THE MICHIGAN PAVEMENT PERFORMANCE STUDY

FOR DESIGN CONTROL AND SERVICEABILITY RATING

By

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INTRODUCTION

The Michigan Pavement Performance Study
is a cooperative research program conducted
by the University of Michigan under a research
contract sponsored by the Michigan State High-
way Department in cooperation with the U. S.
Bureau of Public Roads. This study had its
beginning in 1946, immediately after the sec-
ond World War, when the interest in highway
design and performance was stimulated by the
greatly increased demand on the highway trans-
portation system and a nation-wide accelera-
tion in highway construction. The program
was formulated and has since been conducted
on the proposition that pavement performance
and adequacy could best be measured by care-
ful observation of the response of existing
pavements to the physical conditions and
forces to which they are subjected in actual
service.

Many have asked the question, the answer
to which is still the subject of vigorous de-
bate, how do you know when a pavement is
structurally adequate? How thick must a
pavement be to carry a specified axle load?
Should a pavement be rigid to bridge over a
weak subgrade, designed as an elastic beam on
an elastic support (Westergaard theory)? Or,
should it be flexible in order to develop full
subgrade support, relying on the time-honored
statement of MacAdam that it is the subgrade
which really carries the pavement and the
loads also. How is subgrade support incorpor-
ated in a design formula? How about drainage,
frost action, and soil type? These are the
uncontrolled factors of environment -- how do
they affect service behavior and how are they
recognized in pavement design? What is the
relative effect of volume and character of
traffic?

Pavement Condition Surveys in Michigan

Michigan believes that the ultimate ans-
wers to these questions can be found in the
behavior of roads that have been subjected to
these conditions over many years. Well organ-
ized pavement condition surveys in Michigan
date back to the middle twenties, when the
late V. R. Burton organized a group of re-
search workers who started a series of state-
wide pavement condition surveys, including
comprehensive data on soil conditions and cli-
matic environment.[1,2,3] This work has been
carried on over the years by a number of indi-
viduals, including Kellogg, Benkelman, Stok-
stad, and Olmstead, whose names have become
well known among highway engineers and soil
scientists.

These investigators early found a sig-
nificant correlation between pavement per-
formance and environment, including soil type,
drainage, and climatic factors -- a viewpoint
which has continued to exert a dominant influ-
ence on pavement design in Michigan. Improve-
ments in this approach to pavement design have
led to more accurate evaluation of soil condi-
tions, drainage, and climatic environment, the
utilization of local soil materials of favor-
able characteristics, and the selection of

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pavement structures which more fully utilize available subgrade support. While it is recognized that many of these factors are uncontrolled variables difficult to measure and perhaps impossible to express in a mathematical formula, current pavement performance studies in Michigan have been predicated on the belief that these uncontrolled variables could be reliably evaluated by more accurate field surveys and objective analysis of the results.

Even though many of these factors are not susceptible to design control, these same uncontrolled variables are those for which successful pavement design must provide. Two factors were introduced in pavement performance studies in Michigan in recent years to serve as a quantitative measure of pavement performance. It was felt that the integrated result of these uncontrolled variables could be measured by changes in the pavement profile (roughness index) and structural continuity (cracking pattern). These two factors in pavement performance are the quantitative counterparts of riding quality and durability, the two most important attributes of the highway pavement.

While it may appear to be only stating the obvious, highway builders from the very first have focused their attention on pavement roughness and cracking as direct evidence of the success of their pavement design and construction. What has been lacking, however, is a practical yet sufficiently accurate method of measuring these indexes of pavement capacity and durability in quantitative terms. As once stated by Lord Kelvin, "When you can measure that of which you speak, and express it in numbers, you know something about it." These accurate, quantitative measures of pavement performance are also necessary in order to identify physical changes in the pavement associated with its design and its ability to endure the physical conditions of service.

Corollary to this is the fairly obvious fact that such measurements can only be made under actual service conditions on pavements subjected to real traffic in the natural environment in which they must serve over a period of years that may mark the useful life of each pavement. That there are others who share this view is indicated by the following quotation from a report by the Committee on Rigid Pavement Design of the Highway Research Board in stating the objective of a high priority project on the "Investigation of Existing Pavements". In discussing a number of recent changes in the design of rigid pavement and their effect on pavement performance, it was stated, "It is also believed that, in many respects, the pavements which are presently in existence constitute the only dependable sources of information on which to base future designs."

Pavement Performance - Willow Run Airfield

The first opportunity to test this concept in practice came in 1946, immediately after World War II. At that time, the University acquired title to Willow Run Airfield as war surplus, with the primary objective of developing the facility as a research center. Acquisition of the airfield property involved a definite and somewhat perplexing responsibility on the part of the University of Michigan and the Airlines National Terminal Service Company, Inc. (ANTSCO), the agency operating the field. As stated in the deed, it was required ". . . that the entire landing area . . . and all improvements, facilities, and equipment of the airport property shall be maintained at all times in a good and serviceable condition to insure its efficient operation." The major investment in the airfield property was in the airport pavements; the question immediately arose as to how serviceability was to be measured and how it could be demonstrated that the pavements were maintained at all times in a "good and serviceable condition".

In addition to the normal maintenance required for operation of the field, it was decided that a periodic pavement condition survey would be made to keep an accurate inventory

of the physical condition of the paving over the period of years that the field was in operation. Starting in 1946, and at intervals of approximately five years, aerial photographs of all paved areas have been taken to determine the cracking pattern and the changes in structural continuity of the pavement under its actual service conditions. More recently, since substantial portions of the original concrete pavement have been resurfaced with asphaltic concrete, procedures for making the pavement condition surveys have been altered to correspond with those more applicable to flexible pavements. Pavement roughness, reflected cracking, and the general condition of the pavement surface have taken the place of the continuity ratio computed from the cracking pattern, which can no longer be precisely determined.

It should be here noted that many of the procedures developed in connection with the Michigan Pavement Performance Study have first been used in evaluating the condition of rigid pavements of Portland Cement concrete, and later applied to flexible or asphaltic pavements, which are the particular subject of this conference. This sequence of events is entirely consistent with experience in this country on many miles of concrete pavements. When these concrete pavements have cracked enough to become flexible, they deflect until they have mobilized sufficient subgrade support to sustain them without continued deformation. In the meantime, they may have become rough and lost their riding quality; it has become common practice in this country to resurface these pavements with asphaltic concrete to recover their riding quality and to protect the subgrade from further exposure to the elements. Assuming that the ultimate supporting capacity of the stress-conditioned subgrade is sufficient to sustain the applied loads, such pavements may have many more years of useful life. Such resurfaced pavements are generally classified as "asphaltic" or "bituminous" pavements; the broken concrete slab may serve as a very effective base course for

distributing the loads to the supporting subgrade. The structural behavior of such a composite pavement has many things in common with asphaltic concrete pavements in general, and may be very appropriately used to demonstrate certain basic principles involved in the design of asphaltic pavements. This statement certainly applies to pavement condition surveys and procedures used to evaluate pavement performance.

From this viewpoint, Willow Run Airfield provided an unusual opportunity to serve as a field laboratory for studying pavement design and performance; it was in this program that basic principles and techniques now used in the Michigan Pavement Performance Study had their beginning. Complete details of the airfield pavement at Willow Run have been published elsewhere,[4] and will not be repeated here except where these details refer particularly to deficiencies having a significant influence on pavement performance.

There are approximately 1,500,000 square yards of pavement on the airfield, roughly equivalent to 115 miles of 22 foot highway pavement, which provides an adequate sample on which to study pavement performance. This pavement was constructed in three contracts, in 1941, 1942, and 1943. There were significant variations between the 1941 and the 1943 construction, which developed some sharp contrasts in performance and served to illustrate design or construction defects.

Figure 1 is an aerial photograph taken at the south end of the main west apron, showing typical sections of the apron, constructed in 1941, and additions to the apron and the curved taxiway, constructed in 1943. This photograph was taken in 1946, when the University of Michigan acquired title to the airfield, and shows the condition of the paving at the beginning of the 15 year service period under University operation. Although the pavement at this time had been subjected to almost negligible service in terms of load repetition, the 1943 additions are beginning to show a considerable amount of transverse

3

cracking, with virtually no cracks having de-
veloped in the 1941 construction.

Figure 2 is an aerial photograph of the
same general area, taken in November, 1950,
after four years of service under commercial
airline operation. The 1943 construction al-
ready showed serious crack development, with
transverse cracks in the center of a large
percentage of the 25 foot slabs and unusual
pattern cracking developing in certain lanes,
with some slabs already having been replaced
as shown by the light colored areas. The
1941 construction, on the other hand, shows
very limited development of single transverse
cracks subdividing the 25 foot slabs into two
slab lengths.

One of the most interesting developments
shown in Figure 2 is this pattern cracking,
developing along the edge of every fourth
lane in the 1943 apron. The cause of this in-
cipient cracking was traced to the fact that
this section of the apron was poured in alter-
nate 20 foot widths, and the concrete mixer
was permitted to travel on the recently com-
pleted slab while adjacent lanes were being
poured. This weakness was also associated
with the fact that the paving was done during
the fall of the year under unfavorable weather
conditions. The concrete on which the mixer
traveled was not completely cured, and its
strength was not sufficient to carry the con-
centrated load of the mixer at the edge of the
slab without damage. As a result, these fre-
quently accepted compromises in paving prac-
tice contributed more to the deterioration of
this badly cracked pavement than any other
factors in design or construction.

PAVEMENT PERFORMANCE CRITERIA

Using the pavement at Willow Run to il-
lustrate the development and application of
performance criteria, it is desired first to
emphasize that the primary objectives in eval-
uating pavement performance are to check de-
sign procedures by determining those factors

responsible for good performance, and to iso-
late design or construction deficiencies in
order to identify the physical conditions or
factors responsible for these deficiencies.
A secondary objective, as previously mentioned
in the case of Willow Run Airfield, was to es-
tablish a quantitative basis for measuring
pavement condition or serviceability and a
reliable procedure by which to measure changes
in serviceability and determine the cause of
these changes. From a study of pavement con-
dition surveys and general observation of
highways in service, it had been concluded
that pavement performance or serviceability
could be measured in its major practical as-
pect by two factors which may be referred to
in general terms as structural continuity and
riding quality or its antithesis, roughness.

The Continuity Ratio and Cracking Index

The first of these, structural continu-
ity, derived from the cracking pattern, re-
flects the structural adequacy of the pavement.
The changes in structural continuity or the
rate at which a pavement breaks up may serve
as a means of anticipating the future life of
the pavement. A definite measure of struc-
tural continuity was formulated in terms of
two related quantities which have been defined
as the continuity ratio and the cracking index.

> The continuity ratio (CR) is the ratio
> of the average existing slab length,
> or unbroken slab area, of the pavement
> divided by a selected standard length
> or area representing normal subdivision
> of a concrete pavement, which does not
> substantially impair its serviceability.

Teller and Sutherland found,[5] for example,
that the normal subdivision of a concrete pave-
ment due to warping and shrinkage stresses and
other environmental effects over a period of
years resulted in a normal crack interval
which, in general, varied from 10 to 20 feet.
On this basis, 15 feet was selected as a
standard slab length or reference for trans-
lating the continuity ratio into numerical
terms. Thus, the continuity ratio would be
computed as the average length of uncracked
slab divided by 15 or, in the case of longi-

tudinal cracking or pattern cracking, an equivalent area for any given slab width. While various aspects of the continuity ratio will be discussed in considerable detail throughout this report, it is now desired only to point out that, as a matter of definition, average slab lengths greater than 15 feet will result in continuity ratios greater than unity, while the subdivision of a concrete pavement into slab lengths of less than 15 feet will produce continuity ratios less than unity. Accepting a slab length of 15 feet as representative of normal subdivision of a concrete pavement, regardless of load repetition, it would follow that continuity ratios greater than unity indicate that the structural integrity of the pavement or ability to carry load has not been impaired.

On the other hand, it is presumed that when the subdivision of the slab results in an average length of less than 15 feet, it must be accepted that such a pavement is in the stage of progressive failure under load application, with a probable useful life which may be reduced quite rapidly as the continuity ratio decreases. To give this stage of more rapid deterioration added significance, the cracking index was introduced and becomes the principal measure of structural continuity, or rather the lack of structural continuity, during this stage of progressive failure.

