present) or 0 (not observed) is used to summarize the presence of this type of distress. Transverse cracking is identified by number and letter even though it does not lend itself to an areal classification. In this case, the letter denotes the relative spacing from A, approximately 100 feet apart, to C, approximately 5 feet apart. A notation of 1 indicates a hairline crack, 2, an opening of approximately 1/4 to 3/8 inch, and 3, in excess of 3/8 inch.

Transverse cracking - Some type of transverse cracking was detected on 21 projects or 60 percent. Typical cracking of this type is illustrated in Figure 3. These cracks usually extended across the full width of the paved surface. There were some exceptions to this observation in which the transverse crack would stop at the centerline and displace itself several feet longitudinally before proceeding across the pavement.

FIGURE 3 - TRANSVERSE CRACKING IN DISTRICT 6

The transverse cracks characterize themselves in two ways. First, by the width of opening, and second by their longitudinal spacing along the roadway. The spacing varies from 5 feet to several thousand feet. This variation in spacing is believed to be a function of the thermal stresses induced in the mix and the tensile strength of the mix or the absorptive properties of the aggregate. Transverse cracks due to shrinkage of the base are a unique characteristic of cement treated bases and are usually easily identified. It is significant to note that the average riding quality for these 21 projects was 3.0. This is 0.4 lower than the average value of 3.4 for sections without transverse cracking. In general, this would indicate that pavements associated with transverse cracks do have a slightly lower riding quality than pavements free of this type of distress. However, this conclusion appears to be somewhat weakened by the fact that the lower average is controlled by the inclusion of two of the older projects with low ratings and all types of distress.

On thirteen of the projects (37 percent), transverse cracking had progressed to the "moderate" stage indicative of some consideration for future maintenance. Examination of Table 1 indicates that transverse cracks are most prevalent in the older pavements. There are some exceptions to this observation making it difficult to draw any strong conclusions.

Overall, transverse cracks, per se, do not appear to be particularly damaging to the performance of the roadway. That is, those sections representative of recent construction in which transverse cracking is the dominant form of distress have retained a relatively high level of riding quality and do not appear to be in any serious need of maintenance. This is particularly true of the projects examined in District 6, as can be noted on Table 1.

The cause of transverse cracks of the types observed can generally be reduced to four factors, as follows: (1) thermal stresses in the plant mix, (2) shrinkage (thermal or otherwise) in the underlying materials, (3) absorptive aggregates, or (4) excessive deflection associated with wheel loading. It is believed that the irregular spacings of from 25 to 100 plus feet, normally associated with transverse cracks, is due to one or both of the first two factors noted. For the closer spacing, such as was noted on F-2441(15), five miles east of Junction U.S. 93 to Junction SH50, in District 2, it is believed that aggregate absorption is playing a key role. This judgment is based in part on work by the California Division of Highways (<u>26</u>), indicating an association between aggregate absorption and cracking. For the above mentioned aggregate, the water absorption was reported to be in excess of 3 percent.

In summary, transverse cracking is present on a large percent of the pavements surveyed but does not appear to be highly detrimental to the performance of the pavement. In those cases involving absorptive aggregates, the transverse cracks appear to have progressed to much closer spacing, higher frequency, and as such, pose a significant problem. The effects of frost heave have not been included in this report since the evaluations were made during a period when such effects were not present. Reports by Department personnel indicate that a substantial reduction in riding quality occurs in some areas when the subgrade materials freeze. For a study of the influence of frost heave, Idaho's Research Project 38 should prove informative and potentially beneficial. For the absorptive aggregate, some further studies are required.

Longitudinal (load associated) - Longitudinal cracking has been subdivided into two types, load associated and construction associated. The load associated longitudinal crack is relatively easy to identify. It occurs in the vicinity of the wheel path, and in the case of a four-lane divided highway, it occurs only in the travel lane.

The load associated longitudinal crack is the dominant type of distress encountered on the field survey, considering its rate of occurrence, its progression to more serious forms of distress and its areal extent. A class 2 longitudinal load crack is illustrated by Figure 4. In this particular case, some spalling can be observed along the crack together with some progression to block cracking. Another form of this type crack is shown in Figure 5. In this case, too, cracks are appearing in the general vicinity of the outer wheel path. Examination of the field projects indicates the initial longitudinal load crack can progress to the type of distress noted in Figures 6 and 7. In either case, major maintenance is necessary.

Twenty-two projects (63 percent) were noted to have some observable longitudinal load cracking. Eleven (31 percent) of these were classified as in the moderate stage, indicating the need for programmed maintenance. Longitudinal, load associated, cracks are considered more serious than transverse cracks, which are present on about the same number of projects, because they tend to rapidly progress into block cracking normally identified as alligator cracking. The consequences of this type of distress are considered to be serious, requiring an extensive amount of overlay in order to salvage the basic investment. In the case of the Cotterell project (I-80N-4(1)220 Jct. I-15W-80N), it would seem that 0.1 feet overlay is inadequate. After one year from the time the overlay was placed, a significant amount of longitudinal cracking was reflecting through the new surface. The eastbound lanes of I-15W-5(6)118, Snake River-Raft River, were

FIGURE 4 - LONGITUDINAL LOAD CRACK IN THE INNER WHEEL PATH

FIGURE 5 - LONGITUDINAL LOAD CRACK IN THE OUTER WHEEL PATH

FIGURE 6 - GENERAL DETERIORATION DUE TO PROGRESSIVE DEVELOPMENT OF LONGITUDINAL LOAD CRACK

FIGURE 7 - GENERAL PROGRESSION OF LONGITUDINAL LOAD CRACK TO ALLIGATOR CRACK ELONGATED ON LONGITUDINAL AXIS

reported overlaid with 0.2 feet of plant mix in 1965, and examination by Department personnel in 1966 indicated the surfacing to be free of cracks. This project should be examined carefully to verify this report.

Based on the field condition survey, at least three different observations are considered significant in trying to obtain an estimate of the cause of this type of distress, as follows: (1) they appear to be load associated, (2) they do not appear to be related to the subgrade soil type, strength or construction, and (3) they are highly associated with stripping of the asphalt from the aggregate in the lower portions of the plant mix.

The load association observation is based largely on the absence of this type of crack in the passing lane of multi-lane highways and in the wheel path orientation of this crack. There seems to be little doubt that this cracking is produced by the application of wheel loads to the pavement. It is concluded, largely on the basis of observations by Department personnel, that the longitudinal cracks manifest themselves predominantly in the Fall or Spring during the cold periods of the year. Thus, the induced thermal stresses together with the live loads (wheel loads) cause the initial crack which weakens the pavement and leads to progressive extension of the crack. It is hypothesized that the Spring period could be particularly critical since this represents the period of the year when the aggregate base could be in its most resilient condition. The work by Housel would indicate that even frost-free base materials are susceptible to loss of strength due primarily to the presence of freezing temperatures in these layers. This would tend to indicate the need for an increased ballast section and particularly an increased thickness of the treated layer.

Observations of plant mix cracking over large concrete box culverts on Interstate 15 between Pocatello and Blackfoot leads to the conclusion that the strength or construction of the subgrade may not be a critical factor in the occurrence of longitudinal cracking. In three instances, a walking examination indicated that the longitudinal cracking in the plant mix continued across the box culvert. In each case, the ballast section was composed of base or base plus select borrow to a depth of approximately 1 foot. It should not be concluded that this implies that subgrade construction is not important to the performance of a pavement. However, in the case of the longitudinal crack, apparently the character of the subgrade is not, per se, responsible for the presence of this type distress and, therefore, in trying to cope with this problem, attention should be concentrated on the surface, base, and subbase or select import portion of the roadway.

In an effort to obtain a better understanding of the type and character of materials used on various highway projects, a series of test pits were excavated in five of the selected projects. These projects were, as follows:

District 1

I-15-2(3)71, Junction 15W - Chubbuck G.S.
I-15-2(6)71, Sec. A, Chubbuck G.S. - So. Blackfoot I.C.
I-15-2(6)71, Chubbuck G.S. - So. Blackfoot I.C.

District 2

I-80N-4(1)220, Junction I-15W-80N-Cotterell

District 3

I-80N-1(30)14, Sand Hollow - U.S. 30
I-80N-1(31)18, Junction SH44 - Sand Hollow

A total of 20 areas were examined and in each area two test locations were selected. The test locations were oriented transverse to one another and were normally in opposite wheel paths of the travel lane. In each area, one excavation was in an area exhibiting longitudinal cracking while the companion excavation was in an area free of cracking.

In every case examined, stripping of the lower portion of the plant mix was observed in the test location associated with longitudinal cracking. Figure 8 is fairly typical of the condition found in each test area. This photo is from the Cotterell project in District 2. The two samples represent plant mix from the inner wheel path (left) showing stripping and from the outer wheel path (right) without stripping. On this particular project, a 0.1 foot plant mix overlay had been

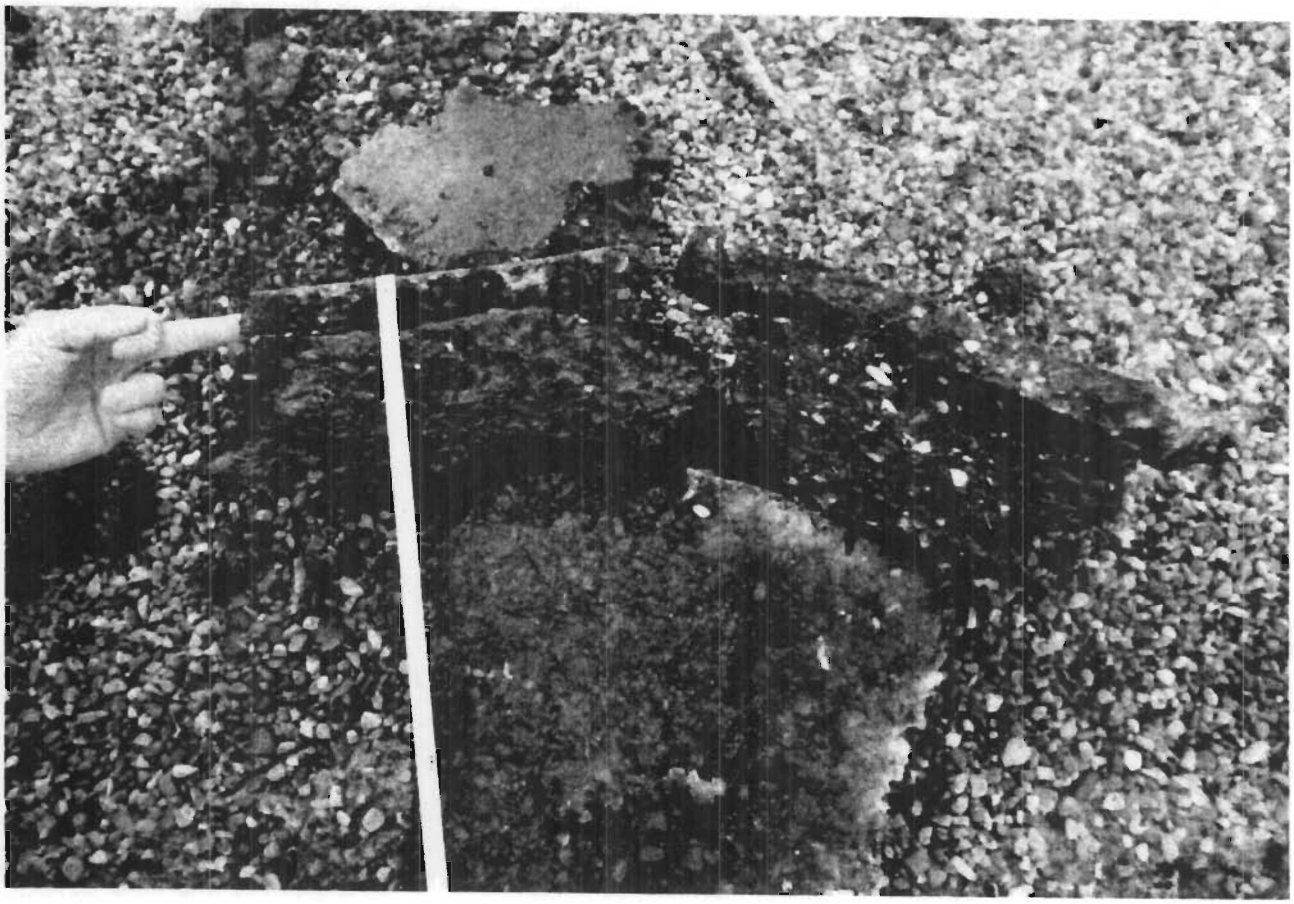


FIGURE 8 - EVIDENCE OF PLANT MIX STRIPPING ON I-80N-4(1)220 - COTTERELL

placed approximately one year prior to the time this photo was taken. The overlay can be seen in the photo as the upper slab. The overlay was rather poorly bonded to the original construction. The crack apparent on the surface of the pavement was continuous through the total depth of the plant mix. It can also be noted in the photo that the fracture face of the unstripped sample includes a high percentage of fractured aggregate. This was not observable in the stripped specimen. In fact, the plant mix exhibiting stripping on all projects would usually crumble or fall apart rather than fracture. It is interesting to note that on the Cotterell job, stripping had progressed through the full 0.3 feet of plant mix including the dense graded upper course of the original construction.