> The cracking index (CI) was defined as the ratio of uncracked slab length or equivalent area to the selected standard length or area expressed as a percentage and subtracted from 100 per cent; thus, the cracking index is the complement of the continuity ratio, expressed as a percentage.

The cracking pattern and other evidences of surface deterioration that may reflect pavement condition have been obtained from aerial photographs, sometimes supplemented by a field survey. Having in mind asphaltic pavements in which cracking is much more difficult to photograph, cracking and surface conditions indicative of deterioration may be determined by field surveys and visual inspec-

tion. More recently, a great deal of attention has been given to strip photography of pavement surfaces, which has reached a stage of development that is quite satisfactory in revealing pavement condition. Costs have also been reduced to a point that makes such strip photographs feasible to use in pavement condition surveys. In a great many projects in which the cracking has not progressed to an advanced stage, very satisfactory mapping and recording of the crack interval can be carried out in connection with pavement profile surveys which will be discussed later in some detail.

Riding Quality Defined by a Roughness Index

Attention may now be directed to the second measure of pavement performance, which reflects riding quality and may be defined by a roughness index. To the highway user who drives over the pavement without stopping to look for cracks or surface imperfections, riding quality or its antithesis, roughness, is the ultimate measure of pavement performance. Vertical displacements at joints or cracks, or other pavement displacements sufficient to produce a reaction from the motorist, represent poor pavement performance, aside from the fact that such roughness may also be an indicator of advancing structural failure due to lack of continuity. Such structural deterioration not only affects load carrying capacity -- such discontinuities become the focal points of faulting, which ultimately reduces the riding quality of the pavement below acceptable standards. Looking beyond the complex interaction of uncontrolled variables and other aspects of pavement roughness, it may be said that the ultimate objective of any pavement is to provide riding quality, a commodity which is purchased at a cost.

It seems reasonable to say that from the very beginning pavements have been built simply to provide durable surfaces with improved riding quality for the safety, comfort, and convenience of the highway user. It has already been pointed out that pavement roughness

as an index of pavement performance is certainly not new; the same thing applies to structural continuity as, in general, pavement builders have found no better way to gauge the success of their endeavors than by direct observation of their pavements under the service conditions to which they are subjected. Consequently, they have used cracking and roughness as the primary bases for pavement evaluation. While a more detailed discussion of equipment and procedures will be included later in this paper, it is necessary at this point in the discussion only to define the roughness index as it has been used in the Michigan Pavement Performance Study.

> The Michigan roughness index (RI) is defined as the cumulative vertical displacement of the pavement surface in inches per mile.

Progressive Changes in the Continuity Ratio

As previously noted, in the early years of the operation of Willow Run Airfield, cracking or changes in structural continuity served as the primary basis of measuring pavement performance and maintaining a satisfactory level of serviceability. Aerial photographs, similar to those shown in Figures 1 and 2, were taken of all paved areas at intervals of approximately five years. Procedures were established for computing the continuity ratios of all runways, taxiways, and parking aprons, following the definition that has been given. The pavement was laid in widths of 20 feet, with a longitudinal keyed construction joint at both sides and a dummy joint at the center of the 20 foot widths, subdividing the pavement into 10 foot lanes. There were transverse expansion joints at a spacing of 125 feet, with 3/4 inch premolded filler and 3/4 inch round steel dowels at 12 inch centers, and dummy contraction joints at a spacing of 25 feet. With a spacing between joints of 25 feet, the initial continuity ratio of the pavement was 1.67 (25 ÷ 15 = 1.67). In the early years, the most prevalent type of cracking in the major portions

of this field, consisting of runways, taxiways, and aprons built in 1941, was a single transverse crack subdividing the slab length of 25 feet into two parts. Table 1 is a summary of the cracking at the time of the 1950 survey, four years after the field was placed in operation. It also shows the marked contrast in deterioration in the form of cracking between the 1941 and 1943 construction, which is directly related to poor construction practice.

In Figure 3 is shown the change in the continuity ratio in the centrally located Taxiway B at Willow Run over a period from its original construction in 1941 until it was resurfaced in 1955. A portion of this center taxiway has been selected as a typical paved area from the 1941 construction to illustrate both computation of the continuity ratio and procedures developed to correlate performance with design, to maintain a given level of serviceability. This area has been selected for two reasons: first, it is subjected to the maximum load application on the field; and second, there was a known deficiency in relation to pavement design.

With respect to load repetition, a brief summary of loading conditions is in order. The U. S. Army Corps of Engineers rated the field in 1944, under "Capacity Operation", at maximum loads of 52,000 pounds gross plane weight for the runways and 41,600 pounds for the field, as limited by the 1943 construction; "capacity operation", as then defined, was based on a 20 year life and 100 scheduled operations per day. When a traffic analysis was made in 1954, commercial planes supplied 80 per cent of the traffic, the remaining 20 per cent being military and civil aircraft. The airline traffic alone amounted to 135 scheduled operations per day, with the gross weight of planes varying from 26,200 to 132,000 pounds. An analysis indicated that 5 per cent of the traffic exceeded the rated maximum loading by 200 per cent, 20 per cent of the traffic exceeded that loading by 100 per cent, while only approximately 15 per cent

6

of the traffic was equal to or less than the
rated maximum loading. Since 1954, gross
plane loads have increased rather than de-
creased; and, even with some recent reduction
in commercial traffic, scheduled operations
during these years exceeded the established
"capacity operation".

With respect to design considerations,
reference has already been made to poor con-
struction practices which accounted for a
sharp contrast in performance between the
1941 pavement construction and that which was
placed in 1943. There were other structural
deficiencies in the pavement construction
which are pertinent to the present discus-
sion. Willow Run Airfield was built in 1941
by the Ford Motor Company under a Defense
Plant Corporation contract. As a consultant
to the Ford Motor Company and their architects
and engineers, the writer had an opportunity
from the beginning of the project to become
familiar with the construction of the field
in considerable detail.

The entire field is located on an outwash
plain of sand and gravel which, over most of
the field, provided an ideal subgrade with
practically unlimited subgrade support in con-
junction with excellent subsurface drainage.
However, in the north-central portion of the
airfield was a lowlying area of virgin hard-
wood with a heavy accumulation of forest de-
bris and organic material, and a water table
close to the surface. Subdrainage was pro-
vided to lower the water table beyond the
normal depth of frost penetration, but it
proved difficult under the emergency construc-
tion conditions then in effect to enforce ef-
fective control of the grading operation and
selective placement of subgrade soil. Conse-
quently, in several of the runways and the
center taxiway, there were areas where failure
to remove the topsoil and organic matter and
to make more adequate provision for surface
drainage affected the pavement behavior in
later years.

One of these areas was in the central
portion of the central taxiway; it is this

area to which particular attention is directed
in the present discussion. As shown in Figure
3, serviceability in terms of the continuity
ratio decreased more rapidly after airline op-
erations started in 1946. By 1950, it became
necessary to start some slab replacement in
the critical area, an operation which contin-
ued periodically for the next five years, un-
til the taxiway was resurfaced with asphaltic
concrete in 1955. Continuity ratios for the
paved area in question are shown as computed
before and after slab replacement, with a
dashed line showing what the continuity ratio
of the original pavement would have been had
there been no slab replacement.

Continuity ratios shown in Figure 3 are
the averages for the entire paved area in
question, and there is significant information
obscured by these averages which must be given
consideration in any objective correlation be-
tween pavement performance and design. This
is one of the weaknesses of statistics result-
ing from the use of averages of one type or
another to obtain a broad correlation between
design factors. In such averages, the ex-
tremes of behavior or abnormal results are
averaged out or discarded, when actually
these radical departures from the norm fre-
quently provide the most significant and use-
ful information on design. In other cases,
relatively small differences in large numbers
carry the full import of basic design factors;
these, too, are lost in statistical manipula-
tion.

Further analysis of the continuity ratios
shown in Figure 3 provides an excellent exam-
ple of the necessity for developing a proce-
dure which goes beyond the statistics and can
be followed, step by step, back to the behav-
ior of individual slabs, if necessary to
identify the basic design deficiency.

In Figure 4 are shown the continuity ra-
tios computed for each lane in the critical
area of the center taxiway for each survey
from the time of construction to the time it
was resurfaced. It is significant that the
four middle lanes, two on either side of the

7

center line, as shown in Figure 4, carried practically all of the channelized traffic down the center taxiway. As a result, the continuity ratios of these lanes decrease most rapidly during the period of airline operation, until they reach minimum values between 0.1 and 0.2, which would correspond to cracking indexes of 90 per cent and 80 per cent, respectively. At this stage, the pavement has been reduced to a block pavement in which a cracking index of 90 per cent, or a continuity ratio of 0.1, would correspond to an unbroken slab area or block measuring approximately 3 feet by 5 feet. Continuity ratios in the outside lanes with no load application remain high.

Computation of the continuity ratios in each lane requires detailed analysis of the cracking pattern from the aerial photographs, carrying back to the individual slab. As shown in Table 1, cracks were classified as transverse, longitudinal, or diagonal. A slab was considered to have developed pattern cracking when it had three transverse cracks and one longitudinal crack, giving a continuity ratio of 0.2 and a cracking index of 80 per cent. A slab which had failed or had been removed was considered as the equivalent of one having four transverse cracks and two longitudinal cracks, giving it a continuity ratio of approximately 0.1 and a cracking index of 90 per cent. As shown in Figure 4, a large percentage of the slabs in the four middle lanes of the critical area of the center taxiway had either, by 1955, been reduced to pattern cracking or the slabs had been replaced.

However, as shown in Figure 3, the average continuity ratio in the paved area under consideration had been maintained between approximately 0.7 and 0.8, or at a cracking index of from 20 to 30 per cent. This was considered to be an acceptable pavement condition, and was selected as the serviceability level to be maintained for the critical area chosen for this study. While this is in itself a useful objective, it is important to

recognize that this serviceability average, or index if it may be called that, will vary through wide limits depending upon the size of the paved area to which it is applied, ranging from the individual slab to perhaps several miles of pavement. This may be illustrated graphically by Figure 5, in which the continuity ratios have been computed for large sections of the pavement at Willow Run Airfield; therefore, the figures which will be designated as the _field average_ must be viewed in quite a different perspective. The two runways shown in Figure 5 represent the average continuity ratio for the equivalent of approximately 10 miles of 22 foot highway pavement. Taxiway B is a paved area roughly the equivalent of 3 miles of highway pavement; other paved areas evaluated are of comparable size. Continuity ratios for runways, aprons, and taxiways were reported in terms of field averages in 1955.[6]

A comparative analysis of the data given in Figures 3, 4, and 5 is enlightening and illustrates the significance of average continuity ratios as an index of serviceability, as governed by the size of the paved area for which the average has been computed. Continuity ratios in Figure 5, for paved areas representing the equivalent of from 3 to 10 miles of pavement, provide an average which is reasonably representative of the airfield pavements at Willow Run. Soil conditions, climate, pavement type, and loading are all involved in samples large enough to provide a valid measure of pavement performance as an average for the field as a whole. In terms of such large samples, there are significant differences which relate to design or construction deficiencies and variations in the number of load repetitions on different sections of the field. Runway 4L-22R shows an almost negligible change in continuity ratio in twelve years of service under runway loading, which is considerably less severe than in the case of taxiways and aprons. This good performance also reflects ideal subgrade conditions in which there was no known deficiency introduced during

8

construction. Runway 9L-27R has been sub-
jected to the same loading and is comparable
in all other respects, except that there is a
known subgrade deficiency over the westerly
portion of the runway, which led to more rapid
deterioration of the pavement in that area.
Taxiway B is comparable to Runway 9L-27R, ex-
cept that the load concentration is greater,
which accounts for the more rapid decrease
in continuity ratio over most of the service
period. On the other hand, at the last obser-
vation in 1953, Taxiway B had been improved by
more extensive slab replacement than in Runway
9L-27R, which narrowed the differential in
performance. The 1941 apron is still in rea-
sonably good condition, being rated as ade-
quate, with a continuity ratio greater than
unity. However, this pavement, with no known
design or construction deficiencies, reflects
the influence of more severe loading condi-
tions than on Runway 4L-22R, with which it
may be compared. The contrast between the
1941 and 1943 apron construction has already
been commented upon, and the factors which
produced the poor performance of the 1943
construction have been identified.