The net effect of the stripping in the plant mix is to reduce the gravel equivalency of the ballast section. In most cases examined, at least 0.2 feet of plant mix has been stripped. This would reduce the gravel equivalency of the ballast section by 0.2 feet (using 2:1 substitution ratio). To illustrate the effect this would have on the expected life of a pavement, consider a section composed of 0.30 feet of plant mix on 0.4 feet of aggregate base as a stage 1 construction. Using Idaho's substitution ratios for a T.I. greater than 7.0, the gravel equivalency would be 1.0 feet if no stripping is assumed, or 0.8 feet if stripping is assumed to reduce the cohesion to that of an aggregate base. From

Figure 16-231.3 of the Surveys and Plans Manual, Part 16, of the Idaho Department of Highways, it is possible to obtain some assessment of the consequences of stripping in terms of the traffic life of the pavement. For example, for subgrades with an R-value of 10, the traffic index (T.I.) would be reduced in excess of one half a T.I. point, say from T.I. 9 to T.I. 8.3. In terms of equivalent wheel loads, this is a reduction of from 11.9×10^6 to 7.8×10^6 or 45 percent. Put another way, the life expectancy would be approximately one half of the planned life. A similar computation for subgrades of higher R-value would indicate an even greater reduction in traffic life. For an R-value of 50, the reduction would be from 11.9×10^6 to 4.45×10^6 or 63 percent.

Thus, if the planned life of stage 1 design and construction is 10 years, the presence of stripping could reduce the expected life to some 3-5 years as a function of the subgrade R-value.

Admittedly, the interpretation applied above is subject to some errors, as for example, the stripping process is obviously a gradual one, and hence, the reduction in gravel equivalency is also gradual. Possibly a more analytical analysis would suggest a 50 percent or 0.1 foot reduction in gravel equivalency. In any event, it would seem rather apparent that the consequences of stripping can be quite significant in terms of the life of the pavement.

In order to further interpret the significance of stripping, it is necessary to consider the mechanism of longitudinal cracking. A number of factors must be considered in order to form a hypothesis as to the mechanism involved.

a. The longitudinal crack quite frequently, and probably in a slight majority of cases, occurs first in the inner wheel path in the general location of the construction joint for the lower course of the two layer plant mix surfacing. Because of the straight line characteristic of the longitudinal crack at its early development stage, some thought and study was given to the possibility that this was a reflection crack from the construction joint in the lower layer.

Department engineers have studied this problem rather thoroughly, and the results are not conclusive. For example, in a letter report by O. C. Grunerud and L. F. Browning, dated July, 1964, prepared in connection with Research Project 24, it would appear that in some cases the longitudinal crack was almost directly over the construction joint. In an almost equal number of cases, or possibly somewhat more, the crack would appear to be displaced several feet transversely from the joint. However, in the great majority of the cases shown, the construction joint is in the zone of the pavement generally described as the inner wheel path.

For the present, in view of the inconclusive association between the longitudinal crack and the construction joint, it is concluded that this is not necessarily a reflection crack. It is, however, suggested that there is an association in terms of the possible weakened plane produced by the construction joint. The fact that the longitudinal crack sometimes occurs first in the outer wheel path indicates this is a structural problem. The presence of a weakened plane under the inner wheel path tends to increase the odds that the initial crack will occur in this area; however, if significant variations in the in-situ material properties are such as to produce a weaker section in the outer wheel path, the initial crack will occur in that area.

The fact that the initial load associated crack is frequently in the longitudinal direction indicates that the critical stress or bending is in the transverse direction. Studies of the

WASHO Road Test (27) and on the Shell Avenue Test Road (28) would tend to indicate that this is the critical case. It is possible that this would help explain the lack of correlation between cracking and deflection since deflection measurements are made in the longitudinal direction.

Thus, it is hypothesized that the longitudinal crack is a manifestation of two factors: (a) the application of vehicle loads and (b) a structural weakness causing excessive stresses in the transverse direction. This structural weakness may be more a matter of the strength of materials than thickness; however, both factors must be considered. The most probable cause for the strength deficiency is the stripping observed in the plant mix.

b. The presence of stripping has been rather firmly established, at least across the southern portion of Idaho, by the test pits previously described.

The cause of stripping is difficult to assess at this time. There are at least three factors to consider: (a) in approximately 90 percent of the test locations, stripping was only noticed when a longitudinal crack was observable on the surface, (b) the upper course did not exhibit stripping, and (c) the moisture causing stripping is believed to be originating from below rather than through the plant mix.

The conclusion as to the origin of the moisture is tenuous. The main evidence of the moisture source is based on the laboratory analysis made by MR&D, indicating the presence of soluble salts in the water found in the Cotterell plant mix. It is believed that these salts originated in the natural ground underlying the pavement. It is also concluded that the water moves up as moisture vapor into the plant mix and is partially trapped by the dense graded upper surface course. It is believed that the moisture vapor is diffused through the films of asphalt spread over the aggregate surface. If the aggregate surface is "hydrophilic", the moisture will tend to condense on this surface and loosen or separate the bond with the asphalt.

It is tentatively concluded that the moisture vapor tends to collect in the zones of the coarse graded aggregate or in areas of relatively high void content. The longitudinal cold joint in the lower course would be a zone in which the moisture could move and collect with considerable ease.

In summary, the hypothetical mechanism leading to cracking is as follows:

a. The structural thickness of the surface and base is considered to be marginal.

b. Any loads applied during the critical periods of the year tend to develop stresses approaching the fracture strength of the plant mix.

c. With time, moisture tends to accumulate in the lower part of the plant mix causing stripping, particularly in (a) zones contiguous to the construction joint, (b) coarse graded mixes, or (c) mixes with high void content.

d. The stripping causes a reduction in the tensile properties of the lower portion of the plant mix resulting in an increase in the stress in the surface layer. Fracture occurs as a combination of the marginal ballast section and the reduction of gravel equivalency of the plant mix layer.

Admittedly, this explanation lacks substantial documentation in the form of hard facts. However, the conclusion to be drawn from the overall observation is that the stripping phenomenon must be stopped and that conditions either as to geographical location, aggregate source or type, asphalt source or type must be identified in the laboratory prior to establishing mix designs for specific jobs. Specific recommendations will be made in Section VI herein.

Longitudinal (construction) - Longitudinal cracks of this type are exclusively construction joints identifiable by their location and by their characteristic straight line. Seven projects (21 percent) were observed to exhibit cracking of this type. These cracks were not considered to be serious in terms of performance. For the most part, the longitudinal joints were considered to be in excellent condition.

Alligator cracking - Alligator cracking is generally defined as interconnected cracks forming a series of small polygons which resemble an alligator's skin (2). The axes of this type of crack may be of equal dimension or may be elongated. Except on the two projects near Malad, in District 1, the project five miles east of Junction U.S. 93 to Junction SH50 (F-2441(15), in District 2 and the cement treated project in District 3, cracks noted as being of the alligator type had a definite longitudinal elongation. It is believed that with the possible exception of these four projects, the observed alligator cracking is a progression of the longitudinal load cracks. Eight projects (23 percent) were observed to have alligator-type cracking.

The project in District 3 is worthy of some further discussion. The Department has made a thorough study of the section as part of Research Project 24 with a report issued in April, 1966. The conclusions in this report appear to be justified on the basis of the available data. Some observations are offered in connection with this project. First, at the time of the condition survey (August 30, 1966), there was a greater amount of alligator cracking in the southerly four miles of its seven mile length. The northern section exhibited a moderate amount of transverse (shrinkage) cracking but was not greatly different from the aggregate base section contiguous to this project on the north end. It appears that the northern three miles is in reasonably good condition after eleven years of service. The southerly four miles is in very poor condition, particularly the southbound lane. Examination of Table 2 of Research Project 24 would seem to indicate that there may be an extra 1.0 feet of subbase in the northern portions as evidenced by actual thicknesses recorded for Station 53+30. This is not intended as a firm finding but rather the type of information which might explain the systematic variation in performance. The higher cement content recommended by the Portland Cement Association seems reasonable in these sand dominant materials if adequate slab strength is to be achieved and an equivalency obtained with aggregate base in keeping with the experience of other design agencies.

Shrinkage - Shrinkage cracks were considered to be meandering cracks which form polygons or blocks with widths of two feet or more. These can occur in the plant mix itself or may be reflected through from volume changes which occur in the underlying, untreated layers.

Five projects (14 percent) were described as exhibiting some shrinkage cracking. Shrinkage cracking was generally believed to be a reflection of shrinkage in the untreated layer or due to the porosity of the aggregates used in the plant mix. This does not appear to be a significant problem on those pavements inspected as a part of this pavement condition survey.

Ladder cracking - Ladder cracking is normally associated with cement treated bases and was not observed on any projects inspected. The one cement treated base project examined was in District 3, and no ladder cracking was noted on this project. Another cement treated project was informally examined in District 1 in the vicinity of McCamman, and no ladder cracking was observed. This project appeared to be in good condition at the time it was inspected.

In final summary of the pavement condition survey, it has served to isolate the major form of distress on the selected projects as being the load associated longitudinal crack. Further, in considering the cause of the type of distress, the observations from the condition survey indicate that stripping in the plant mix and possibly the structural (ballast) section are prime factors for consideration in analyzing the data available from a portion of those projects examined.

V. ANALYTICAL CORRELATIONS OF PROJECT DATA TO OBSERVED PERFORMANCE

This section of the report attempts to relate certain objective measurements relative to material properties, construction, or design which may be significant to the various types of distress noted. The analysis has been concentrated on the following four items; (1) riding quality, (2) structural design, (3) stripping in the plant mix, and (4) absorptive aggregates for plant mix.

A. Riding Quality

Several attempts were made to relate riding quality to some parameter of traffic, age, ballast section, or subgrade classification. The basic source of information for this analysis was obtained from Tables 1 and 2 as provided by Department records.

The comparison of age to riding quality is illustrated on Figure 9 and is generally indicative of efforts to relate riding quality to some pertinent parameter. In Figure 9 there is a general lack of any trend with age, at least for those pavements examined.

In an effort to combine age with traffic, a coefficient was developed to combine both age and the accumulation of commercial traffic. This Index of Traffic is obtained by multiplying the age of the pavement, the percent of commercial traffic, and the most recent average daily traffic. It can be seen in Figure 10 that this combination did not