From the standpoint of pavement perform-
ance and design, perhaps the most important
comparisons to be made are those between the
average continuity ratios for large areas in
Figure 5 and those which have been computed
for critical areas of known deficiencies in
Figures 3 and 4. In November, 1953, the av-
erage continuity ratio for the entire Taxiway
B is approximately 1.1, taken from Figure 5,
as compared to 0.78 for the partially rebuilt
pavement in the critical area shown in Figure
3. The corresponding continuity ratio for
the original pavement before slab replacement
was 0.61 in November, 1953, which was an aver-
age for the full width of the taxiway in the
critical area. This may be compared with a
continuity ratio varying from 0.15 to 0.2, as
shown in Figure 4, for the four central lanes,
which represents the effect of heavy concen-
tration of load application on a pavement
whose lack of supporting capacity has led to

its reduction to a block pavement. The con-
trolling factors were excessive subgrade de-
flection in combination with a weak unrein-
forced concrete pavement, which was still too
rigid to mobilize the available subgrade sup-
port until both it and the subgrade had be-
come stress-conditioned in service.

Resurfaced Pavements - Willow Run Airfield

The performance of this pavement after
being resurfaced in 1955 is quite a different
story and illustrates the advantages of flexi-
bility combined with adequate subgrade sup-
port. The results of a pavement condition
survey made in October, 1961, have been re-
ported in considerable detail,[4] and may be
summarized briefly for the present discussion.
The resurfaced pavements are in good condition
after six years of service under heavy airport
loading. Reflected cracking has not been a
serious problem and has been readily con-
trolled by timely maintenance, consisting
mainly of resealing in 1960. In 1957, after
two years of service, the reflected cracking
varied from 25 to 40 per cent and averaged 30
per cent. One area of badly cracked pavement
in the 1943 apron had not been resealed and
showed a reflected cracking of approximately
50 per cent in 1961. There has been no per-
ceptible rutting or displacement of the bitu-
minous surface course.

As previously indicated, the continuity
ratio can no longer be used to evaluate per-
formance of the resurfaced pavement; there-
fore, changes in the pavement profile, in
terms of vertical displacement or roughness,
have been adopted as a basic measure, supple-
mented by observations of the surface condi-
tion described in the previous paragraph.
Test runs in 1961, with the Michigan Truck-
Mounted Profilometer, on Runway 9L-27R show
that the roughness index in the two outside
lanes, which had not been resurfaced, ranged
from 234 to 315, with an average of 267 inches
per mile, which is rated as extremely rough.
These outside lanes have had practically no
wheel load applications; thus, this roughness

was caused by frost displacement and tempera-
ture differentials in the annual cycles of
freezing and thawing. Lack of adequate sur-
face drainage at the edge of the runways was
one of the deficiencies in the original con-
struction, with the result that the edge lane
was subjected to more severe frost action.
The extreme roughness which has resulted is
consistent with highway roughness values for
comparable conditions.

The four center lanes of this runway,
which were resurfaced in 1959, had roughness
indexes in 1961 varying from 74 to 80, with
an average of 77 inches per mile, which would
be rated as "good" in terms of riding quality.
The two middle lanes of the center taxiway,
resurfaced in 1955, have also been profiled
and had roughness indexes in 1961 varying
from 78 to 88, with an average of 83 inches
per mile, which would also be rated as "good".
The center lanes of the runway and, particu-
larly, the center taxiway are subject to heavy
load repetition; while the traffic volume can-
not be compared to highway traffic, the magni-
tude of the loads is considerably greater.

The performance of the resurfaced pave-
ments at Willow Run provides evidence to sup-
port several important conclusions concerning
pavement design. In the first place, the
original concrete pavement would have to be
rated as a structural failure in terms of
rigid pavement, having been reduced to a block
pavement without structural continuity and de-
pending entirely upon subgrade support to
carry the heavy wheel loads of modern airline
traffic. In the second place, the ultimate
supporting capacity of the granular type sub-
grade has proved to be sufficient to carry the
heavy load concentrations applied without dam-
aging deflections, once the known deficiencies
in subgrade support had been corrected by what
may be termed stress-conditioning in service.
A third factor, which should not be minimized,
is the fact that after the concrete pavement
had been reduced to a block pavement, it
gained sufficient flexibility to fully mobil-
ize the available subgrade support. Finally,

it should be said that the favorable condition
which has been produced is the result of a
fortuitous combination of favorable natural
conditions which have produced a serviceable
pavement in spite of the complete failure of
that pavement to function as it had been de-
signed.

WIRE REINFORCEMENT INSTITUTE PROJECT

The next opportunity to test the continu-
ity ratio and roughness index as measures of
pavement performance came on a cooperative
research project sponsored by the Wire Rein-
forcement Institute. This study was conducted
by the University of Michigan in cooperation
with the Michigan State Highway Department,
with the objective of evaluating the effect of
steel reinforcement on the performance of con-
crete pavements under actual service condi-
tions. Pavement performance surveys were con-
ducted on selected projects in southern Michi-
gan which had been in service for periods
varying from 10 to 25 years. Each project in-
cluded comparable reinforced and unreinforced
sections which were observed over a period of
approximately five years, from 1952 to 1956.
Aerial photographs, supplemented by field sur-
veys, were used to record the cracking pattern
from which the continuity ratio was computed.
Roughness was measured by the U. S. Bureau of
Public Roads' single wheel profilometer. The
results of this project were reported in
1954.[6,7]

This investigation added to the evidence
showing that the behavior of existing pave-
ments in actual service could be measured in
realistic terms by carefully conducted pave-
ment condition surveys. Characteristics of
pavement performance were again expressed
quantitatively by the continuity ratio, which
characterized the structural condition of the
pavement, and a roughness index, which gauged
significant variations in riding quality. It
was found from an analysis of pavement condi-
tion in these selected projects that pavements

with steel reinforcement were measurably smoother and that cracking was measurably less than in unreinforced pavements. Experience on this project, in combination with studies made at Willow Run Airfield, encouraged the continuation of this approach; when the opportunity arose, a state-wide survey of the Michigan trunkline system was undertaken. This project, which was designated as the Michigan Pavement Performance Study, was initiated in 1957 and has continued to date.

MICHIGAN PAVEMENT PERFORMANCE STUDY

The Michigan Pavement Performance Study, as organized in 1957, was conducted by the University of Michigan through the Office of Research Administration. From early 1957 to July 1, 1959, the project was sponsored by a group of agencies associated with the trucking industry, with the cooperation and assistance of the Michigan State Highway Department. The sponsoring agencies were: the Michigan Trucking Association; the American Trucking Associations, Inc.; and the Motor Truck Division of the Automobile Manufacturers Association. Since July 1, 1959, the project has been sponsored by the Michigan State Highway Department in cooperation with the Bureau of Public Roads. Under this sponsorship, the project is a part of the Michigan Highway Planning Survey Program financed by HPS funds under the supervision of the U. S. Department of Commerce Bureau of Public Roads. Results from several phases of this investigation have been presented in various forms including unpublished departmental reports, other unpublished papers, and several published papers listed as references in this report. Results of this investigation will be summarized briefly in this paper to illustrate the type of information obtained from field surveys and its value in pavement design, construction, and maintenance. Reference may be made to the listed publications for a more detailed study. In summarizing these results,

it is pertinent to take some note of the magnitude of the investigation and the coverage that it affords of the Michigan trunkline system as a whole.

During the four years, from 1958 up to the present time, that the Michigan Truck-Mounted Profilometer has been in operation, more than 9500 lane miles of pavement profile have been recorded. All but a negligible part of this mileage is on the Michigan trunkline system of some 9435 miles, which include 8050 miles of two lane pavement, 135 miles of three lane pavement, and 1250 miles of divided four lane pavement. The trunkline system thus amounts to some 21,500 lane miles of pavement; in the four years of profile surveys, the total mileage of pavement profiles available amounts to approximately 40 per cent of the trunkline mileage, after allowing some 900 miles for duplication of profiles on special projects. These pavement profiles provide state-wide coverage and some mileage of every classification and type of paved road in the trunkline system. In presenting these data, it has already been noted that the primary objectives of the Michigan Pavement Performance Study were to check Michigan design procedures, determine those factors controlling pavement performance, and identify physical conditions which contribute to poor performance. Another objective, which has been added more recently because of current interest in the AASHO Road Test, has been to explore the possibilities of using pavement performance criteria developed in connection with surveys of existing pavements in correlating pavement design and performance in Michigan with the AASHO Road Test and the procedures that have been recommended as a result of that test.

The Michigan Truck-Mounted Profilometer

In the earlier Michigan pavement condition surveys that have been discussed in this paper, emphasis was placed on structural continuity, as measured by the continuity ratio, as an index to pavement performance. It was recognized, however, that pavement roughness

11

as a measure of riding quality was also a very important factor which, in itself, may be an excellent index to pavement performance. It was felt, however, that pavement roughness should be measured from an accurate surface profile which, aside from supplying a roughness index, could provide a valuable record of the configuration of the pavement surface at any one time and serve as a more accurate and reliable basis for determining changes in the pavement profile over a period of time. In this regard, changes in the pavement profile, such as faulting at joints and cracks, may be identified with specific conditions and locations which may be more closely related to design in terms of pavement deficiencies.

The equipment selected for measuring and recording accurate pavement profiles is the truck-mounted profilometer shown in Figure 6. This truck is equipped to trace and record an accurate profile in each wheel track of the pavement. Two sets of so-called "bogie wheels" located in front and back of the truck, 30 feet apart, provide reference points on the pavement surface from which vertical displacement is measured by the recording wheel midway between the two sets of bogie wheels. Pavement profiles are recorded on a continuous chart and may be retraced after any designated period of service to measure changes that may have taken place. The tracing and recording mechanism is connected to electronic integrators which measure the cumulative vertical displacement of the pavement in inches for any selected length of pavement. This cumulative vertical displacement, when expressed in terms of inches per mile, is used as a roughness index in subsequent pavement classification. The cumulative displacement is stamped on the profile chart for each quarter mile to aid in selecting critical sections which may be isolated for further investigation of possible defects in design or construction.

The University of Michigan equipment is modeled after that designed and used by the California State Highway Department. Their

design was made available to Michigan by Francis N. Hveem, Materials and Research Engineer for the California State Highway Department. In addition to recording the cumulative vertical displacement, equipment is provided for measuring and recording the location of joints and cracks to provide a measure of the structural continuity of the pavement surface.

When progressive changes in the profile are to be used as a basis for measuring pavement performance, it is obvious that this trace must be very accurately measured and recorded. One of the first questions asked about the Michigan profilometer is, just what is the profile truck measuring and how accurately will it reproduce the actual pavement profile? The answer to this question depends upon the selection of a datum or reference from which the recorded vertical displacement is measured. For the purpose of explanation, two pavement profiles taken on the test course at Willow Run Airfield have been plotted for comparison in Figure 7. The profile shown as a heavy line is the trace recorded by the profilometer truck. The profile shown as a thin line has been plotted by computing the vertical displacement from elevations taken by levels at intervals of 2.5 feet along the test course. The latter profile represents the difference between the elevation at the recording wheel at the center of the truck and the average elevation of the bogie wheels at the front and back of the truck. In other words, the reference plane for vertical displacement is represented by a straight line trace drawn from the average elevation of the two sets of bogie wheels.

This computed profile is considered to be most representative of the vertical displacements recorded by the profile truck, which is actually measuring the deflection at the center of the floating base line 30 feet in length. By visual inspection, there is reasonably good agreement between the recorded profile and that constructed from the level survey. There are several other problems of measurement and instrumentation involved in

12

this simulated pavement profile which have been the subject of much additional investigation that cannot be presented in this paper. However, several examples will be given which will indicate that the equipment is capable of reproducing pavement profiles with sufficient accuracy to reflect significant differentials in the pavement surface, and still keep the equipment and its operation within practicable limits.

Before presenting several typical profiles for further discussion, it is desirable to provide a rating scale in terms of pavement roughness to serve as a basis of reference in comparing different pavement profiles and judging the significance of the roughness as an indication of pavement performance. Such a rating scale is shown in Figure 8; and, while it is still considered tentative, it has compared favorably with other procedures for rating pavement roughness with which it has been correlated. While the range of vertical displacement in inches per mile as tabulated is self-explanatory, it may be well to emphasize a few typical ranges of riding quality from exceptionally smooth to extremely rough. Cumulative vertical displacement of less than 50 inches per mile is considered to indicate an exceptionally smooth pavement; however, experience has shown that it represents a riding quality that is not at all impracticable to obtain under ordinary construction conditions. The riding quality of a pavement with a cumulative vertical displacement of 50 to 100 inches per mile would be considered good; between 100 and 150, acceptable; and, between 150 and 200, poor. A roughness index in excess of 200 inches per mile is considered to indicate extremely rough or unacceptable pavement, although there are many examples of such roads in service.

Typical Pavement Profiles

Several typical pavement profiles have been selected to illustrate pavement performance under the various conditions of service for which Michigan design standards have been

formulated. In this connection, it is necessary to supply for the record the rating system now being used by the Michigan State Highway Department to classify Michigan highways from the standpoint of adequacy to carry legal axle loads. Four levels of adequacy were selected and defined as follows:

Class 1

No seasonal restrictions. Pavement and subgrade adequate for year-round service, as represented by natural sand and gravel subgrades with superior natural drainage.

Class 2

No seasonal restrictions. Pavement designs which compensate for seasonal loss of strength, as represented by subgrades of fine-grained soils and generally inferior drainage corrected by the use of free draining sand and gravel subbases, raising grade line to improve drainage, removal of frost-heave soils.

Class 3

Spring load restrictions required. Pavement designs which do not compensate for seasonal loss of strength, as represented by fine-grained soils susceptible to frost-heaving and pumping and with inadequate drainage provisions.

Class 4

Spring load restrictions required. Pavement designs inadequate for legal axle loads at all times, as represented by older roads completely deficient and requiring continuous maintenance to provide year-round service for legal axle loads.

The real test of the value of pavement profiles and the validity of criteria that have been selected to measure pavement performance is in the practical application of this approach to actual roads in service. In the next seven figures there are examples of pavement profiles which have been previously reported in more detail,[8] but may be used here to illustrate significant variations in pavement performance and the general range of riding quality that is found on Michigan highways under Michigan service conditions. Pavement profiles, as presented in these figures, follow a definite format which will be explained at the outset with no further references being

13

required. Each graph presents approximately 450 feet of profile in the outer and inner wheel paths of the traffic lane. The profile of the outer wheel path, on the right-hand side of the lane at the edge of the pavement, is at the top of the sheet. The direction of the survey, as shown on the figures, is from right to left. In multiple lane highways with a traffic lane and passing lane, the passing lane is profiled in the same direction as the traffic lane; thus, the outer wheel path recorder is on the right-hand side of the passing lane or near the center line of the pavement, and the inner wheel path recorder is actually at the outer edge of the pavement, next to the median strip. Cut and fill is indicated at the top of the chart; the roughness index in inches per mile taken from the recorded profile is shown in the center of the sheet below each of the two profiles. In the visual examination of these profiles, it should be noted that vertical displacements are recorded full scale (1 inch = 1 inch), whereas horizontal distances are to a scale of 1 inch = 25 feet. This results in a 1:300 magnification of vertical displacements, thus exaggerating the roughness in terms of the pavement grade or slope.

In concrete pavements, joints and cracks are shown along the horizontal line at the bottom of the sheet above which the continuity ratio is given, the first figure being for the pavement as originally constructed and the second figure, on the right, being computed for the combination of cracks and joints. In evaluating data shown on the chart, it may be recalled that the continuity ratio has been defined in terms of a standard slab length of 15 feet representing a continuity ratio of unity; thus, an uncracked slab between 100 foot joints would have a continuity ratio of 6.67. From the results of previous studies, it has been assumed that the structural integrity of a concrete pavement has not been impaired until the slab length has been reduced by cracking below the standard length of 15 feet.

In the case of flexible pavements, it has been the practice in field surveys to record cracks and patching as the most visible evidence of structural discontinuities and surface deterioration that can be readily observed from the truck profilometer as the profiles are being recorded. In the case of resurfaced concrete slabs, reflected cracking is also recorded but it is recognized that it is not an accurate measure of the structural continuity of the buried slab. The major use made of such data in the case of bituminous pavements is to have a record of areas in which surface deterioration is somewhat advanced and shows the need for a field survey to more accurately record the pavement condition.

Under each of the profiles in the subsequent figures, pertinent data are given on the pavement section, including the type and thickness of various components, the subgrade classification, drainage conditions, road classification from the standpoint of adequacy, project identification, traffic count based on the 1957 surveys, which were the latest available at the time these data were recorded, and the date of the profile survey.

Figure 9 - Class 1 Flexible Pavements: In Figure 9 are shown two examples of Class 1 flexible pavements, representative of high type bituminous construction and present Michigan design standards for this type of pavement. The pavement in Figure 9-A has had only three years in service, but the subgrade, pavement components, and construction details are so closely similar to those of the pavement in Figure 9-B, with 22 years of service, that they should give comparable performance over longer periods of time. Both are on a superior type of subgrade, of sand outwash with excellent drainage characteristics. In both cases, the gravel bases were constructed first and subjected to traffic for a sufficient period of time to be thoroughly compacted before adding the bituminous wearing course. In terms of riding quality, both pavements would be rated as exceptionally smooth, indicating excellent

construction practice as well as completely adequate load carrying capacity. It may also be pointed out that subgrade stabilization, in one case with earth fill and in the other case with stabilized gravel, was employed to assist in thorough compaction of the rather loose incoherent sand before adding the gravel bases. Aside from the superior physical conditions involved in the construction of these two roads, the steps taken to obtain good compaction in all portions of the supporting foundations are important factors in the superior riding quality that has been produced and its permanence.

One other special condition should be noted in connection with the pavement in Figure 9-B, which is reported as having a service period of 22 years, even though it was resurfaced in 1956. This pavement had still retained its superior riding quality after a service period of 20 years and gave no evidence of any deficiency in load carrying capacity. However, surface abrasion and hardening of the bituminous material had produced a fragile surface which had to be either sealed or resurfaced. This, combined with the necessity for widening the road from 20 to 22 feet, led to the use of the bituminous concrete retread.

Figure 10 - Class 4 Flexible Pavements:
In Figure 10 are shown two sections of an old road built up over a period of years by county forces and later taken over by the State Highway Department and added to the trunkline system. The predominant soil type in this area is a sandy silty clay with inferior to poor drainage, a general condition which has not been compensated for by design. Consequently, the road has been rated as Class 4, inadequate for legal axle loads at all times of the year, a classification that is generally borne out by its poor performance record. However, there are exceptions; the data in Figure 10 show a sharp contrast in pavement performance on two sections of the same road just a few miles apart. The section in Figure 10-A is a

cut through an old beach ridge, on a good sand subgrade with excellent drainage. After 19 years of service, the roughness indexes of 84 in the outer wheel path and 77 in the inner wheel path would still be rated good. The section in Figure 10-B, over a sandy clay subgrade with poor drainage, has roughness indexes of 363 in the outer wheel path and 282 in the inner wheel path, outside the selected limits of the tentative rating scale. The comparison between these two sections of road provides an excellent example of the direct correlation between pavement performance and a controlling design condition, and the value of an accurately recorded pavement profile in differentiating between those sections which are giving adequate service and those which would have to be rebuilt in order to do so.

Figure 11 - The Effect of Trapped Peat:
In Figure 11, two sections of pavement have been selected which illustrate a sharp contrast in performance arising from another source. The old gravel road over a poorly drained sandy clay had always been badly affected during the spring breakup and required heavy maintenance the year around. When it was rebuilt, in accordance with present day standards, there were still some weak sections which led to its being rated as a Class 3 road, inadequate for legal axle loads during the spring breakup. Most of this contract, as indicated in Figure 11-A, gave excellent performance; after a service period of five years, it would still be rated as exceptionally smooth, with roughness indexes of 34 in the outer wheel path and 40 in the inner wheel path. In Figure 11-B is shown one of the weak sections, which had become extremely rough, with roughness indexes of 219 in the outer wheel path and 204 in the inner wheel path. The reason for this poor performance is obvious, as this section was built over an old peat deposit which had never been removed. Continued surface subsidence has been compensated for by extensive bituminous patching, a maintenance procedure which, as usual, falls

far short of providing satisfactory riding quality. The basic difficulty is permanent; as long as the unstable peat is not removed, no amount of strengthening of the pavement will correct the defect. Except for the weak sections, it would seem that this pavement should be rated as Class 2, adequate for legal axle loads the year around, because the subgrade deficiencies and poor drainage had been compensated for in the design of the rebuilt pavement.

Figure 12 - Pavement Roughness Before and After Resurfacing: In Figure 12 are shown two sections of pavement selected from US-112, one of the most heavily travelled roads in the state, in order to illustrate several aspects of pavement performance including a sharp contrast in performance, again related to soil conditions and drainage. The old road of unreinforced concrete, built in 1926, is shown as having a service period of 32 years when profiled in 1958. The section in Figure 12-A is over a fairly adequate subgrade of sandy clay loam with fair drainage, and shows relatively good performance. It was resurfaced in 1948 and, after an additional service period of 10 years, has retained its riding quality, as represented by roughness indexes of 72 in the outer wheel path and 75 in the inner wheel path. It may also be deduced that it has retained a fairly high continuity ratio, as few reflected cracks have been logged in this section.

In Figure 12-B is a weak section over a sandy clay loam subgrade, but with poor drainage. It was resurfaced in 1943 and was being resurfaced again in 1958. Before resurfacing, it had become extremely rough, with roughness indexes of 291 in the outer wheel path and 395 in the inner wheel path. This abnormal relationship between the roughness indexes of the outer and inner wheel paths was felt to be due to pumping at the center joint. This section was also profiled after the resurfacing; a new profile has been superimposed on the old one to indicate the degree of improvement result-

ing from the resurfacing. The roughness indexes of the resurfaced pavement are 66 in the outer wheel path and 65 in the inner wheel path, which would be rated very good. However, it is felt that this section will not retain its riding quality, but within a relatively short period of years will revert back to an extremely rough pavement, reflecting the basic weakness of the section which will control pavement performance until this section is rebuilt.

The Unique Value of Pavement Profiles

In the preceding examples, emphasis has been placed on the quantitative measure of riding quality by the roughness index and its correlation with controlling factors in design. The real value of an accurately recorded pavement profile goes beyond the roughness index derived from it. Such a profile is a realistic picture of the pavement itself and its physical condition resulting from a variety of influences which may have affected it. Such a profile is as individualistic as a signature, reflecting characteristics that can be fully appreciated only by examining the profile itself in whatever detail may be required to read the pavement's past history.

In Figure 13 are shown the profiles of two sections of pavement to illustrate this point. The pavement in Figure 13-A is of particular interest because it represents the effect of short, 20 foot slabs without load transfer at the joints, in what turns out to be a rather ineffective attempt to control structural continuity in terms of cracking. The subgrade was a heavy lake-bed clay, with a fill of several feet produced by side-casting from the ditches. This fill was allowed to weather for two years before the pavement was constructed, at which time an 18 to 24 inch sand subbase was added on top of the grade to protect the 9 inch plain concrete pavement from quite certain pumping. The small reduction in the continuity ratio indicates that the crack control was excellent, but the riding quality produced was still outside the

tentative roughness scale. The most interesting feature of this pavement is the characteristic sawtooth pattern that has been produced by the tilting and faulting of the short slabs, illustrating another type of valuable information to be obtained from pavement profiles. The sawtooth pattern in this pavement profile is fairly distinctive, and it seems hard to imagine how it could be produced by anything other than the tilting of the short slabs. However, there is nothing unique or individualistic about the roughness indexes of 233 in the outer wheel path and 225 in the inner wheel path.

An interesting comparison is provided in Figure 13-B, which demonstrates not only the necessity for detailed study of the profile but its intimate relation to the pavement itself which must also be taken into consideration. In this section, the profile in the outer wheel path again shows a sawtooth pattern, almost identical to that caused by the tilting of short slabs, and a roughness index of 236, as compared with 233 in the previous example. One might be tempted to conclude that this pattern of displacement would certainly be due to the tilting of short slabs, except that the inner wheel path does not follow the pattern. Furthermore, the pavement is a 9 inch reinforced concrete pavement, 99 feet between contraction joints, and there are no cracks coinciding with peaks of displacement in the profile. Further investigation indicated that this was "built-in" roughness resulting from careless form setting, with displacement at the junction between 10 foot forms and sagging of the forms between points of support. This type of built-in roughness was most apparent in the outer wheel path, but also showed in the inner wheel path which has a roughness index of 105.