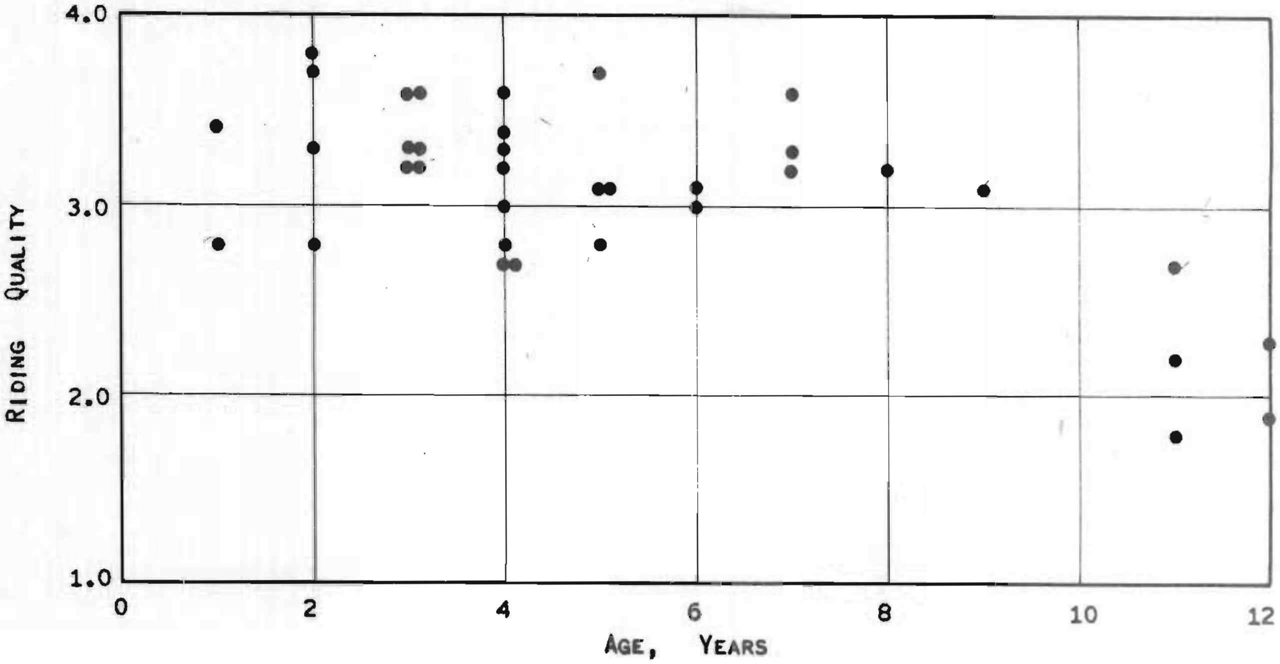


FIG. 9 - EFFECT OF AGE ON RIDING QUALITY

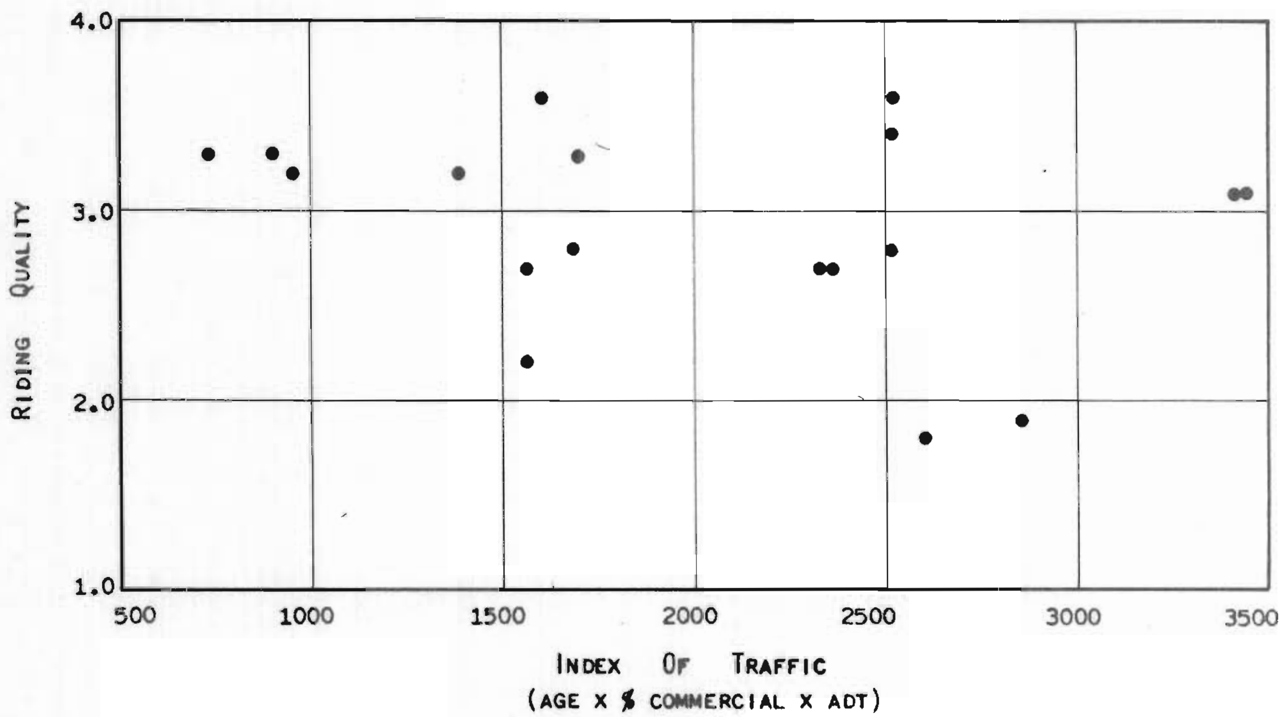


FIG. 10 - INFLUENCE OF CUMULATIVE COMMERCIAL TRAFFIC ON RIDING QUALITY

improve the correlation with riding quality.

The quotient of the Index of Traffic to the total ballast section was used in an attempt to introduce thickness into the correlation with riding quality. This relationship is shown in Figure 11. There appears to be a very slight indication from this figure that thickness is influencing long term riding quality.

The final attempt to relate riding quality to some performance factor is shown in Figure 12. In this case a longitudinal cracking index was used to summarize the extent and progression of the longitudinal load crack. This index was obtained by multiplying the degree of seriousness of the crack, i.e., 1, 2, or 3, by a numerical equivalence for the areal extent of the cracking, i.e., A = 1, B = 2 and C = 3. Thus, the index can range from 0 to 9. There appears to be some slight association between the longitudinal cracking index and the riding quality, although it is admittedly weak.

In summary, this analysis tends to show that the riding quality, at least for the first 8 or 9 years of service life, is highly dependent on the quality of the original construction. This conclusion is based on the lack of any strong correlation between age, traffic or thickness and riding quality. There is a tenuous indication that the presence of longitudinal cracking would reduce

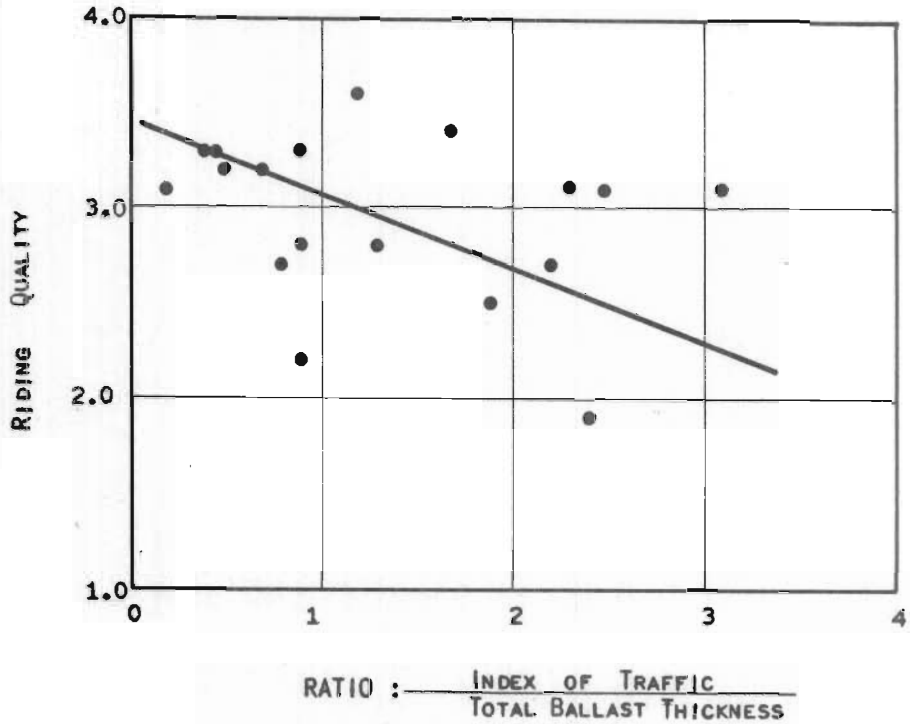


FIG. 11 - INFLUENCE OF TRAFFIC AND BALLAST SECTION ON RIDING QUALITY

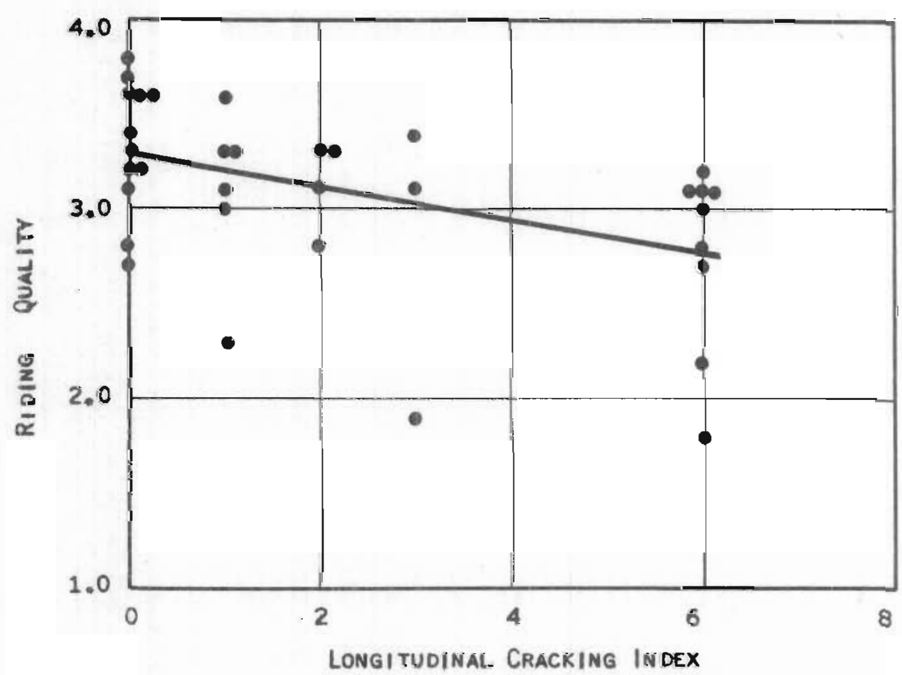


FIG. 12 - INFLUENCE OF LONGITUDINAL CRACKING ON RIDING QUALITY

the riding quality. Although this would seem to be an obvious conclusion, it is pertinent to point out that the cracking, per se, plays a small role in estimating riding quality according to the present serviceability index (PSI) equations developed from the AASHO Road Test.

B. Structural Design

In considering the adequacy of the ballast section, two dependent variables were considered pertinent. First, the riding quality and second, the presence of load-associated cracks in the plant mix.

Apparently, at least on the basis of this survey, the riding quality is not strongly related to the structural (ballast) section. This is illustrated somewhat by Figure 11. There would appear to be some slight indication from this figure that for the same type and amount of traffic, the heavier ballast sections are providing improved performance. The relationship is weak and would require a more detailed analysis in order to draw a positive conclusion. One of the problems in the present analysis, at least for some projects, is the fact that several ballast sections exist on a single project. In most cases, it was not possible in the field to confidently associate the changes in the ballast section to changes in performance. Hence, those sections which have major changes in the ballast section have not been included in plotting Figure 11. The same situation reoccurs for the analysis shown in Figure 13, in which the ballast section is related to longitudinal load cracking.

The lack of a relationship between the total thickness of the structural layers and longitudinal load cracking is shown on Figure 13. This analysis includes only the A-4 subgrade materials. The data tends to show that the ballast thicknesses are not associated with the presence or absence of load-associated cracks.

Additional correlations involving longitudinal cracking and thickness were also attempted in which age and traffic were considered, however, with no more success than shown in Figures 12 and 13.

Thus, it is concluded that, based on the selected pavements included in this survey, very little correlation exists between the ballast section (structural thickness) and riding quality or longitudinal cracking. It is pertinent that if such a relationship did exist, in order to show up in this survey, it would be necessary for the ballast thickness to be highly associated with differences in performance. This is due to the rather "broad-brush" type of study made herein as compared to a "case study" type of analysis. In view of the fact that the Department is using a well established thickness design procedure, it is not surprising that there is no indication of an obvious deficiency in pavement thickness. Closer examination could disclose

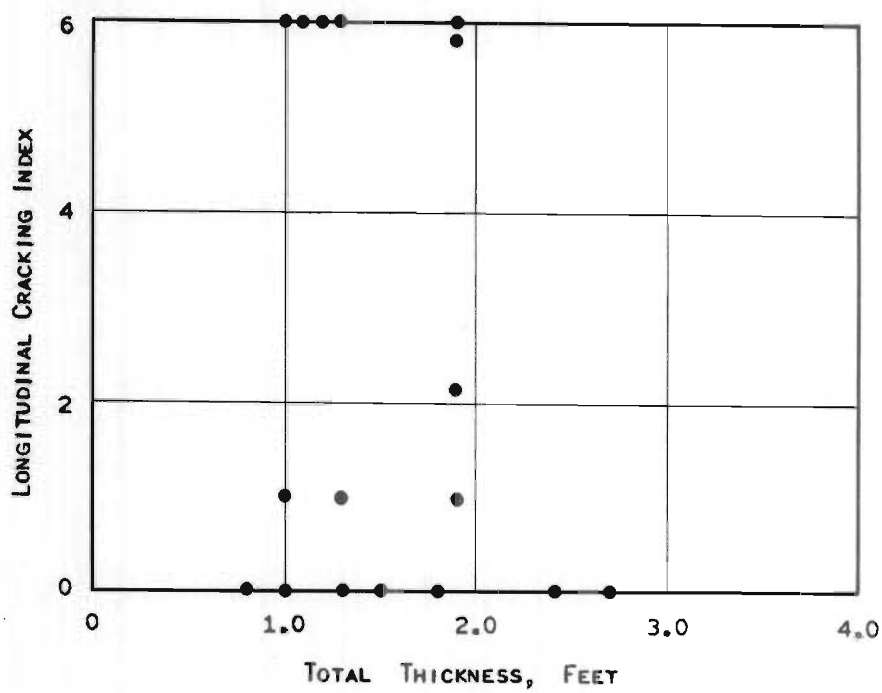


FIG. 13 - EFFECT OF BALLAST ON LONGITUDINAL CRACKING
FOR SUBGRADE OF A-4 CLASSIFICATION

that there is a minor yet significant effect. It would be concluded that this is not the major problem in terms of the performance of those pavements included in this study.

C. Asphalt-Aggregate Stripping

Stripping of the lower course of the plant mix was discussed in Section IV. It was concluded that the loss of cohesion could effectively reduce the potential traffic-carrying capability of a pavement. It seems reasonable to hypothesize that stripping is a major source of the difficulty related to the longitudinal load associated crack. The principal mechanism for distress has been considered to be fatigue in this evaluation; however, fracture under a single load during critical periods (cold) should not be ruled out.

On the basis of information in Section IV, it was assumed that stripping and cracking were always present together. If stripping is a clue to performance problems, it was thought that an examination of project records relative to the following items might be pertinent.

1. Asphalt source
2. Asphalt grade
3. Asphalt percent
4. Asphalt properties
5. Aggregate gradation
6. Field void content

Unfortunately, there was a rather poor match between the projects for which suitable data was available and the projects available for examination. Also, it was impossible, on the basis of the general examination made in the field, to pinpoint performance to material properties. Information relative to asphalt source, asphalt grade and percent of asphalt used is summarized for 9 projects in Table 3, for which field information is available. In this case cracking was categorized as present (yes) or absent (no). Unfortunately, only one of the 9 projects was reported free of cracking.

Some conclusions can be made from the information in Table 3.

1. Three asphalt sources are included in the tabulation, and longitudinal cracking was noted on projects with asphalt from each source.

2. Both 85/100 and 120/150 penetration asphalts were associated with cracking. The one section for which no cracking was reported used 85/100 penetration asphalt in the lower course.

3. The design asphalt content and average asphalt content from construction records show good correspondence. The range of asphalt contents reported in the construction records is high, from

TABLE 3 - SUMMARY OF PERTINENT INFORMATION RELATIVE TO PLANT MIX SURFACE

PROJECT	Lower Course: Aggregate Specification	Lower Course: Asphalt Penetration	Lower Course: Asphalt Content Design: Lab	Lower Course: Asphalt Content Design: Field	Lower Course: Asphalt Content Field, Avg.	Lower Course: Asphalt Content Field, Range, Avg.	Lower Course: Supplier	Lower Course: Asphalt Penetration / Ductility	Top Course: Aggregate Specification	Top Course: Asphalt Penetration	Top Course: Asphalt Content Design: Lab	Top Course: Asphalt Content Design: Field	Top Course: Asphalt Content Field, Avg.	Top Course: Asphalt Content Field, Range, Avg.	Top Course: Supplier	Top Course: Asphalt Penetration / Ductility	PAVEMENT CRACKING
District 1																	
3. I-15-1(5)17 Deep Creek-Colton Lane	B	120/150	5.8	5.8/6.2	5.9	1.4	American	64*/136	D	120/150		6.2	5.8	1.4	American	72*/138	Yes
5. I-15-1(18)70 Pocatello Cr. G.S. - Jct. 15W									E	85/100	5.3	6.5	5.7	0.8	Phillips	66*/140	-
7. I-15-2(6)71 Sec A Chubbuck G.S. - So. Blackfoot I.C.	C	85/100		5.4	5.4	0.8	American	40*	D	85/100		5.5/5.6	5.7	2.1	American	40*/126	Yes
8. I-15-2(6)71 Sec B Chubbuck G.S. - So. Blackfoot I.C.	C	85/100		5.4	5.4	1.2	Phillips	43*/42	D	85/100		5.5/6.0	5.6	1.4	Phillips	43*/42	Yes
9. I-15-2(9)88 Sec A So. Blackfoot I.C. West Blackfoot I.C.	B	120/150		5.6	5.4	2.0	Phillips	66*/140+	D	120/150		5.7	5.8	1.6	Phillips	66*/140+	Yes From Dept. Reports
11. I-15-2(11)96 Sec A Porter Bridge G.S. - Gr. Western Canal	B	120/150	5.0	5.2	5.1	2.1	American	55*/117	D	120/150			5.6	1.1	American	55*/117	Yes
12. I-15-2(11)96 Sec B Gr. Western Canal - Bonneville Co. Line	B	120/150	5.2	5.2	5.3	1.7	American	58/140+	D	120/150		5.2/5.3	5.5	0.8	American	39/83	Yes
District 2																	
E. I-15W-5(6)118 Snake River-Raft River Eastbound Lanes	B	85/100	5.2/5.6	4.8/5.4	4.9	0.5	American			120/150	5.0/5.2	5.8/6.3	5.5	0.9	American		No From Dept. Reports
District 3																	
E. I-80N-1(18)3 Jct. US 95 - Jct. US 30	C	85/100	5.0	4.6/5.0	4.7	2.0	Sinclair	21*/11	E	85/100	5.0	5.0/5.3	5.0	2.0	Sinclair	21*/11	Yes

*Composite Sample

0.5 to 2.1, with 16 out of 17 cases exceeding an 0.5 percent range. Admittedly, the range tends to exaggerate the variation which occurs for a particular project. However, it is reasonable to conclude that a large range is probably associated with large variability in asphalt content and such variability could lead to a substantial proportion of the mix being placed with insufficient asphalt. It would suggest that the void content also has a high variability for the in-situ plant mix.

The two properties most commonly associated with asphalt quality are the recovered penetration and ductility. Based on information in References (29) and (30) the critical value of penetration is approximately 20 and ductility 15. Halstead (31) suggests a critical ductility-penetration relationship rather than lower limits. With the exception of I-15-2(6)71, Section B project, all of the asphalts reported in Table 3 would appear to be above the critical relationship. The values reported in Table 3 indicate that asphalt properties well above the suggested critical limits are associated with cracking. This suggests that asphalt properties per se may not be responsible for stripping and cracking.

Examination of information contained in Table 3 indicates that different aggregate gradations were used in the lower and upper courses. Field observations tend to indicate, with only one or two exceptions, that stripping was confined to the lower course. A rather detailed examination of the aggregate gradations was made.

Three techniques were used in evaluating the aggregate gradation: (1) the standard semi-log plot, (2) the log-log plot used by the Bureau of Public Roads (32), and (3) log-log plots described in Reference (33).

Comparison of Idaho's Class A and B aggregate gradations, on a semi-log type plot, with similar gradations of the California Division of Highways indicates they would each be categorized as "coarse" graded. A similar type comparison to gradations of the Washington Highway Department indicates the Class A gradations of each agency are practically identical.

A plot of the Class B grading (average) on the log-log plots of the Bureau of Public Roads is shown in Figure 14. This type of plot has been developed to provide information as to the amount of voids to be associated with a given grading (voids mineral aggregate or VMA). The theoretical maximum grading (minimum voids) would be achieved if the slope of the grading curve fell on the 0.45 line. With the possible exception of the Class G grading, all of Idaho's gradation curves will fall below the 0.45 line. It should be mentioned that the actual gradings never fall on

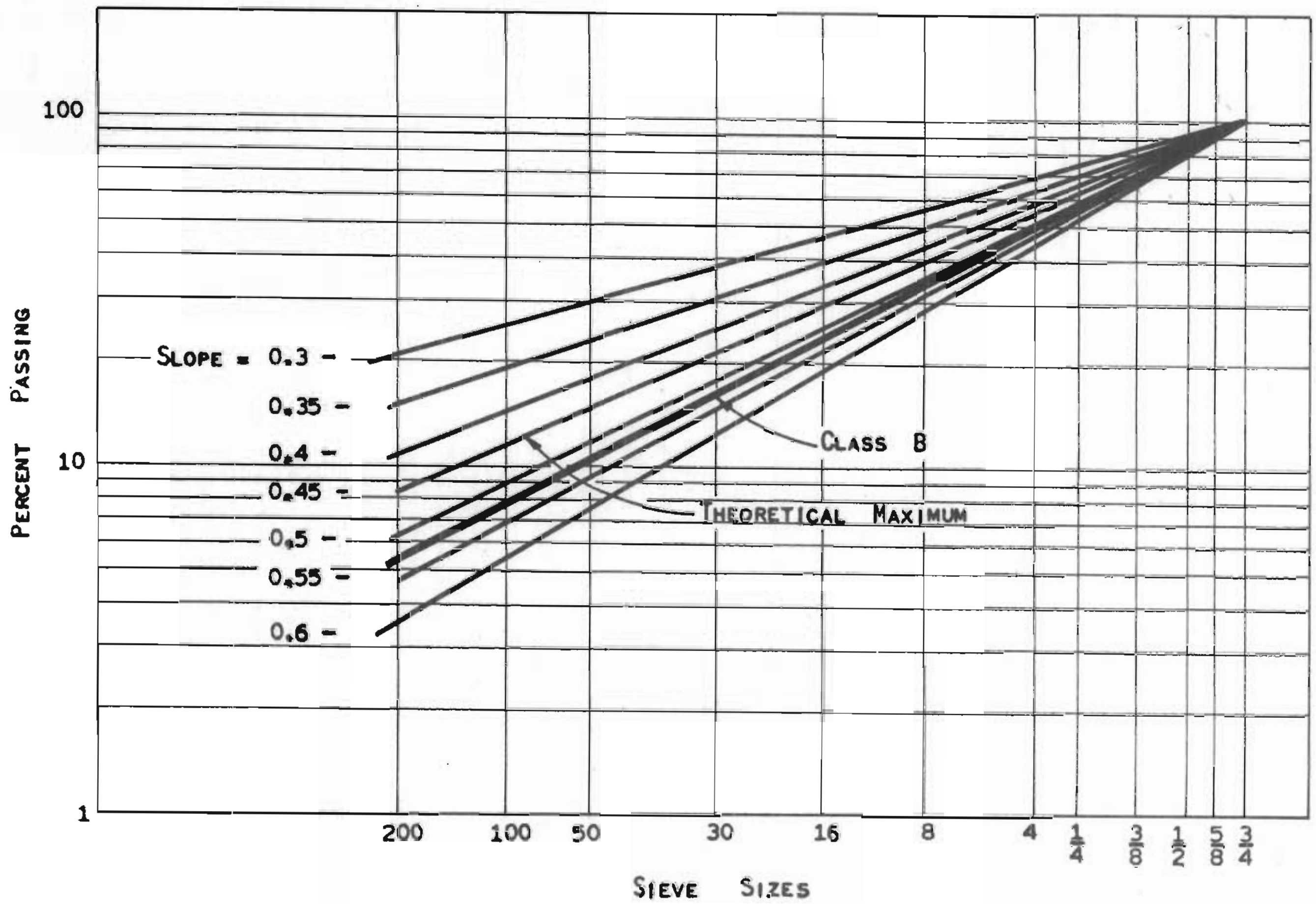


FIG. 14 - CLASS B AGGREGATE GRADATION ON BPR LOG - LOG CHART

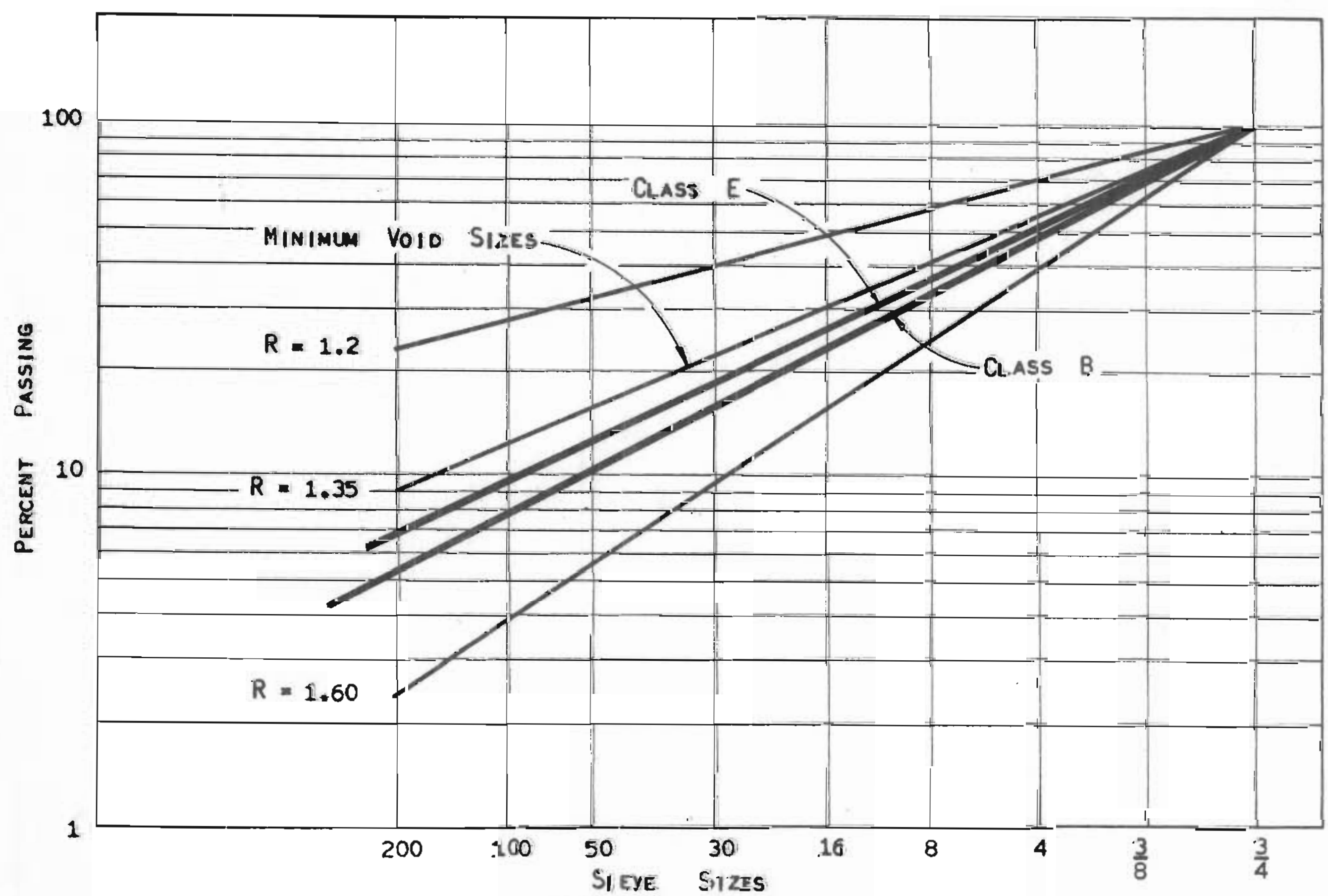


FIG. 15 - EVALUATION OF VOID SIZES IN AGGREGATE

a straight line and hence, the slope is actually changing through the spectrum of sieve sizes used. The California 3/4-inch fine grading plots are a curve, concave down, at a general slope, slightly less than 0.45. The conclusion to be reached from this plot is that, in general, Idaho's gradings would result in a porous mix. The recently adapted Class G grading would tend to be an exception to this in that it tends to lie along a 0.45 slope. Probably the one factor that would contribute most to this porosity is the low percentage of filler (passing No. 200 sieve) found in Idaho's gradation requirements. Also, examination of actual gradings on some 40 construction projects indicates that approximately 60 percent of all the gradings reported had less than 5 percent of material passing the No. 200 sieve.