Frost Displacement

One of the environmental factors which is a dominant factor in pavement performance in Michigan comes under the general heading of frost action, but is related to the annual climatic cycle, which includes much more than just the effect of freezing. This is particularly true in the case of concrete pavements, which warp, curl, and shrink under fluctuations of temperature and moisture. An investigation of the changes in the pavement profile, due to all of these factors, was given special attention in the Michigan Pavement Performance Study. For several years, repeated pavement profiles have been run on specially selected sections of all pavement classes in both southern and northern Michigan. Some of the details of these observations have been published and will only be summarized in this discussion as an example of these important factors in pavement performance.[9,10]

In Figure 14 are shown two sections of pavement which have been selected to illustrate frost displacement. In Figure 14-A is shown a Class 2 flexible pavement, adequate for legal axle loads at all times as subgrade deficiencies have been compensated for in design. Three profiles and the dates on which they were taken have been shown on each chart. The first is a profile taken in the late fall of the year, before frost penetration, when the pavement is presumably in its most stable condition. The roughness indexes are 38 in the outer wheel path and 44 in the inner wheel path, which would be rated exceptionally smooth. The second profile was taken in late winter, when frost action would be close to its maximum. This profile has been superimposed on the fall profile by matching low points, which is presumably as closely as the two profiles might be combined to indicate the location and magnitude of frost displacement, which is considerable. The roughness index in the outer wheel path is 108; that in the inner wheel path is 95. The third profile was taken in the late spring, when the subgrade and pavement structure are presumed to be approaching stability, although there may be some improvement during the succeeding summer. At any rate, the important point to note is that there is some residual roughness, as the pavement did not recover completely from the

17

winter's frost displacement. The term "frost displacement" is used to differentiate this phenomenon from deep-seated heaving, as the roughness occurring in these observations is felt to be due to moisture accumulating in the free-draining granular base and subbase. The source of this moisture is considered to be infiltration at the edge and moisture condensing under the cold pavement surface after having moved to that surface in the vapor phase. Finally, it may be noted that the cumulative roughness or loss of riding quality was observed in Class 1 and Class 2 roads, which are adequate for legal axle loads at all seasons of the year, so is not a function of load repetition but an environmental phenomenon over which design procedures may have little influence.

In Figure 14-B are shown three similar profiles on a Class 4 flexible pavement which is deficient in load carrying capacity and for that reason has become extremely rough. This is the same section of pavement discussed in Figure 10, so with respect to frost displacement it is necessary only to note that it became somewhat rougher during the period of maximum frost action but the differential was much less significant. Furthermore, the residual roughness in the spring profile was also less significant and, as a matter of fact, in the inner wheel path the recorded roughness in the spring was actually less than in the fall. Possibly traffic or other influences might have actually smoothed out some of the peaks of the displacement, resulting in a decreased roughness index. While the differentials in the roughness index were small, this and similar evidence from other profiles indicates that there may be some justification in concluding that after a flexible pavement has become extremely rough, it may reach a limit not likely to be exceeded.

Cumulative Changes in the Pavement Profile

The next two figures will be used to illustrate cumulative changes in the pavement profile, showing significant trends which may

prove useful in evaluating the performance of different pavement designs over longer periods of service. The first example shown in Figure 15, from US-31, the Muskegon - Grand Haven expressway, has been selected as typical of heavy duty bituminous pavement. This road is a four lane divided highway. The representative section selected for study was built under two contracts which, combined, are approximately six miles long, providing about 24 lane miles of pavement on which a series of profiles have been run over a period of almost four years.

Special attention was given to the construction of this pavement to insure superior riding quality, and the design anticipated that this riding quality would be preserved over a considerable period of years. The first profiles, run in the fall of 1958, were exceptionally smooth, with roughness indexes varying from 20 to 40 inches per mile. In March, 1959, at the peak of the frost action period, there were certain sections, as shown in Figure 15, that developed a roughness increase that had not been anticipated. Special study revealed that these sections were near culverts or at points of high water table which, during this winter of unusually deep frost penetration, were subjected to some deep-seated heaving. This condition did not occur in March of 1960 or 1961, but was repeated in 1962, a winter when there was another severe frost penetration.

Examination of the series of profiles in Figure 15 indicates the reproducibility of such pavement profiles by the truck-mounted profilometer and gives some indication of the reliability of this equipment within practical limits. At the right-hand side of the figure are shown survey dates and the roughness index in inches per mile for the particular section shown in Figure 15, which is considered representative of the entire mileage involved in this special study. In addition to the cycle of roughness changes, particularly during severe winters, there is a slow upward trend in the roughness index, indicating some residual

18

displacement accumulating from either frost action or load repetition.

In order to obtain a clearer idea of the cumulative change in roughness on these two contracts, the data have been presented in Figure 16 in terms of the average roughness index for the total mileage in each of the two contracts, showing also the differential between the traffic lane and the passing lane. The annual cycle in roughness can be seen in all lanes, with the high points generally being in March, at the peak of the frost period, and the low points during the summer and fall. Inasmuch as the low points represent a stable profile and the points of maximum frost action are not a constant reference, the rates of cumulative increase in roughness have been taken as the slopes of these curves through the low points. These slopes are summarized in Table 2, in which maximum and minimum rates of change have been determined. The maximum rates have been determined by two low points which may be questioned. The first is the low point in September, 1958, before the pavement had been opened to traffic or was exposed to its first winter cycle; this first cycle has been found to produce an abnormal increase in roughness as compared to subsequent years. The second low point which is questioned is in March, 1960, when a normal winter would have produced a high point rather than a low point. During the spring of 1960, there was some difficulty with calibration of the equipment; also, that winter was one in which there was very little frost penetration until after the first of March.

The minimum rates are based on low points in the summer and fall of 1959 and low points in the fall of 1961, which also correlate well with the reading obtained in the spring of 1962. Maximum rates of increasing roughness vary from 3 to 4.5 inches per mile per year; minimum rates vary from 1 to 2.5 inches per mile per year. In terms of averages, the rates of increase in roughness vary from approximately 2 to 3.5 inches per mile per year. It is also interesting to note, in connection with these data, that there is a measurable differential in rates of increase in roughness between the traffic and the passing lanes, which would indicate that there is some response to the fairly heavy traffic carried on this route, a deduction which is not inconsistent with some changes in the condition of the pavement surface which have been observed. In concluding the discussion of the data shown in Figures 15 and 16, it should be noted that the period of time involved is very short and that this is only one pavement design although it does cover some 24 lane miles of pavement. The trends shown over this period of time must be considered only indications and not a sufficient basis from which to draw any final conclusions.

CUMULATIVE CHANGES IN ROUGHNESS
WITH AGE IN SERVICE

One of the major questions to be answered in pavement design is the long range performance of pavements and the anticipated useful life before a pavement must be replaced. In the more than 9000 miles of pavement profile, there is a considerable mileage of older roads which have been in service up to a maximum of 35 years. In the surveys to date, more attention has been given to concrete pavements than to the flexible bituminous pavements; thus, only scattered data are available on the older pavements of the flexible type. However, what is available will be presented to at least indicate trends.

In the next three figures, available data on cumulative changes in roughness of pavements having longer periods of service will be presented. Three main types of pavement will be included in these figures: first, Class 1 rigid pavement (Portland cement concrete); second, Class 1 recapped concrete pavement (bituminous resurfacing); and, third, Class 1 flexible pavement (asphaltic concrete on aggregate bases). In all cases the data will be for the traffic lane; no more than passing comment

will be made with regard to differentials in performance between the passing lane and the traffic lane.

In all three figures, the method of presenting the data is the same: years in service is the ordinate; the abscissa is the roughness index in inches of vertical displacement per mile. Each plotted point on the chart represents the average roughness index in one wheel path of a pavement construction contract. When only one lane of a contract has been surveyed, there will be two plotted points for that contract; when both traffic lanes have been surveyed, there will be four such points for that contract, as illustrated on certain contracts identified on these figures.

In general, it has been found that the outer wheel path is rougher than the inner wheel path due, presumably, to greater exposure or weakness at the free edge of the slab. There are exceptions to this which can usually be associated with some specific pavement condition. Special studies have been made of variations of this nature but will not be discussed in detail as part of this paper.

A rather elementary but readily understandable procedure has been evolved for evaluating the cumulative changes in pavement with age in service. The first step is to compute the average roughness index for each five year period as the center of gravity of all observations in that period. These averages are shown as large circles on the three charts under consideration and are used to determine an average slope or rate of increase in roughness with years in service. Two limiting lines are drawn parallel to this average slope, establishing a band of normal behavior within which approximately 85 per cent of the plotted points are included. The width of this band and the average slope have, as a matter of fact, been determined by trial and error, with some modifications made to take care of unusual variations or abnormal results.

When the number of projects from which data are available is very limited, abnormal

results from one project exert undue influence on the average and would distort the trend that might be most representative of long time service behavior. In these cases, as a second trial, results falling above or below the normal band of behavior that has first been determined are not used in the average, which is then computed on the basis of the points within the band of normal behavior. The scattered data in Figure 19 for Class 1 flexible pavements more than 10 years old is an example in which it may be necessary to use this procedure to get any valid indication of what the trend may be in later years.

Class 1 Rigid Pavements

The data for Class 1 rigid pavements are shown in Figure 17, and constitute the most comprehensive set of data available, both from the standpoint of years in service and the number of contracts and mileage covered. Most of these data have been previously presented and are used in this paper as an example of the interpretation of roughness measurements and to serve as a basis for generalizing less comprehensive sets of data on other types of pavement. The data shown in Figure 17 have been taken from 139 construction contracts and 664 lane miles of pavement.

For the first 25 years the averages fall quite consistently along the line with the slope of 4.5 inches per mile per year, which represents the rate at which these pavements lose riding quality with age in service. The boundaries of the band of normal behavior, drawn parallel to this line to include 85 per cent of the roughness data, have intercepts on the horizontal axis ranging from a roughness index of 30 to a roughness index of 105, or a range of variation in the roughness index of 75 inches per mile. There is a discontinuity indicated in Figure 17 at an age of 25 years, which is accounted for by the fact that many pavements more than 25 years old have been reconstructed, recapped, or changed in classification. With few exceptions, only those projects exhibiting superior performance are still

20

in service and have been included in the pro-
file surveys. A review to trace the history
of all concrete pavement built before 1936
has been only partially completed; thus far,
indications are that very few of these older
concrete pavements remain in service after
having become very rough.

At this point in the discussion, the most
important interpretation of the data shown in
Figure 17 is that these pavements have suf-
fered a cumulative or continuous increase in
roughness at an average rate of between 4 and
5 inches per mile per year. When it is con-
sidered that Class 1 pavements are those that
have been rated as adequate or more than ade-
quate for legal axle loads at all times of the
year, it must be recognized that load carrying
capacity is not the controlling factor in this
progressive loss of riding quality. From this
it follows that the predominant causes of this
type of pavement deterioration are environmen-
tal factors, mainly associated with seasonal
cycles and fluctuations of moisture and tem-
perature combined with frost action.

In correlating pavement performance with
design, the most revealing data in Figure 17
and, in fact, on all three of the charts show-
ing cumulative changes, are provided by those
projects which show abnormal behavior, falling
outside the band of normal behavior. A number
of projects, whose performance is either supe-
rior or very poor, have been identified in
Figure 17, and the factors leading to their
abnormal performance have been discussed in
some detail in a previous report.[11] For the
present discussion, only two of the most il-
luminating examples will be presented for il-
lustration.

The first such example is given by two
contrasting projects, US-31 (3-C1) and US-31
(5-C1). The first project, after 33 years of
service, shows exceptionally good performance,
with a roughness index falling well below the
band of normal behavior and a riding quality
still rated fair to acceptable. The second
project, after 34 years of service, is rated
extremely rough, with the roughness index

falling near the upper limits of the band of
normal behavior. Both were rated as Class 1
pavements on the basis of an area soil survey
which identified the soil series as Plainfield
sand, a superior subgrade with high internal
stability and excellent drainage. These two
projects, within several miles of each other,
were built by the same contractor and have
closely comparable traffic. The soil classi-
fication of Plainfield sand is correct for the
project showing superior performance but in-
correct for the project which has become ex-
tremely rough. The latter contract is located
at a transition in soil types, with the major
portion of this project in an area of silty
clay loam with inferior drainage conditions.
This part of the pavement should have been
rated as Class 3 or Class 4. The transition
in soil series and the marked changes in pave-
ment performance are accurately identified on
the recorded pavement profile.