Figure 15 attempts to evaluate the size of the voids in the mineral aggregate. According to Reference (33), it is common practice to grade the lower courses so as to produce relatively large voids ($R > 1.35$) and wearing courses to produce small voids ($R < 1.35$). As can be seen on Figure 15, even Idaho's Class E mix would appear to have an R-value greater than 1.35. California's 3/4-inch fine gradation would also plot slightly higher than 1.35. In general a value of 1.35 is not considered particularly desirable since the resultant void size may not leave room for the asphalt and filler. The net conclusion of this examination would tend to indicate that the mixes used by Idaho tend to be somewhat of a porous character.

It is emphasized, however, that the current gradation requirements of the Department are typical of gradations used in the Western U.S. Except for the susceptibility for stripping, the aggregate grading problem could be ignored.

Void content of the plant mix (in-place) could also be related to the susceptibility to stripping with the highest void contents being associated with highest susceptibility to stripping.

Examination of laboratory data relative to void content did not prove particularly helpful. The problems in the correlation were the same as previously described for the analysis associated with asphalt content and gradation. In order for this information to be helpful, it would require investigations of the case study type as represented by the Department's Research Project 24.

Certain isolated information is of some interest. For example, of all the data examined, the void content in the lower course (determined by the Rice method), ranged from 2.3 (I-15-2(11)96, Sec B) to 12.6 on I-80N-3(3)206 with an average value of 6.9 percent voids for 10 projects in five of the six districts. This would seem to be a reasonable value for voids. However, it is not the average value that controls performance but the critical value as represented by the upper limits of the range. Minor (34) has suggested a zone of highest durability between 6 and 8 percent voids, based on ratio to voidless mass. This information suggest that cohesion increases as voids decrease. Considering the type of distress observed on Idaho's highway system, it would seem essential to tend toward the lower void content of 6 to 8 percent or considering the difference in specific gravity procedures, 5 to 7 percent by the Rice method.

In summary, relative to asphalt-aggregate stripping:

1. Stripping does not appear to be associated with any particular source or sources of asphalt.
2. Stripping does not appear to be associated with any particular grade of asphalt.
3. Stripping does not appear to be associated with any particular percent of asphalt.
4. Stripping does not appear to be associated with recovered penetration or ductility of asphalt.
5. Stripping tends to be concentrated in the lower, coarser graded, zone of the plant mix surfacing. There is no clear evidence that the finer graded Idaho plant mix would not also strip.
6. Stripping could not be associated with void content. This may be due to a lack of "case study" type of information.

Although not specifically considered as a factor in the above discussion, some comments relative to recent research dealing with the durability of asphalt mixes in the presence of water seems pertinent. Skog and Zube (35) have described a number of ways for identifying the effect of water on asphalt mixes. The results of these studies indicate that improved performance for specific cases will be obtained by (a) increasing the asphalt content, (b) using a lower penetration asphalt, (c) increasing the density, (d) using selected anti-stripping agents, and (e) selection of aggregates. Aggregate source was indicated to be the most important single factor in the performance of the asphalt mixes tested and can override all of the other efforts to achieve satisfactory performance.

D. Absorptive Aggregate

Absorptive aggregate in some parts of Idaho is believed to be a contributor to shrinkage type cracking. Research by the California Division of Highways (26) reports a strong association between aggregate absorption, as measured by a modified centrifuge kerosene equivalent test, and cracking.

The projects selected for field survey included some cases in which absorptive aggregates were used in the plant mix. It was difficult, in the field, to identify the effect on performance of these aggregates. One project, F-2441(15) five miles east of Junction U.S. 93 to Junction SH50 - did exhibit some cracking considered to be associated with the absorptive aggregate. This project had a high percentage of shrinkage cracks and closer spacing of transverse cracks. The amount of cracking reduced significantly from the north to the south end of the project. Discussion with the Resident Engineer indicates that the asphalt content was increased in the same way, indicating that the increased amount of asphalt was offsetting the effect of the absorptive aggregate. Results of tests reported in reference (26) indicates that increases in asphalt content will have a major effect on reducing the cracking potential of asphalt mixes manufactured with absorptive aggregate.

In general, highway engineers have tried to avoid using absorptive aggregates. Some of the more common reasons for this are:

1. The optimum asphalt requirement is difficult to establish. No precise way has yet been developed for accounting for the amount of asphalt to be absorbed. The most common methods are based on water absorption as measured by the difference between apparent and bulk specific gravities or impregnated tests made on aggregate which has been coated with asphalt. The standard centrifuge kerosene equivalent test can also be used as a

measure of aggregate absorption in combination with surface area. The principal shortcoming of these tests is that they do not reflect the long-term absorption characteristics.

2. Attempts to compensate for absorption can lead to an excess of asphalt during construction.

3. If asphalt absorption occurs, it could lead to the selective absorption of the oils from the asphalt and result in a hard, brittle mix.

4. If asphalt absorption occurs, it will reduce the thickness of the asphalt films and result in a reduction in the tensile strength properties of the mix.

5. If asphalt absorption occurs, it will tend to reduce the net volume of the mix, resulting in shrinkage stresses and possibly shrinkage cracks. Further reference to this factor will be made in Section VII.

E. Discussion of Analysis

It is unfortunate that the analytical analysis has not provided some better evidence as to the potential causes for distress. The lack of such correlations suggests that a limited number of detailed studies may be necessary to obtain documentation as to the specific causes for distress. Surveys and correlations such as are represented herein look more for strong correlations or suggest areas for more detailed studies. Both of these results have been obtained. For example, stripping of the asphalt-aggregate system is concluded to be a matter of major concern which will require the immediate implementation of corrective procedures based on material and construction technology. Areas requiring more sophisticated studies have to do with: (a) a better understanding of the mechanism of stripping, (b) a better understanding of the mechanism of the longitudinal load crack, (c) a re-evaluation of the ballast section especially for subgrades with relatively high (greater than 50) resistance (R) values, and (d) procedures for designing mixes which are manufactured with absorptive aggregate.

VI. TESTING AND EVALUATION OF ASPHALT

Phase II of this investigation called for specific tests on asphalts representative of those currently being supplied to the Department for use on highway construction. These tests were specifically oriented to the determination of asphalt durability in accordance with the procedures developed by F. S. Rostler and R. M. White. The results of tests made on ten asphalts supplied by the Department are shown in Table 4.

The interpretation of the data shown in Table 4 must be made with a degree of caution since each series of tests was made on a single sample of asphalt. Test data of this type is known to have some variability as a function of the repeatability characteristics of the test and the variability associated with product manufacturing. Nevertheless, these data do tend to indicate some information relative to the durability of asphalt. Temperature susceptibility properties were evaluated to determine if radical variations or abnormal values were obtained for the asphalts tested.

Interpretation of the test data is based mainly on information in footnote references (4) and (5) on Table 4, together with unpublished papers by Welborn, Oglio and Zenewitz and by Halstead, Rostler and White. The latter papers were presented at the 1966 annual meeting of the Association of Asphalt Paving Technologists.

TABLE 4 - TEST RESULTS ON ASPHALT SAMPLES FROM STATE OF IDAHO

Asphalt Number	1-66	2-66	3-66	4-66	5-66	6-66	7-66	8-66	9-66	10-66
Penetration Grade	120/150	120/150	85/100	85/100	85/100	60/70	60/70	85/100	120/150	85/100
Penetration @ 77°F	133	131	85	87	83	66	70	94	128	84
Viscosity @ 77°F, megapoises										
@ 0.05 sec^{-1} (1)	0.565	0.432	1.07	1.05	1.21	1.76	1.75	0.930	0.430	1.69
@ 0.001 sec^{-1}	0.900	0.458	1.18	1.07	1.45	1.76	2.96	0.995	0.550	1.69
Shear Susceptibility	tan 6.8°=0.12	tan 0.8° 0.01	tan 1.5°=0.03	tan 0.3°=0.01	tan 2.6°=0.05	0	tan 7.7°=0.14	tan 1.0°=0.02	tan 3.6° 0.06	0
Calculated Penetration (2)	115	130	85	86	81	68	68	91	130	69
Viscosity @ 39.2°F, megapoises										
@ 0.05 sec^{-1}	89.5	127	181	138	224	362	174	87.5	107	129
@ 0.001 sec^{-1}	365	298	521	508	760	1130	1670	400	472	690
Shear Susceptibility	tan 19.9°=0.36	tan 12.4°=0.22	tan 15.3° 0.27	tan 18.8°=0.34	tan 17.4°=0.31	tan 16.3°=0.29	tan 30.0°=0.58	tan 21.3°=0.39	tan 20.9°=0.38	tan 23.3°=0.43
Temperature Susceptibility (0.05 sec^{-1}) (3)	-3.42	-3.83	-3.35	-3.21	-3.39	-3.39	-2.97	-3.03	-3.74	-2.82
Chemical Composition, % (4)										
A (Asphaltenes)	21.6	23.2	22.4	21.5	21.9	21.1	16.8	20.0	19.4	21.9
N (Nitrogen Bases)	16.7	19.6	24.3	21.0	18.6	21.7	15.2	29.4	20.0	17.3
A_1 (First Acidaffins)	18.9	18.6	22.8	19.2	19.7	21.7	26.1	14.4	21.2	19.6
A_2 (Second Acidaffins)	20.1	24.5	22.4	26.5	26.6	23.4	25.0	19.8	25.7	26.7
P (Paraffins)	22.7	11.1	8.1	11.8	13.2	12.1	16.9	16.4	13.7	14.5
Wax	+ Room Temperature	+ Room Temperature	+ Room Temperature	+ Room Temperature	+ Room Temperature	+ Room Temperature	+ Room Temperature	+ ice	+ Room Temperature	+ Room Temperature
$\frac{N + A_1}{P + A_2}$	0.83	0.99	1.54	1.05	0.96	1.22	0.99	1.21	1.05	0.90
Durability Group (5)	I	I	IV	II	I	III	I	III	II	I
Pellet Abrasion @ 77°F										
% Loss, Mix	0.05	6.4	2.0	10.7	3.3	24.4	3.8	3.5	1.5	1.8
Aged 7 Days @ 140°F	0.19	29.6	35.0	52.4	30.5	86.5	3.6	10.3	1.8	4.1
Average, M & 7 Days	0.12	18.0	18.5	31.5	16.9	55.5	3.7	6.9	1.7	2.9

(1) Shear Rate

(2) Pfeiffer, J. Ph., Properties of Asphaltic Bitumen, Elsevier Publishing Company, 1950.

(3) Walther equation:

$$\text{viscosity-temperature susceptibility} = \frac{\log\log \eta_2 - \log\log \eta_1}{\log T_2 - \log T_1}$$

(4) Rostler, F. S. and R. M. White, "Composition and Changes in Composition of Highway Asphalts, 85-100 Penetration Grade," Proceedings, Association of Asphalt Paving Technologists, 1962.

(5) Rostler, F. S., "Prediction of Performance Based on Asphalt Composition Using Precipitation Methods," Proceedings, Highway Conference on Research and Development, Bureau of Public Roads, 1965.

The penetration tests require no particular comment. Asphalts 5-66 and 10-66 were outside the penetration range set for the assigned grade; however, repeat sampling would probably show the asphalt to be on grade.

Viscosity tests at 77°F lead to the following conclusions:

1. The asphalts are essentially Newtonian. This is indicated by the low shear susceptibility and by the ability of the Pfeiffer formula to accurately predict penetration from viscosity. Asphalts 1-66 and 10-66 vary somewhat from the calculated penetration values. In the case of 1-66, this could be accounted for, in part, by the high shear angle. No explanation is suggested for 10-66.

2. For a Newtonian type asphalt, the viscosity does not appear to be low. This is confirmed by a comparison of the data in Table 4 with penetration-viscosity relationships presented in the paper by Welborn, Oglio and Zenewitz. Non-Newtonian asphalts would tend to have higher viscosity values.

3. The shear susceptibility values at this temperature do not appear to be abnormally high. Asphalt 1-66 may be on the high side, and its performance should be evaluated with particular regard to the relative amount of non-load associated cracking.

Viscosity tests at 39.2F are not easy to evaluate since research findings with this type of information is not well defined. In a general way, the information indicates the following:

1. The asphalts tend to be non-Newtonian at this temperature as indicated by the shear susceptibility values. No calculated penetration value is given at 39.2F since this loading time exceeds 5 seconds.

2. The viscosity at 39.2F is comparable or slightly higher than reported by Welborn, et al. The data of Halstead, et al., indicates the viscosity of the Idaho asphalts to be within the range of values obtained in testing asphalts from all parts of the United States.

3. The shear susceptibility values are comparable to data presented by Welborn, et al.

4. The temperature susceptibility values do not appear to be unusual. Welborn, et al., indicate that other investigators have found this value to range between -3.1 and -3.9 between 140F and 275F. Data presented in their paper would tend to indicate this slope could be slightly higher between 77F and 39.2F.

There is nothing particularly unusual about the chemical composition of the ten asphalts tested. The presence of wax, in the amounts qualitatively indicated, is considered normal.

The durability groups, determined by the Rostler parameter as the ratio of $\frac{N + A_1}{P + A_2}$, rank the asphalts into five groups. Group I asphalts are described as having Superior durability, Group II are Good, Group III are Satisfactory, Group IV are Fair, and Group V are Inferior.

On the basis of the sample tested, only asphalt 3-66 would be considered in the Fair category, while the remainder of the asphalts are ranked as Satisfactory or higher. It should be noted that the division between Groups III and IV is made at a ratio of 1.5. Thus, asphalt 3-66 tends to be of the better durability asphalts in this group.

The pellet abrasion test is a performance test designed to obtain the same type of information as is obtained from Shot Abrasion Test of the California Division of Highways. The results of this test indicate that asphalts 4-66 and 6-66 could have less than desirable durability. Part of the high value associated with asphalt 6-66 could be the lower penetration value of the original asphalt. The results reported by Halstead, et al, indicates that some increase in abrasion loss will be obtained with the lower penetration asphalts, however, not to the extent indicated for asphalt 6-66.

In summary, the test data would seem to indicate that the majority of the asphalts tested would be comparable to asphalts of the same grade being supplied in most parts of the United States. The durability of the majority of the asphalts tested would be considered average or better when compared with other areas of the country. On the basis of the samples submitted, and in the absence of replicate samples, asphalts 3-66, 4-66, and 6-66 would bear some further study. Hardening after aging in the thin film oven would be one area to focus some attention.

It is pertinent to point out that both the Rostler parameter and the pellet abrasion test were developed to evaluate the effect of asphalt embrittlement due to aging. The test method and the field correlations have not, as yet, included the mechanism of stripping. It is conceivable that ranking on the basis of stripping would be quite different than that indicated on Table 4. For example, air-blown asphalts, which would rank better by the Rostler durability rating, would probably have poorer adhesive characteristics. This may be the answer to the stripping problem since experience indicates blown asphalts are poorer performers, with respect to adhesion to aggregates, than straight-run or cracked asphalts.

Annual and semi-annual condition surveys should be made of each project using the asphalts described in Table 4. In this way, some better definition can be obtained as to the value and significance of this particular type of test data for the Idaho environment.

VII. RECOMMENDATIONS

The recommendations contained herein are based on four sources of information: (1) field examination of in-service pavements, (2) examination of laboratory data and correlation with field performance, (3) engineering technology as presented in technical publications, and (4) engineering experience of the Review Panel members, Department engineers and the MR&D staff and consultants.

Unfortunately, the laboratory information was not as useful as had been hoped. Experience indicates this is very often the case. That is, unless test data is collected according to a well thought out plan, aimed at specific objectives, pertinent information is often lacking in critical areas. In this case, the situation was complicated by the fact that many of the projects, for which adequate data was available, had been overlaid prior to the time the field condition survey was made.

Recommendations are given here somewhat in order of confidence and priority, in other words, for those factors for which information is reasonably strong and which should be considered of primary importance to the Department.

A. Asphalt-Aggregate Stripping

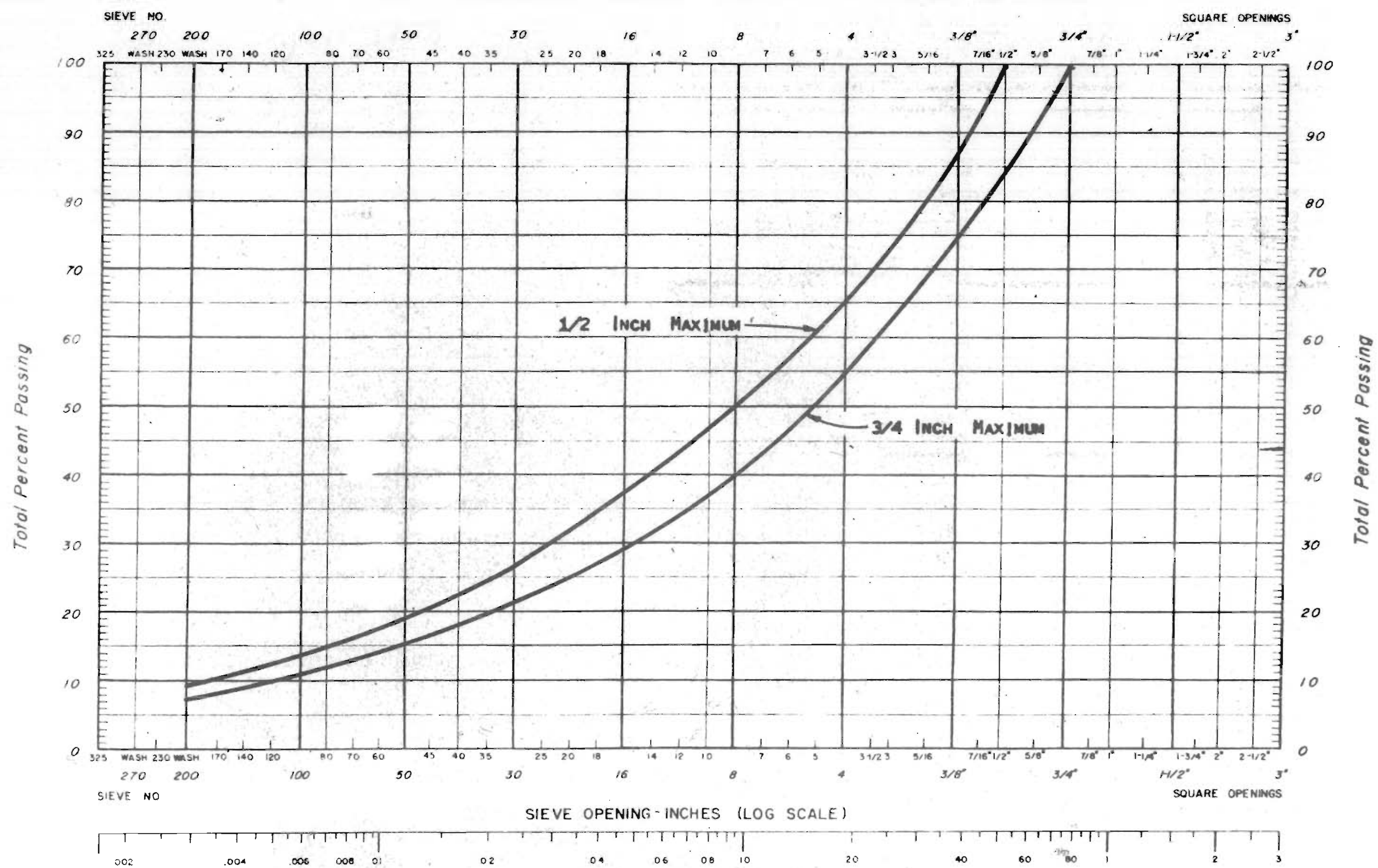
Evidence relative to the presence of stripping is very strong. The cause of stripping is obscure. The argument has been posed that since stripping had not been observed in previous condition surveys, its occurrence now must be associated with some change in the properties of the asphalt. While it is possible that modifications in refinery processes could contribute to the tendency for stripping, there does not appear to be any documenting evidence. It seems more likely that stripping is associated with aggregate source and gradation and possibly with voids in the in-place plant mix and that the mechanism is accelerated by the development of cracks in the surface layer. There is a possibility that the soluble minerals in the water are associated with stripping. If this is the case, the water is being pumped from below through the subbase and base into the plant mix.

The following recommendations are made in an effort to minimize stripping until research can find positive answers to the overall problem.

1. The aggregate gradation for plant mix should be modified to produce a denser (less porous) mix. A recommended gradation is given in Figure 16 for both the 3/4-inch and 1/2-inch maximum size. A job mix formula may be added to provide operating limits; however, it should not allow less than 5 percent of filler (passing 200 sieve).

2. Commercial hydrated lime should be added to all plant mixes. The amount of lime to be added should be limited until such time as better evidence is

FIG. 16 - RECOMMENDED AGGREGATE GRADATION FOR ASPHALT CONCRETE



U. S. STANDARD SIEVES - ASTM DESIGNATION E 11-39 available as to the influence of additional amounts of lime on flexibility and asphalt requirements. At least one percent of lime should be used. Amounts in excess of two percent should be used only after a thorough evaluation of the properties of the mix. Fatigue properties in particular should be evaluated.

Commercial lime can be obtained in various gradations. It is recommended that at least 75 percent of the lime should pass the No. 200 sieve.

Lime may be added either at the pugmill or to wet aggregate at the cold stockpile in accordance with procedures of the State of Colorado as outlined in their letter of September 16, 1966, to L. F. Erickson. Lime has long been used by highway engineers as a means for improving stability and to reduce loss of strength in the presence of water. Reference (36) describes some of the background for this type of treatment based on work of the Colorado Highway Department.

Mix designs and asphalt requirements should reflect the presence of lime in the mix. The added surface area of the lime may require additional asphalt as compared to plant mixes with similar aggregate gradation using natural fillers.

3. The moisture-vapor-susceptibility test is recommended as an aid in identifying the susceptibility to stripping of any given aggregate and asphalt combination. Tests should be made with asphalts supplied from sources to be used by the contractor. The moisture-vapor-susceptibility test procedure can be found in the California Division of Highways Materials Manual as Test Methods 304 and 307.

   Some preliminary experimentation is recommended to determine if additional time in the oven improves the correlation with field performance or if cycling is required. It is suggested that aggregates which have been shown to be associated with stripping and aggregates which have not shown stripping be used to establish criteria for identification. Until such time as specific criteria are established, the stabilometer (S value) after curing according to M-V-S procedures should not be less than 30 for Heavy and Average (see Figure 16-231.21, Surveys and Plans Manual) Traffic classifications and 20 for Light Traffic.

   In lieu of the M-V-S, the Department may, on the basis of past experience, want to explore further the use of the Immersion-Compression Test (AASHO

T-165, ASTM D1075). Reference (31) and the previously mentioned letter of September 16, 1966, to L. F. Erickson suggest working criteria for this test.

4. Known sources of aggregate for plant mix should be tested and identified as to susceptibility to stripping. Procedures (grading, additives, asphalt grade, etc.) for minimizing stripping should be identified with these sources.

5. Extensive field studies should be implemented to identify the full extent and character of the stripping in the plant mix. Where possible, the following items should be determined.

   Identify presence of stripping with regard to:

   a. presence of crack and transverse location of crack

   b. presence of stripping as a function of transverse width

   c. presence of stripping as a function of thickness, particularly associated with aggregate gradation

   d. asphalt source and grade

   e. aggregate source

   f. pavement condition

After the extent of stripping has been determined, a series of case studies are recommended which will encompass the primary sources of aggregate (e.g., Snake River, etc.), range of climates (precipitation and frost), and classes of traffic (Heavy, Average, and Light). It will also be very important to include mixes which have exhibited stripping and which have not exhibited stripping. This, of course, cannot be determined until the initial survey is made.

The following laboratory tests would be desirable for each selected project.

a. stripping tests

   (1) moisture-vapor-susceptibility

   (2) immersion-compression

   (3) others (see Reference 35)

b. asphalt content and properties (penetration and ductility at 77°F and 39.2°F)

c. aggregate gradation

d. in-situ void content

Other tests may be added to these providing some justification can be found as to their relevancy. It is suggested that the number of tests be limited to those which would be expected to provide useful information.