Another interesting example is Project
M-21 (35-C10): most of the roughness index
values on this contract fall in the lower
range of the band of normal behavior, showing
acceptable performance over a service period
of 19 years; but, there was one exception to
this statement. This was a single observation
of a roughness index of 225 along the outer
edge of a quarter mile section of pavement
widening on this contract. A field investiga-
tion of this section revealed that a storm
sewer had been laid along the edge of the
pavement; backfill settlement undoubtedly pro-
duced the high roughness index.

Class 1 Recapped Concrete Pavement

The roughness index data shown in Figure
18 have been taken from 118 construction con-
tracts covering 461 lane miles of pavement.
Surveys have been conducted over a period of
four years, from 1958 through 1961. There are
no projects more than 20 years old, which more
or less establishes the period over which
there has been extensive bituminous resurfac-
ing of old concrete pavements. Determination
of the band of normal behavior and the slope

of the lines which define the rate of increase
in the roughness index followed the previously
described procedure, within practical limits.
Actually, the average slope of these lines
would have been 4.85 inches per mile per year,
or slightly greater than in the case of the
rigid pavements in Figure 17, and the inter-
sections of the lines with the horizontal axis
would have been slightly different. However,
it was obvious that in the treatment of data
of this kind such differences were negligible;
to report a differential that could be inter-
preted as significant would hardly be justi-
fied by the available data.

Consequently, the slopes and limits found
for rigid pavements were superimposed on the
data in Figure 18 and represent no more than a
negligible departure from the precise average.
From this it is concluded that in recapped
concrete pavements, the cumulative change in
roughness with age in service is substantially
the same as in the concrete pavements them-
selves, which are now serving as the base
course for the bituminous surface. Recalling
that the roughness changes under discussion
are the effect of curling, warping, and
shrinkage of the concrete pavement under the
influence of environment, it would be deduced
that this same behavior is being reflected
through the bituminous surface.

There are a number of projects in Figure
18 which fall outside the band of normal be-
havior and have been identified by contract
number for further investigation of the ab-
normal behavior which they have shown. While
complete investigation including field check-
ing is still in progress, there is some sig-
nificant information from Figure 18 which
throws light on important factors relating
pavement design to performance. While there
are exceptions, in which the influence of
other conditions enters, a review of all the
projects under discussion brings out the fol-
lowing fact: a large majority of the projects
having the higher roughness index values are
plain concrete, while a corresponding majority
of the projects falling in the lower range of

roughness index are reinforced. Reinforced
and unreinforced pavements have been indicated
on the figure.

Before leaving the discussion of resur-
faced concrete pavements, it should be pointed
out that, in Figure 18, the years in service
are figured from the date of the last resur-
facing. Studies have also been made of the
relationship between cumulative changes in the
roughness index and the date of the original
concrete pavement construction. Some signifi-
cant relationships have been brought out by
this study, but space will not permit presen-
tation as part of the current discussion.

Class 1 Flexible Pavements

Data available on the cumulative changes
in the roughness index of flexible pavements,
from 50 contracts and 370 lane miles, are
shown in Figure 19. Determination of the band
of normal behavior and the slope of the lines
showing the rate of change in the roughness
index followed the procedure previously used.
Again, the evidence, particularly for service
periods greater than five or six years, is too
scattered to justify selection of a rate of
change any different from that already deter-
mined for the other pavement types that have
been discussed. It may be noted that a rate
of change of 4.5 inches per mile per year
seems reasonable and that the parallel bounda-
ries for the band of normal behavior include
the scattered points for the older projects
quite satisfactorily.

There are several general comments that
should be made with respect to this type of
road construction. The older projects were
built during a period when bituminous pave-
ments in Michigan were thought of as low cost
road construction. As a consequence, both de-
sign and construction requirements were much
less exacting than those practiced in recent
years. Consequently, a wider range of behav-
ior must be expected in projects more than 15
or 20 years old. It should also be noted that
a much larger number of contracts and greater
mileage of roads should be included in the

22

data before the long range performance can be considered to be anything more than an indicated trend. On the other hand, the more recent projects, no more than six years old, show definite evidence of closer control of design and construction than in the older projects. The points are more closely grouped and the band of normal behavior is narrower than in any of the other sets of data. In the case of initial construction, the range of roughness index varying from 20 to 80 inches per mile is evidence of good construction control.

A number of the projects shown in Figure 19 have been identified by contract number and are under investigation to determine the cause of the abnormal behavior which their performance indicates. Field investigation of these projects is a necessary step in further analysis, and these data are not yet available. General information on construction conditions indicates that the contrast between abnormally good performance and that which is abnormally poor is closely related to soil conditions and to design and construction practices which, as already noted in the case of the older roads, left much to be desired. In this connection, perhaps the outstanding feature of the data on flexible pavements is the rather sharp contrast between the wider range of pavement performance on the older projects and the better control evidenced in the more recent construction. In concluding the discussion of the data available on flexible pavement construction, it should be stated that there is not sufficient evidence now available to establish rates or range of cumulative change with age in service of these pavements that might differ from the findings for the other pavement types previously discussed in this paper.

CORRELATION OF
MICHIGAN PAVEMENT PERFORMANCE STUDY
WITH AASHO ROAD TEST RESULTS

The great interest in the last 10 or 15

years in evaluating pavement performance as related to design has led many highway agencies to undertake research programs directed to this objective. The AASHO Road Test is by far the most elaborate and extensive project of this kind conducted to date. The results of this test have recently been presented at a special three day meeting in St. Louis, May 16-18, 1962. With the AASHO Road Test data made available for public dissemination and analysis, it is now possible to make comparisons with the results of other research projects and to work out useful correlations.

One of the objectives of the Michigan Pavement Performance Study has been to establish a relationship between pavement performance criteria developed from condition surveys on existing roads and AASHO pavement performance concepts. In February, 1959, pavement profiles were run on Test Loops 3, 4, 5, and 6 at the AASHO Road Test with both the Michigan and the Road Test profilometers. The theoretical relationship between the cumulative vertical displacement and slope variance based on an assumed sine wave profile was worked out by Irick and is given by the following equation:

$$RI = 57 \sqrt{SV}$$

in which

RI = Michigan Roughness Index in inches per mile

SV = AASHO Slope Variance (an abstract number)

This relationship is shown graphically in Figure 20 and is a straight line when the roughness index is plotted as the ordinate and the square root of the slope variance is plotted as the abscissa. Experimental data from a number of projects have been plotted to check this relationship or, rather, to test the validity of the assumptions on which it is based. Measurements made by both profilometers are conditioned by the base line length from which either vertical displacement or angular displacement is measured and the positioning of this base line with respect to the pavement grade.

Measured vertical displacement is also

23

affected by the length of the up and down displacements or wave lengths in the pavement profile. Both profilometers measure with respect to a floating base line; it may be shown that if either encountered a regular sine wave of critical length, the result might be seriously in error. However, if the profile contains random wave lengths of sufficient number and variety, errors of this nature are compensating rather than cumulative and the reported roughness index or slope variance may be truly representative of the pavement's characteristics.

Data from seven projects are plotted in Figure 20. They indicate that the theoretical equation based on a sine wave profile is reasonably representative of actual pavements. This means that the length of pavement is sufficient and the character of the displacement is such as to provide randomization of cyclic deviations or wave lengths. The points shown as triangles are from two test loops of the AASHO Road Test for which the average roughness index has been plotted against the average slope variance. Each factor has been measured by the profilometer designed for that specific type of observation. The plotted points are the averages of observations from the 24 test sections, 100 feet in length, which were available. Further analysis of these data is planned as soon as time will permit, now that detailed results can be made available. It may be noted that comparison of the short test sections showed little evidence of correlation in a preliminary study. This result is consistent with the above comments on randomization. It also follows variations in which the average index value changes with the size of the sample, as pointed out in the discussion of the continuity ratio of the center taxiway at Willow Run Airfield earlier in this paper.

Two other points, shown as open triangles, are far removed from the straight line shown in Figure 20. These are also measured values, taken, however, from a selected test section on US-20 in Indiana where a

number of agencies tested their pavement evaluation equipment. Further analysis of these data is required, but it is known that this pavement had some unusual characteristics which may have prejudiced the data from one or the other of the profilometers.

The points shown as circles are from Michigan projects selected to cover a rather wide range of roughness, and limited in number by the short time available since the AASHO results were released. The roughness index (RI) was measured by the Michigan profilometer; the slope variance (SV) was computed from level measurements made at intervals of one foot.

Present Serviceability Index (PSI)

The next step in relating results from the Michigan Pavement Performance Study to AASHO criteria is the computation of the present serviceability index (PSI). This has been done for 12 Michigan pavement sections, six of rigid pavement and six of flexible pavement. An example of the computation for one of the flexible pavements is given in Appendix A; the results for all 12 sections have been plotted in Figure 21.

On these projects, the roughness index was measured by the Michigan profilometer and the slope variance was computed from level measurements made on all sections of flexible pavement and three sections of rigid pavement. The other three sections of rigid pavement are projects for which the slope variance had been obtained by the Chloe profilometer. The slope variance data were not available to the writer, but the present serviceability indexes (PSI) computed for these projects were obtained. While correlation of many more Michigan projects would be desirable, time would not permit; enough examples have been presented to demonstrate that the Michigan roughness index and continuity ratio can be readily translated into terms of the AASHO serviceability index.

In the computations in Appendix A, there are two observations that appear to have some

importance from a practical point of view. In the example, the influence of rutting and of cracking and patching appears to be quite negligible in changing the serviceability index. It may be deduced from this that pavement deterioration and rutting would have to be quite advanced before they would exert a controlling influence on the serviceability index. However, from the standpoint of timely maintenance or detection of a structural weakness associated with rutting, such evidence of potential loss of serviceability is much more important than the formula indicates. This suggests that these aspects of pavement performance should be treated at their face value as an independent maintenance control, even though they lose importance when combined in a single number which is largely controlled by pavement roughness.

The second comment has to do with converting the roughness index (RI) to slope variance (SV) by use of the relationship shown in Figure 20, rather than computing the slope variance from level readings. Such an alternate computation, as shown in Appendix A, changes the serviceability index (PSI) from 3.0 to 3.06. This, too, suggests that the Michigan roughness index can be used in such a computation with negligible effect in establishing the serviceability level of the pavement at any given point in its useful life. If this is a reasonable deduction, the cumulative change in roughness with age in service presented in Figures 17, 18, and 19 takes on added significance. The point is that these changes are associated with environmental factors which provide the only reliable basis for predicting future performance. This basis for forecasting future behavior can only be established by observing the reaction of existing pavements to years of service. This normal change in roughness can be converted into a future serviceability index and a limit selected to establish the estimated serviceable life of the pavement in its specific environment.

CONCLUSION

Pavement condition surveys of existing roads have been used for many years in Michigan as the basic control for pavement design. Correlation of performance with design has shown the dominant influence of soil conditions and environment. As a result, Michigan design has been developed primarily to produce pavements adjusted to their environment. In general, these pavements are then capable of carrying legal axle loads at all seasons without damage due to load repetition. Michigan pavement design may then be characterized as the design of the foundation of the pavement; the pavement itself can then be of standard design of nominal thickness.

In recent years attention has been given to improving procedures for conducting condition surveys, more precise correlation between performance and design, and developing quantitative measures of pavement performance. The primary objectives of these efforts were to identify controlling factors in design and to evaluate the manner in which a.d the extent to which they contribute to performance. Another objective was to evaluate serviceability and establish standards for maintaining pavements at all times in a good and serviceable condition to insure their efficient operation. Because of the great interest in the AASHO Road Test, a further objective, which is of current importance, is to establish the relationship between pavement performance developed from condition surveys on existing roads and the AASHO pavement performance concepts.

In the past fifteen years, the study of pavement performance has been a cooperative program in which the University of Michigan and the Michigan State Highway Department were the principals. A number of other agencies have entered into the program as sponsors and made substantial contributions to it. From these studies, two basic quantities have been selected to measure pavement performance. First, a continuity ratio has been defined

which expresses in numerical terms the structural continuity of a pavement, the loss of which and the rate at which this loss takes place gauge the deterioration of the pavement. Second, the roughness index, measured in inches per mile of cumulative vertical displacement, expresses the riding quality of the pavement at any given time.

Early studies at Willow Run Airfield and a five year investigation of highway pavements to explore the value of steel reinforcement showed that the continuity ratio and roughness index were promising measures of pavement performance. The Michigan Pavement Performance Study was then organized; in the period from 1957 through 1962, more than 9000 lane miles of pavement profile have been accumulated. Typical examples given in this paper demonstrate the close correlation of the roughness index and continuity ratio with design factors which control pavement performance. Furthermore, it has been found that an accurate profile of the pavement surface has unique value as a record of the physical condition of the pavement, serving as a key to its past history and its ability to sustain the forces and exposure to which it has been subjected.

A number of more specific conclusions may be listed:

1. All Michigan pavements investigated suffer a cumulative increase in roughness which, based on present data, occurs at a rate of some 4 to 5 inches per mile per year. It has been found that this change is due to the effects of the environmental factors, climate and soil conditions, and represents normal behavior of pavements capable of carrying legal axle loads, and more, without damage to the pavement structure.

2. Bands of normal behavior were established which bracket 85 per cent of the data, excluding those projects showing abnormal performance. This band, which varies in width from 60 to 75 inches per mile is now presumed to be a range within which riding quality can be controlled by current methods of design and construction.

3. While there is some evidence that the range of controlled riding quality may vary between different pavement types, there is insufficient data now available on some pavement types to establish such differentials.

4. The performance of old concrete pavement recapped with an asphaltic surface is so close to that of the original concrete pavement itself that it seems reasonable to conclude that the distortion of the concrete slab due to fluctuations of moisture, temperature, and related environmental factors is reflected through the protecting asphaltic surface with little or no change.

5. Early studies of reinforced and unreinforced concrete pavement showed that among the projects surveyed, reinforced pavements were measurably smoother and had measurably less cracking than did the unreinforced pavements. Old concrete pavements, both reinforced and unreinforced, resurfaced with asphaltic concrete were surveyed; the results are reported in this paper. The performance of these pavements, as shown in Figure 18, gives evidence of a strong correlation between reinforcement and improvement in riding quality.

6. A comparison was made between the Michigan roughness index and continuity ratio and the pavement performance concepts from the AASHO Road Test. It was found that the Michigan data and performance criteria can be readily converted to the AASHO serviceability index. This means that the pavement condition surveys of more than 9000 lane miles of pavement can be so converted and used to establish the serviceability level of these pavements in terms of a standard which the AASHO Road Test has proposed. It is pertinent to note that the pavement profiles of these 9000 lane miles provide a great volume of basic design information revealed by the physical condition of these pavements in their natural environment. This information includes design conditions to be met and deficiencies to be corrected to insure that the future serviceability of these pavements will meet whatever standards may be adopted.

TABLE 1

PERCENTAGE OF CRACKED SLABS

1950 SURVEY

Type of Crack	Year of Survey	Date of Construction	
		1941	1943
Transverse	1946	4	35
	1950	10	85
Longitudinal	1946	Negligible	15
	1950	Negligible	25
Diagonal	1946	Negligible	2
	1950	Negligible	4

TABLE 2

RATE OF INCREASE IN ROUGHNESS INDEX

US-31, MUSKEGON - GRAND HAVEN EXPRESSWAY

CLASS 1 FLEXIBLE PAVEMENT

Contract	Lane	Maximum Rate Inches per Mile per Year	Minimum Rate Inches per Mile per Year
74-C3	Traffic	4.4	2.2
	Passing	3.0	1.0
16-C2	Traffic	3.9	2.3
	Passing	3.2	1.7

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COMPUTATION OF AASHO SERVICEABILITY INDEX
FROM MICHIGAN PAVEMENT PERFORMANCE DATA

COMPUTATION OF PRESENT SERVICEABILITY INDEX (PSI)

Survey Data

Location: M-50, near Charlotte, Michigan

Lane: Eastbound

Length: 500 feet

Pavement: 10 foot wide flexible pavement

Area of Cracking = 388 square feet

Area of Patching = 180 square feet

Total = 568 square feet

Michigan Roughness Index: RWP = 180
LWP = 126

AASHO Road Test PSI Formula   (Refer to Eq. 11, Page 23 of HRB Special Report 61E, 1962)

$$PSI = 5.03 - 1.91 \log (1 + \overline{SV}) - 0.01 \sqrt{C + P} - 1.38 \, \overline{RD}^2$$

in which

$\overline{SV}$ = The mean of the slope variance in the two wheelpaths from the profile. (An abstract number)

C = Cracks, in square feet per 1000 square feet of pavement

P = Patches, in square feet per 1000 square feet of pavement

$\overline{RD}$ = A measure of rutting depth in the wheelpaths, in inches

Slope Variance ($\overline{SV}$)   (Refer to Eq. 1, Page 14 of HRB Special Report 61E, 1962)

$$\overline{SV} = \frac{\Sigma Y^2 - \frac{1}{n} (\Sigma Y)^2}{n - 1}$$

in which

Y = The difference between two elevations of pavement surface, one foot apart.

n = Number of level readings

ΣY^2:   RWP = 5951×10^{-6}
              LWP = 5016×10^{-6}

Average ΣY^2 = 5484×10^{-6}

ΣY: RWP = 824×10^{-3}
            LWP = 878×10^{-3}

$(\Sigma Y)^2$:   RWP $= 678,976 \times 10^{-6}$
LWP $= 770,884 \times 10^{-6}$

Average $(\Sigma Y)^2 = 724,930 \times 10^{-6}$

$$\overline{SV} \, [\times 10^6] = \frac{5484 - \frac{1}{500}(724,930)}{499} = 8.09$$

$\log(1 + \overline{SV}) = \log(1 + 8.09) = 0.959$

Cracking and Patching (C + P)

$$C + P = \frac{568}{500} = 113$$

$$\sqrt{C + P} = \sqrt{113} = 10.6$$

Rutting Depth ($\overline{RD}$)

$$\overline{RD} = 0.261$$

$$\overline{RD}^2 = 0.068$$

Present Serviceability Index (PSI)

$PSI = 5.03 - 1.91 \times 0.959 - 0.01 \times 10.6 - 1.38 \times 0.068$

$\quad = 5.03 - 1.83 - 0.11 - 0.09$

$\quad = 3.00$

CONVERTING ROUGHNESS INDEX TO SERVICEABILITY INDEX

$RI = 57 \sqrt{\overline{SV}}$

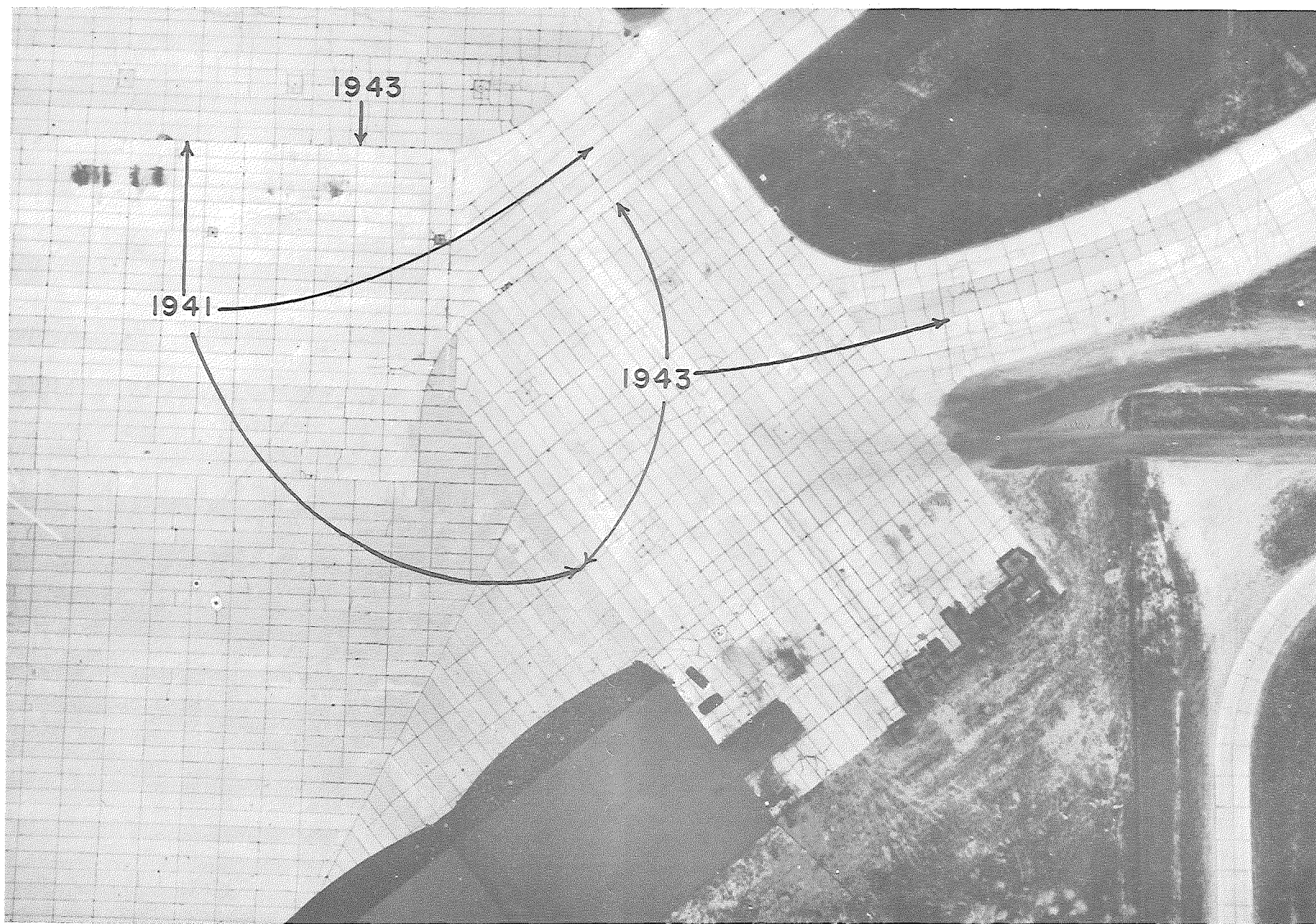$\overline{SV}$:   RWP $= (\frac{180}{57})^2 = 3.16^2 = 10.0$

$\quad\quad$ LWP $= (\frac{126}{57})^2 = 2.21^2 = 4.88$

Average $\overline{SV} = 7.44$

$PSI = 5.03 - 1.91 \log(1 + 7.44) - 0.01 \sqrt{113} - 1.38 \times 0.261^2$

$\quad = 5.03 - 1.77 - 0.11 - 0.09$

$\quad = 3.06$

1943

1941

1943

SOUTH END OF MAIN WEST APRON – 1946
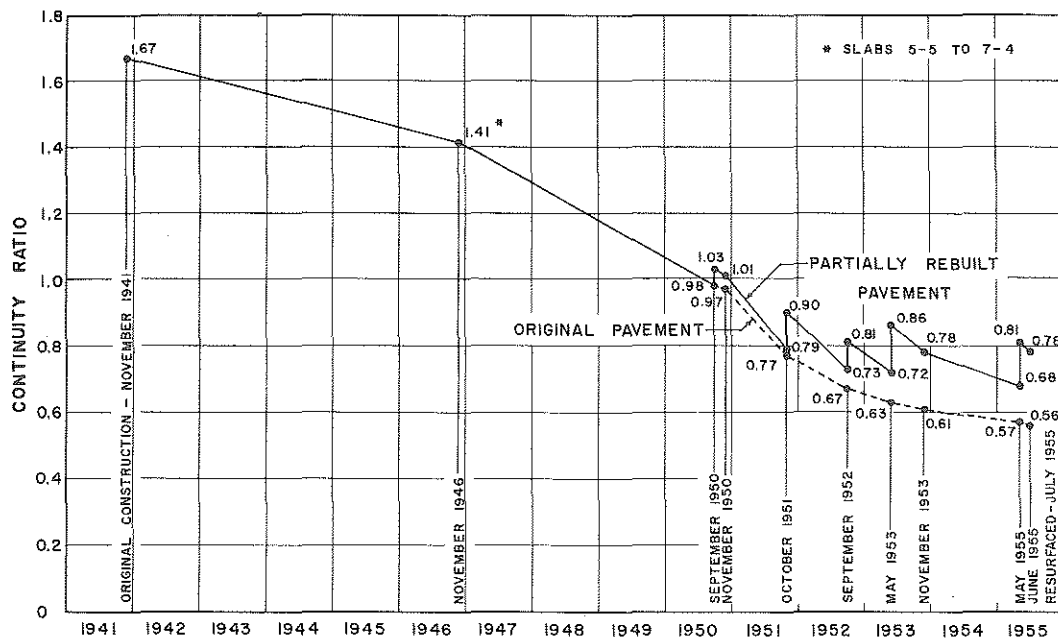
WILLOW RUN AIRFIELD