In obtaining information from each project selected for extensive study, considerable effort should be given to developing a working plan. It will also be necessary to obtain sufficient testing to assure confidence in interpretation of data and to provide a statistical base for analysis. On the basis of factors used to test for significance, it is recommended that from six to ten samples be used to identify material properties at each sampling point. Thus, a minimum of twelve to twenty samples would be required to test one stripped and one unstripped mix or one cracked and one uncracked area. A strip map should be developed for each area to follow the progress of cracking or to identify reflection cracking in the event an overlay is made.

B. Structural Design

There is no clear evidence from the field examination of any overall structural deficiency. However, because of the granular nature of on-site materials in many parts of Idaho, a considerable number of projects are designed with a minimum ballast section. On those projects examined as part of this investigation, the minimum design (initial stage) of 0.3 feet of plant mix and 0.4 feet of aggregate base was found in most cases. A planned overlay of 0.1 feet of plant mix is scheduled for all of the projects on the interstate system to satisfy total design standards. Comparison of the minimum design (including overlay) with similar requirements of other agencies would indicate this thickness is marginal and probably should be increased. For example, California would recommend a minimum thickness of 0.35 feet of asphalt concrete plus 0.60 inches of untreated aggregate base for a traffic index of 6.5 to 7.0. The Asphalt Institute recommends a minimum of 0.4 feet of asphalt concrete plus the equivalent of 0.4 feet of aggregate base for an average daily traffic of 2000 with average truck loadings. Over 50 percent of the projects surveyed have ADT values in excess of 2000. Hence, some increase in the minimum design requirements is indicated. In this regard, the Department has already initiated action to require a minimum of 0.4 feet of plant-mix and 0.4 feet of treated base for initial construction. Recommended minimum thicknesses for State Highway Projects are given in Table 5. This table is predicated on design curves shown in Figure 16.231.3 of the Plans and Surveys Manual. Current policy requires a minimum of 0.4 plant mix surfacing plus 0.4 plant mix base. From Table 5, this thickness will be adequate up to a traffic index of 9.5. As indicated in the footnote to the table, these minimum thicknesses are predicated on a minimum R-value of 50 for materials underlying the base layer. California (37) has adopted similar tables for surface and base. Table 7-603.1 of Reference (37) shows typical depths for surface and base as a function of base type. These recommendations provide for additional thickness of surfacing in excess of the amount indicated as required by the basic design nomograph. This adjustment, which does not increase the total gravel equivalency of the structural section, is provided as a factor of safety for the thickness above the subbase. No such adjustment or "safety factor" is provided in Table 5. If the material underlying the base is commercially produced and controlled, the need for a safety factor in Idaho should wait for some type of documentation resulting from case studies. If, however, this material is on-site material or select borrow, an increase in the gravel equivalency of the surface and base of 0.16 feet is recommended. The justification for this adjustment is the variability in the R-value which can be expected when using materials which are not subjected to any quality control requirements. For example, laboratory data for I-80N-1(18)3 (Junction U.S. 95 to Junction U.S. 30 in District 3) indicates the subgrade R-value ranges from 82 to 15 with an average of 54. Normal sampling and testing would account for most of this range. The odds are good that the total range may not be encountered by such testing, and hence some protection is required.

TABLE 5 - RECOMMENDED MINIMUM(1) THICKNESS FOR BASE(2)

TYPE OF BASE	T.I. = 6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0
Class 2 AB (78 R-value)	0.40(4)	0.40(4)	0.40	0.50	0.55	0.60	0.70	0.8
Asphalt(3,5) Concrete Base	0.40(4)	0.40(4)	0.40(4)	0.40(4)	0.40(4)	0.40(4)	0.40	0.50
Cement(3) Treated Base	0.40(4)	0.40(4)	0.40(4)	0.40(4)	0.40(4)	0.40	0.45	0.55

(1) Based on minimum thickness of cover of 0.4 ft plant mix.
(2) Based on minimum R-value of 50 in the select borrow or subgrade.
(3) Based on substitution ratios given in 16-231.6, Surveys and Plans Manual.
(4) Minimum allowable section for State Highway Projects.
(5) Recommend using dense graded mixes.

It is important to emphasize that in the determination of the ballast section, it is necessary to thoroughly test those materials which are expected to be or which are (in-place) representative of the subgrade for the pavement section. The design of the pavement should be predicated on the lower 10 or 20 percentile strength values and not the average, since it is not the average condition which controls performance and maintenance requirements.

Further evaluation of the Regional Factor should be undertaken by the Department to determine if adequate allowance has been made for critical climatic conditions unique to some areas of the State of Idaho. The AASHO Road Test results have indicated that the most critical seasonal effects occur when the temperature ranges around freezing, that is, periods of freezing followed by periods of thaw. The thickness design basic curves used by the Department were developed in California and subsequently adapted to Idaho conditions. These curves reflect the WASHO and AASHO Road Test results plus the continued accumulation of experience in California. However, it should be recognized that the results of both Road Tests encompassed only two spring conditions and experience in California is predominantly in environments less critical than Idaho. Regional Factor adjustments as given in the Plans and Survey Manual should be applied to the thicknesses in Table 5 where appropriate.

C. Materials Requirements and Quality Control

Some recommendations for modifications in materials requirements have been discussed under aggregate-asphalt stripping. Additional recommendations are as follows:

1. Identify absorptive aggregates by stipulating in the specifications and Plans and Survey Manual that the K_c and K_f factors (obtained from the Centrifuge Kerosene Equivalent Test) shall not exceed 1.8 and that special designs will be required when the K_c and K_f factors exceed 1.5. The absolute value of the upper limit can be modified based on Department experience; however, this factor is currently being used by the State of California pending modifications currently being developed.

   The Department should initiate some research in this area at an early date in order to be able to provide working criteria for mix designs. The California Division of Highways is conducting research supplemental to that reported in Reference (26) relative to the development of a modified CKE test. The objective of this investigation will be to establish acceptable limits for aggregate absorption. The Department should keep in touch with this study by California. In the meantime empirical design procedures may be the best way of handling designs involving absorptive aggregate. Local performance experience will be the basis for design criteria.

2. It is recommended that Class A and Class B aggregate gradation requirements for plant mixes be discontinued.

3. It is recommended that 60/70 or 85/100 penetration asphalt be used in the lower course or in the asphalt base with 85/100 or 120/150 in the wearing course. Preference should be given to the 120/150 penetration asphalt in climatic regions 3 and 4 (see Figure 16-231.5 in Survey and Plans Manual).

4. It is recommended that some consideration be given to preparing a specification for asphalt concrete. This specification would be used as a special provision on projects requiring especially high type plant-mix. By characterizing the material with a new identity, contractors and engineers would realize that higher standards of design and construction would be required as compared with conventional plant mix. Asphalt concrete should be specified for the upper 0.4 feet of the ballast section. The exact determination as to where the special provision would be used would be decided on an individual basis by Department engineers. In

general, use should be based on traffic and past performance of plant-mixes. The asphalt concrete specifications should embody the following requirements.

a. Dense-graded aggregate as recommended in Figure 16.

b. Deficiencies in filler material (passing No. 200 sieve) shall be satisfied by the use of commercial filler with the following properties.

Sieve Size	Percent Passing
No. 30	100
No. 200	75-100
Maximum Surface Area	650 sq ft/lb

c. Job mix formula shall be such that the combined grading during any day's run will not vary more than stipulated in the 1961 Standard Specifications, Section 407.07, Mixing, except that in no case shall the gradation fall outside of the basic gradation requirements, and in no case shall the amount of material passing the No. 200 sieve fall below 5 percent. The variation in asphalt content shall not exceed 0.3 percent, plus or minus.

d. Specify an aggregate durability requirement.

(1) Loss in Los Angeles Rattler (after 500 revolutions) not more than 50 percent.

(2) K_c and K_f (obtained from Centrifuge Kerosene Equivalent Test) not to exceed 1.7 maximum with special designs required for aggregates between 1.5 and 1.7.

(3) Sand Equivalent not less than 45.

e. Asphalt concrete mixes shall meet the following requirements:

(1) Minimum optimum asphalt content 4.5 percent by weight of dry aggregate.

(2) Minimum stabilometer value of 35.

(3) Maximum swell of 0.030 inches.

(4) Minimum moisture vapor susceptibility of 30.

f. Specific controls should be placed on the gradation of the cold feet stockpiles. (It is understood that special requirements are being planned for the revised standard specifications which will take care of this requirement).

g. Special rolling requirements should be stipulated. Two alternatives are available; one, stipulating the equipment and procedure, and two, stipulating the end result. Until such time as end results can be reasonably evaluated, the procedural requirements are recommended. In general, the requirements of Section 212, Rolling, of the 1961 Standard Specifications are adequate except that the pneumatic-tired rollers should be equipped with 11.00 x 20-inch tires with the capability of changing tire inflation pressure, "on the run" from 30 to 105 psi. (This is taken from Reference 34 by Minor.) Rolling time would be based on an hourly schedule. Adequacy of rolling to be based on evaluation of water or air permeability. Rolling to be continued until no further increase in density is achieved or until the void content is estimated to be at 5 percent. Reference (38) describes a qualitative procedure for using the air permeameter to evaluate the amount of rolling required. Rolling patterns may also be developed on the basis of core samples and density measurements.

Discussions of rolling procedures will be included in the next portion of the report under Construction.

In general, mixing and placing of asphalt concrete shall be accomplished in accordance with Section 407 of the 1961 Standard Specifications.

Bituminous surface treatments should not be included as part of the original design for asphalt concrete.

It is recognized that the Department does not always stipulate material requirements since materials often come from State approved sources. If these procedures were to be followed for asphalt concrete, the above listed requirements should be met.

The need to establish quality control procedures in materials processing, handling, and placing is becoming of extreme importance. In order to provide for quality control, the Department has established minimum field and laboratory test requirements which are listed in the Field Test Manual of the Department. With reference to MTR Reference No. 407 for Plant Mix, it is recommended that the following requirements be established.

1. Hot bin grading samples for each 500 tons of plant-mix produced.

2. Hot extraction test (field) be performed at least once a day or for each 1000 tons of mix produced.

3. That manually operated Centrifuge Kerosene Equivalent test equipment be placed in each field

laboratory to assist in identifying when asphalt content changes should be made and the amount of the change. CKE tests should be made each day for the first few days of plant operation. After the mix has been "set", CKE tests will not be necessary unless changes are made in aggregate gradation.

4. Laboratory tests (Central) should include determination of stability (S-value), density, extraction, grading, and specific gravity (Rice method) determinations. Tests and information for sand equivalent data and asphalt properties should be continued as presently programmed. Standard procedures should be developed for summarizing results of each test for tabulation on IBM punch cards. Average values and standard deviations for each item should be included in the summary.

D. Construction Control Recommendations

Recommendations relative to construction are directed primarily at enhancing the tensile strength of the plant mix by reducing the void content at the time of construction. Recommendations are also made relative to (1) control of asphalt content and (2) for monitoring construction which requires that the untreated materials be placed during one season with the base and surfacing to be placed in a subsequent season.

A discussion of rolling procedures was contained in a previous portion of this section relative to a suggested specification for asphalt concrete.

The principal purpose in emphasizing compaction, density, and void content is to: (a) improve the durability response of the asphalt, (b) reduce the water permeability of the mix, (c) develop sufficient shear strength to prevent plastic deformation, and (d) increase the tensile strength and fatigue life potential of the mix.

Recent research has rather firmly established that the rate of hardening of asphalt is related to the proportion of continuous voids in the mix. It is generally true that the mixes with the greatest voids would also have the greatest number of continuous voids for unit volume. Since hardening of the asphalt does not appear to be a highly significant problem, this is not used as the primary reason for desiring higher densities.

The proportion of continuous voids is also associated with the permeability of the mix. The more permeable mixes are believed to be the most susceptible to stripping. In the absence of any significant amount of rutting or plastic deformation, it would seem that present compaction is adequate. However, for future traffic, as loads increase, the sensitivity to rutting will also be increased. As shown on the AASHO Road Test, increases in density reduced the rate at which rutting occurred.

Research in the United States, England, and Europe, as summarized in Reference (43), shows that tensile strength and fatigue life are inversely proportional to the total voids in the plant mix. It is primarily for this reason that emphasis is placed on density of the mix.

To develop procedures for optimizing density will require field experimentation. The Department is currently conducting field trials to obtain information relative to more effective ways of compacting mixes. For this purpose, a systematic series of experiments should be implemented to explore compaction procedures. Some of the most significant work to improve field compaction procedures has been carried out by the Washington State Highway Commission (34,38). The following is an interpretation of results from Reference (38).

1. "Pavement Density

"Four basic facts seem to appear each time an investigation of compaction of dense graded asphalt concrete is made.

a. "Standard compaction methods (steel breakdown rolling followed by high pressure pneumatic rolling and finishing with steel) have usually resulted in an air void (density) content of 11 ±2 percent.

b. "The simple expedient of switching the pneumatic roller to the breakdown position and using the steel rollers to compact edges, smooth the mat and finish has usually resulted in an air void content of 8 ±2 percent.

c. "Better densities have resulted each time compaction studies were made. When the roller operator discovers what can be done by aggressive rolling of stable mixes, even steel compaction improves.

d. "Air flow from 20 to 400 seconds for 300 mls. at 0.25 inches pressure usually means an air void content from 10 percent to 6 percent.

"Test results show that density is directly related to:

(1) Rate of surface exposure of the mixture (mix available, lift thickness, paver speed, width of spread) (width more important than length).

(2) Temperature of compaction completion (when pneumatic roller has rolled out of its tire tracks, i.e., number of passes).

(3) Sufficient compaction equipment to accommodate the requirement of points a and b.

(4) Ambient temperature and weather as it influences points a and b.

2. "Temperature of Compaction

"It seems unnecessary to state but the key to effective compaction of asphalt concrete with any type of equipment is <u>temperature</u>.

"Compaction must be completed at the highest possible temperature to achieve the highest densities. On these projects, where the pneumatic breakdown roller was used, it was possible in many instances to complete all but the finish rolling before the temperature reached 225°F.

"Tests show that prolonged rolling is of no value in mix consolidation when temperatures, roller weights, viscosity/temperature susceptibility of the asphalt exceed the compactive ability of the equipment available.

"With conventional compaction it has often been observed that it is necessary to delay rolling until the right temperature is reached to prevent marking or distortion. This practice depends on the increased viscosity of the asphalt to support the roller and thus inhibits its compaction.

3. "Lift Thickness

"Thicker lifts, without question, result in a marked improvement in density over thinner lifts. As lift thickness increases, the greater mass retains heat longer. With thicker lifts, the reduced length of pavement per ton of mix permits more coverages with the same rolling equipment before the mix temperature drops below that for optimum compaction. These factors are additive in achieving better compaction.

"If thick base lifts (0.5 ft) are placed, it may be necessary to provide roller support along the shoulder edge.

"As the thickness of the lift is increased, the need for a pneumatic tired roller becomes less important in obtaining optimum density. However, if maximum density is to be achieved, the advantage of the pneumatic over the steel roller is still evident with thicker lifts.

"As the thickness of the lift is decreased, the advantage of using the pneumatic roller for breakdown compaction to improve density becomes sharply evident."

The key to better compaction is high temperature rolling. For this reason, the breakdown roll and intermediate roll should be completed before the mix cools to 200°F.

Actual procedures will necessarily have to wait until planned field experiments are completed. In carrying out these initial experiments, Department engineers should be prepared for some initial difficulties with high temperature pneumatic tired rolling, e.g., grooving, pick-up, etc. However, based on the experience of other agencies, the net results should be beneficial to the overall performance of asphalt type pavement constructions.

It is recommended that changes in asphalt content should be made as a joint decision of the Resident Engineer and Materials Engineer. Observations on field projects indicate apparent changes in asphalt content. This could also be due to changes in aggregate gradation without changes in asphalt content. In any event, the performance of plant mix is very sensitive to the asphalt content as a function of surface area, surface texture and absorption. Changes in asphalt content represent a major change in terms of the properties of the mix. To assist in judging the need for or amount of change, a manually operated Centrifuge Kerosene Equivalent test should be part of the basic equipment of the field laboratory.

A number of Idaho projects have been constructed in stages with the embankment and portions of subbase and base during one season and the remaining base and surfacing the subsequent year. When such procedures are found necessary or desirable, it is recommended that the top lift (approximately 6 inches) be thoroughly tested for in-place density before the next construction layer is added. Laboratory tests for strength requirements should also be made to assure that design strength criteria are met. Recompaction or stabilization may be required in order to satisfy basic construction or specification requirements.

E. Recommendations for Structural Reinforcement

The main problem to the successful strengthening of existing constructions is the prevention of reflection cracking from the existing longitudinal load associated cracks. The type and amount of overlay will require further study. However, on the basis of the poor performance of a 0.1 foot overlay on project I-15W-80W, after one year, it was concluded that this amount of strengthening was not adequate.

Assuming that the lower 0.2 foot of surfacing on those pavements exhibiting longitudinal cracking is performing somewhat as an unbound layer, the amount of cover (ballast) could be estimated from the basic design curves or in accordance with engineering criteria established by the Department. The latter criteria would normally require a 0.4 foot layer of asphalt concrete over the unbound aggregate base for full design. Assuming the present surfacing layer to be equivalent to approximately 0.15 foot of asphalt concrete, an additional thickness of 0.25 foot would be required. If this thickness is used, some pretreatment of the existing surface is recommended. For example, a slurry seal or an asphalt rejuvenator (surface treatment) would be effective in providing continuity of the old surface prior to the placing of the overlay. Alternately, the thickness of the overlay could be increased to 0.30 foot.

It is recommended that some investigations be made to ascertain the amount of overlay required by constructing wedge sections of varying thickness. The thickness of these overlays should vary from 0.3 feet to 0.1 feet. The reduction in thickness should be in the direction of traffic.

VIII. FUTURE RESEARCH

In planning future research, emphasis should be given first to those major problems which can reasonably be expected to produce useful results within 6 to 12 months. As funds and personnel are available, long range studies should be initiated, according to need and the expected productivity.

Basic to any research program is the need to accurately identify problems. That is the principal objective of the project for which this report is being prepared. To do this will require: (a) the systematic collection of materials and construction data and (b) the continuing need to condition survey a portion of the highway system. Without this information, major efforts could be misdirected on the basis of isolated subjective impressions which are not significantly general to warrant research. These isolated problems should be handled on an individual basis at the local level. Liaison between District offices and the Headquarters Materials and Research Department is essential in cataloging local findings to specific problems.

Paving technology is progressing at a very rapid rate in the United States and in foreign countries. Universities, highway departments, federal agencies, and private industry are all combining to produce new and better technology for use in highway design and construction.

Assimilating and evaluating research is becoming a major problem for most agencies who have the responsibility for implementing the results of research. If these efforts are to be beneficial and productive and if it is possible to eliminate duplication, user agencies must expect to expend some matching effort. It is, therefore, recommended that the Materials and Research Department be adequately staffed to evaluate research in the light of Idaho's problems, practices, and needs and to implement field projects to document the potential advantages to the State. It is sincerely believed that research can be maintained at a realistic level if the efforts of others are fully exploited.

Specific recommendations for research are limited to statements of the type of problems considered to be important. To some extent these recommendations are repetitions; however, they are itemized here for emphasis. The development of experiment designs is beyond the scope of this report and in itself represents an important and major effort for any project. Recommendations are divided into two categories: (a) those which require immediate attention and (b) those which may be deferred temporarily.

1. Category 1
    a. Identify full extent of asphalt-aggregate stripping on statewide basis.

b. Develop laboratory tests to identify susceptibility to stripping.

c. Develop corrective procedures to eliminate stripping.

d. Develop case study program to further identify mechanism for longitudinal load crack to include the following items.

   (1) Cut transverse trenches across the pavement with tests for density, grading, etc. More particularly, this evaluation should include a measure of the in-place strength of the layered materials. Plate load tests are probably best suited for making evaluations of this type. It is recommended that Idaho develop its own plate load procedure. It should be quick and not require heavy reaction equipment. A suggestion would be to use an 8 or 12-inch diameter plate and load to 5, 10 and 15 psi in repetitive loading. The loading procedures used on the AASHO Road Test are given in Reference (3) and should be suitable. Analysis of this information can include the computation of the elastic modulus of each layer much as was done in References (3,7,39 and 40).

   (2) Strip maps should be prepared for each trench program in order to follow future performance.

   (3) A minimum of three trenches should be required for each test section.

   (4) Both distressed and non-distressed pavements should be evaluated.

e. Continue to study inter-relationship between asphalt properties and pavement performance. Some emphasis should be given to cold weather properties of the asphalt. Reference (41) describes some of the relationships between low temperature properties of asphalt and the cracking of pavements.

2. Category 2

   a. Develop alternate nondestructive test methods to identify load carrying capability of pavements. Measurements of beam deflections do not appear to be well correlated with performance. Vibratory methods as developed by the British Road Research Laboratory would be one possibility.

   b. Conduct continuous studies on selected projects with the basin beam developed by Idaho to determine if deflection measurements currently being reported are representative of the critical periods when damage is occurring.

   c. Continue to re-examine thickness design requirements, particularly for those pavements which are constructed to minimum design standards or on high R-value subgrades. This should automatically "fall-out" of pavement inventory or condition survey program. This would also be expected to produce information relative to the establishment of Regional Factors on a statewide basis and would provide information as to the adequacy of the R-value test to correlate with field performance.

d. Develop statistical procedures for collecting materials and construction information as a means for providing uniformity in evaluating quality control and as a basis for revisions in specifications.

Reference (33) describes in detail techniques for writing futuristic specifications. The information contained in this report can be helpful in understanding the use of statistical methods for highway engineering. Also, information in Reference (42) can be helpful in understanding statistical concepts. It is recommended that the Materials and Research Department begin to plan and eventually implement these techniques.

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APPENDIX A

PAVEMENT CONDITION SURVEY

Date:________________
Rater:________________
Project No.:________________

District:________________ Project Identification:________________

Limits of Project:________________ Length:__________

Traffic Information: ADT______ Commercial Vehicles per day______ T.I.______

Construction Period:________________ Age:__________

	Very Poor	Poor	Fair	Good	Very Good
Ride	0 — 1	1 — 2	2 — 3	3 — 4	4 — 5

Surface Appearance: Good______ Fair______ Poor______

Too much asphalt______ Not enough asphalt______ Satisfactory______

TYPE OF DISTRESS

Lane																	Summary	Approx. Length, %
Wheel Path	OWP				IWP				OWP				IWP					
Location	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4		
Rutting																		
Ravelling(Pitting)																		
Spalling																		
Corrugations																		
Cracking																		
Transverse (T)																		
Longitudinal																		
Type L																		
Type C																		
Alligator (A)																		
Ladder (D)																		
Shrinkage (S)																		

0 - No distress
1 - Minor distress
2 - Moderate distress
3 - Major distress

APPENDIX B

RECOMMENDATIONS FOR TABULATION OF PAVEMENT CONDITION INFORMATION

The following summarization provides for the inclusion of types of information considered to be pertinent to a pavement condition survey. It is recognized that the print-out of this information will be across the page rather than in a vertical listing as presented here.

In actual operation, the print-out sheet should include the various headings noted below. Also, adjustment in lines may be necessary in order to code each card for selective retrieval. That is, cards should be capable of being retrieved according to: (a) date, (b) project identification, and (c) Highway District, as a minimum. Retrieval by milepost would be desirable in the event that certain sections are selected for case studies.

The following tabulation identifies the lines, headings, and coding required for the summarization. It is suggested that this format be considered temporary and subject to review and modification by the Department.

Lines[(1)]	Heading	Description
1	District Number	Coding not necessary
3-14	Project Identification	Coding not necessary
15-17	Starting Milepost	Coding not necessary
18-20	Ending Milepost	Coding not necessary
21-22	Age	Coding not necessary - round-off to nearest year

(1) Inclusive

Lines[1]	Heading	Description
23	Surface Appearance	Good - 1, Fair - 2, Poor - 3.
24-26	Designation and Thickness of Surfacing	Designation refers to letter used in specifications to identify aggregate gradation; thickness to nearest .05 feet.
27-28	Thickness of Base	Thickness to nearest .05 feet.
29-30	Thickness of Subbase	Thickness to nearest 0.1 feet.
31-32	Thickness of Select Borrow	Thickness to nearest 0.1 feet.
33-36	Subgrade Soil Classification	Use AASHO Group Index Designations
37-39	Asphalt Grade - wearing	Use lower limit of penetration grade, 120 for 120/150, etc.
40-42	Asphalt Grade - binder	Use lower limit of penetration grade, 40 for 40/50, etc.
43-45	Asphalt Grade - base	Use lower limit of penetration grade, 40 for 40/50, etc.
46-47	Asphalt Supplier	To be numerically coded.
48-49	Asphalt Content - average	Job average to nearest 0.1 percent.
50-51	Asphalt Content - distribution	Standard deviation to nearest 0.1 percent.
52-53	Average Daily Traffic	In thousands to nearest 0.1, i.e., 2470 ADT coded as 2.5.
54-55	Commercial Vehicles per Day	In hundreds to the nearest 0.1.

(1) Inclusive

The following lines are to be used for results of field evaluation with two lines for each item. The first line will be the level of distress from 0 to 4, the second line for extent of distress, each as suggested in the report but adding one additional category for level of distress.

Line(1)	Heading
56-57	Rutting
58-59	Ravelling
60-61	Spalling
62-63	Corrugations
64-65	Transverse Cracking
66-67	Longitudinal Cracking - Construction
68-69	Longitudinal Cracking - Load
70-71	Block Cracking - Alligator
72-73	Block Cracking - Shrinkage
74-75	Ladder Cracking

(1) Inclusive