FIG. I
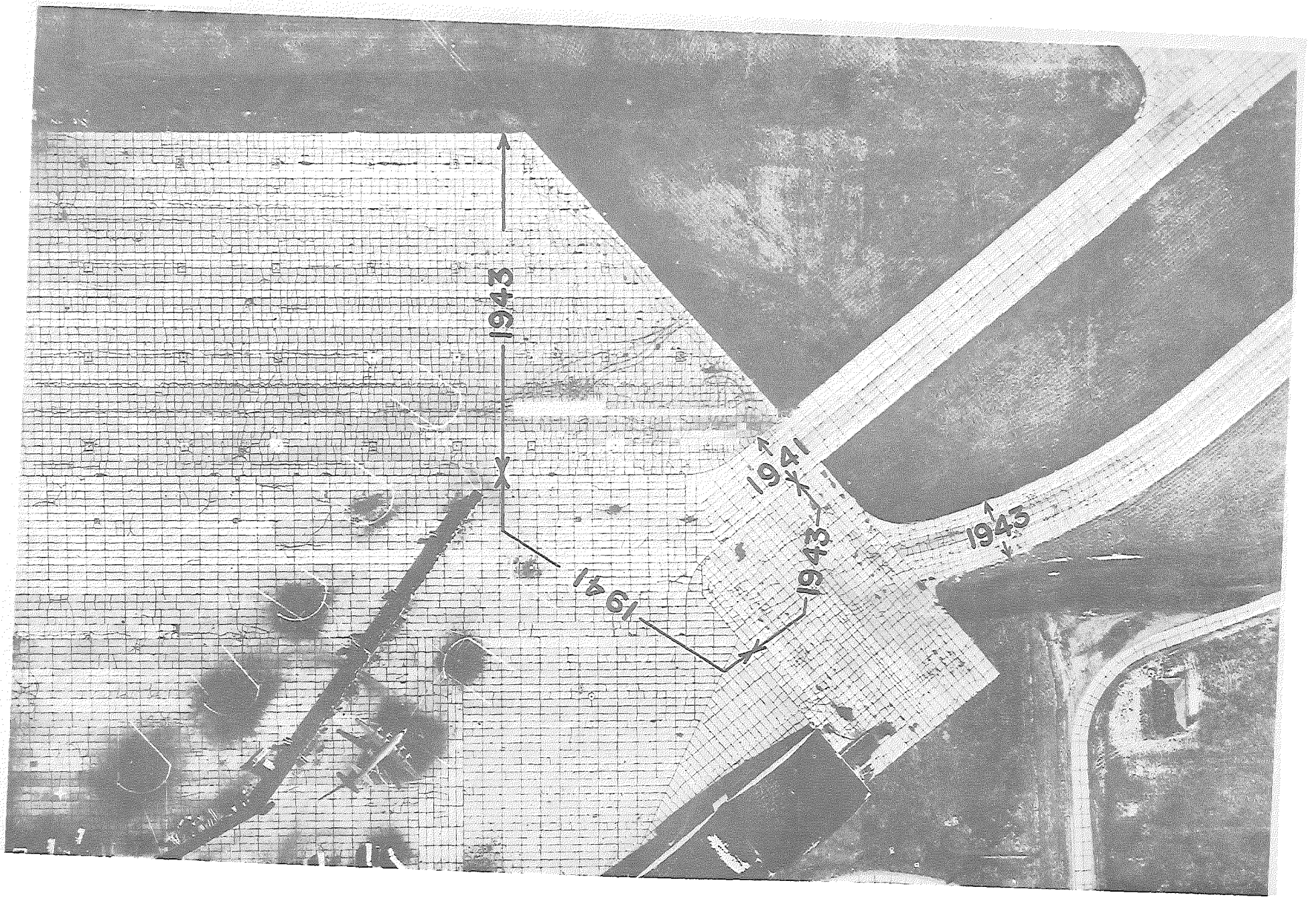
SOUTH END OF MAIN WEST APRON - 1950
WILLOW RUN AIRFIELD

FIG. 2

CHANGE IN CONTINUITY RATIO

CENTER TAXIWAY B (SLABS 5-5 TO 8-6)

WILLOW RUN AIRFIELD   FIG. 3
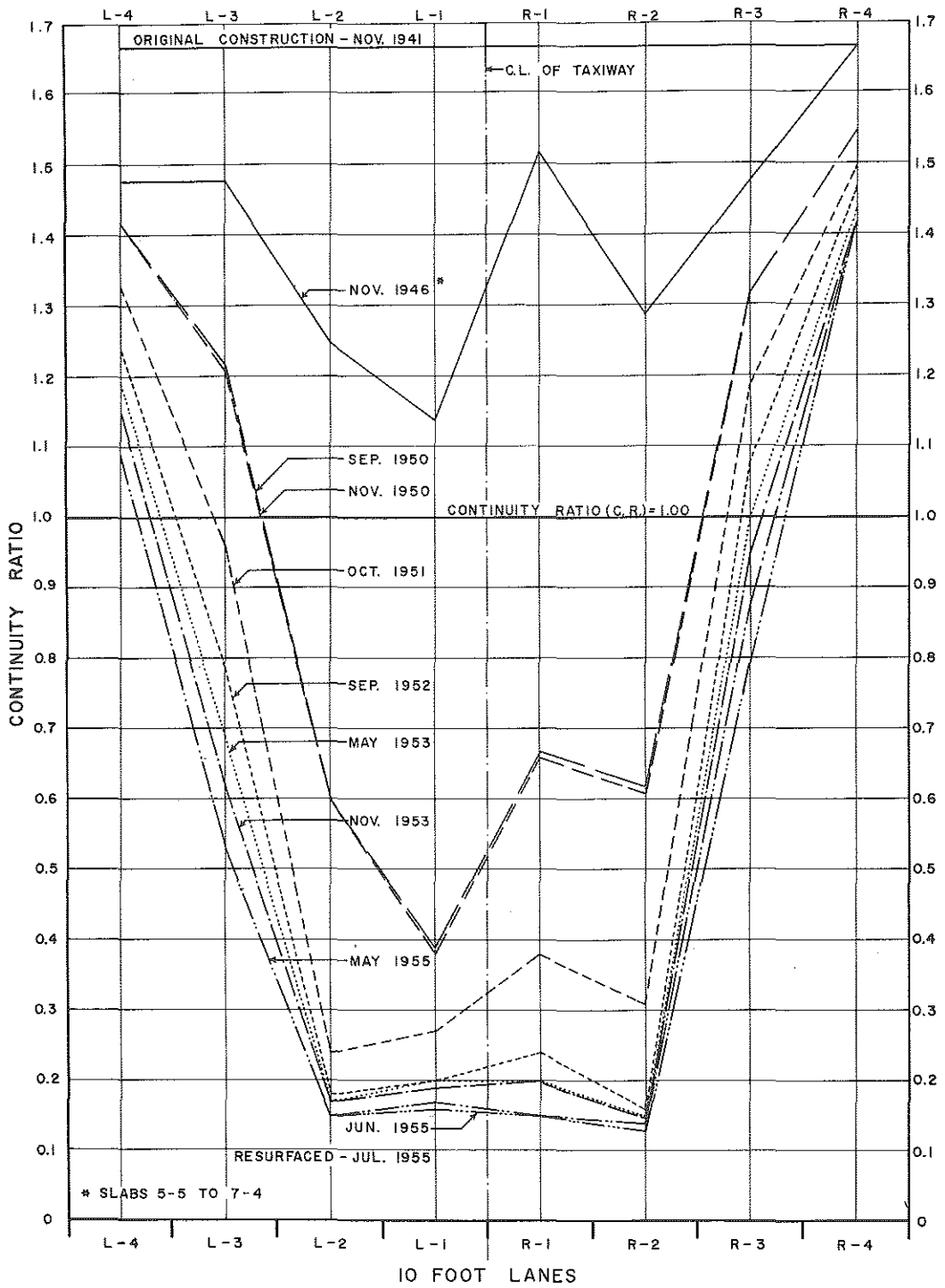
CHANGE IN CONTINUITY RATIO BY LANES

CENTER TAXIWAY-B-ORIGINAL PAVEMENT (SLABS 5-5 TO 8-6)

WILLOW RUN AIRFIELD   FIG. 4
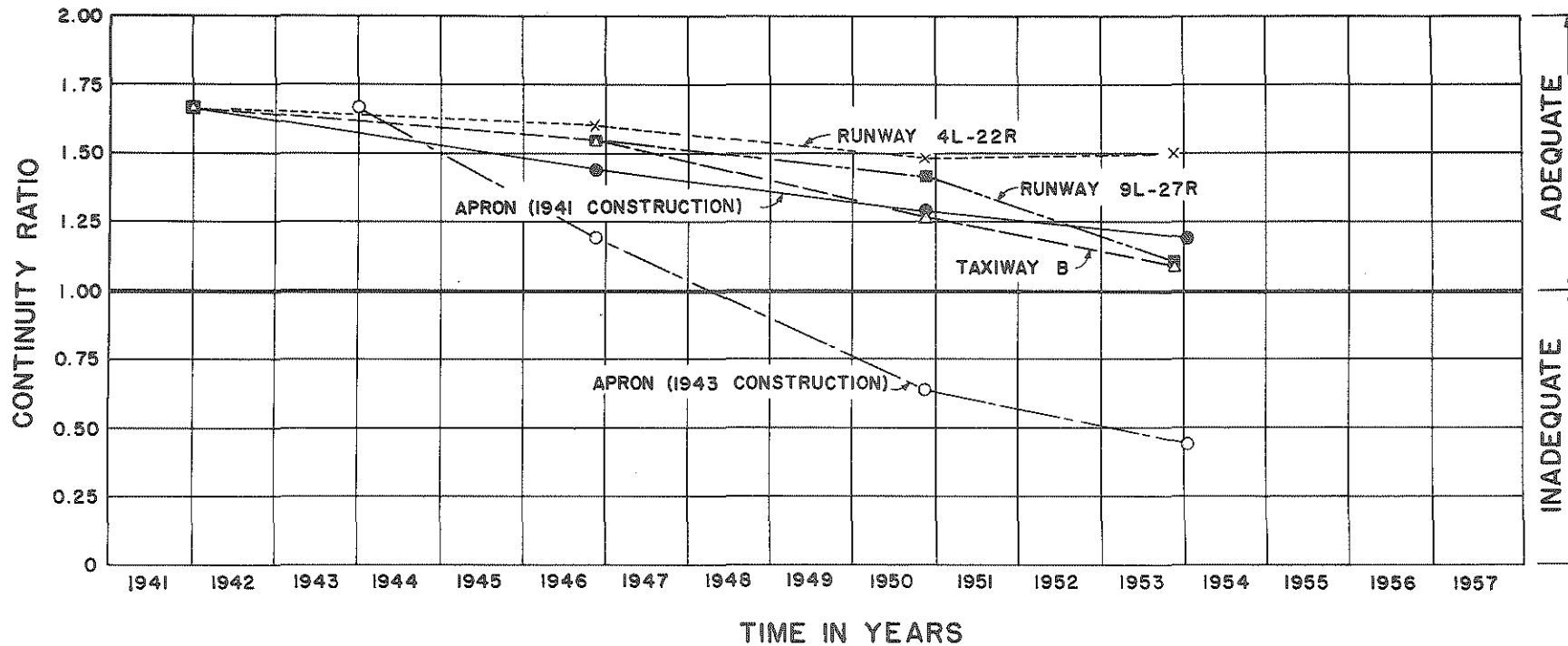
CHANGE IN CONTINUITY RATIO – FIELD AVERAGE FOR TYPICAL PAVEMENT
WILLOW RUN AIRFIELD

FIG. 5

MICHIGAN TRUCK-MOUNTED PROFILOMETER

FIG. 6

RI = 133 (PROFILOMETER)   RI = 146 (LEVELS) |——100'——|

FRONT BOGIES   BASE LINE   REAR BOGIES
PAVEMENT SURFACE   RECORDING WHEEL   Δ —
Δ +
Δ = 0
15'   15'
30'

〜 TRACE RECORDED BY PROFILOMETER
〜 TRACE COMPUTED FROM LEVELS

COMPARISON BETWEEN RECORDED AND COMPUTED PROFILES
ON TEST COURSE
WILLOW RUN AIRFIELD
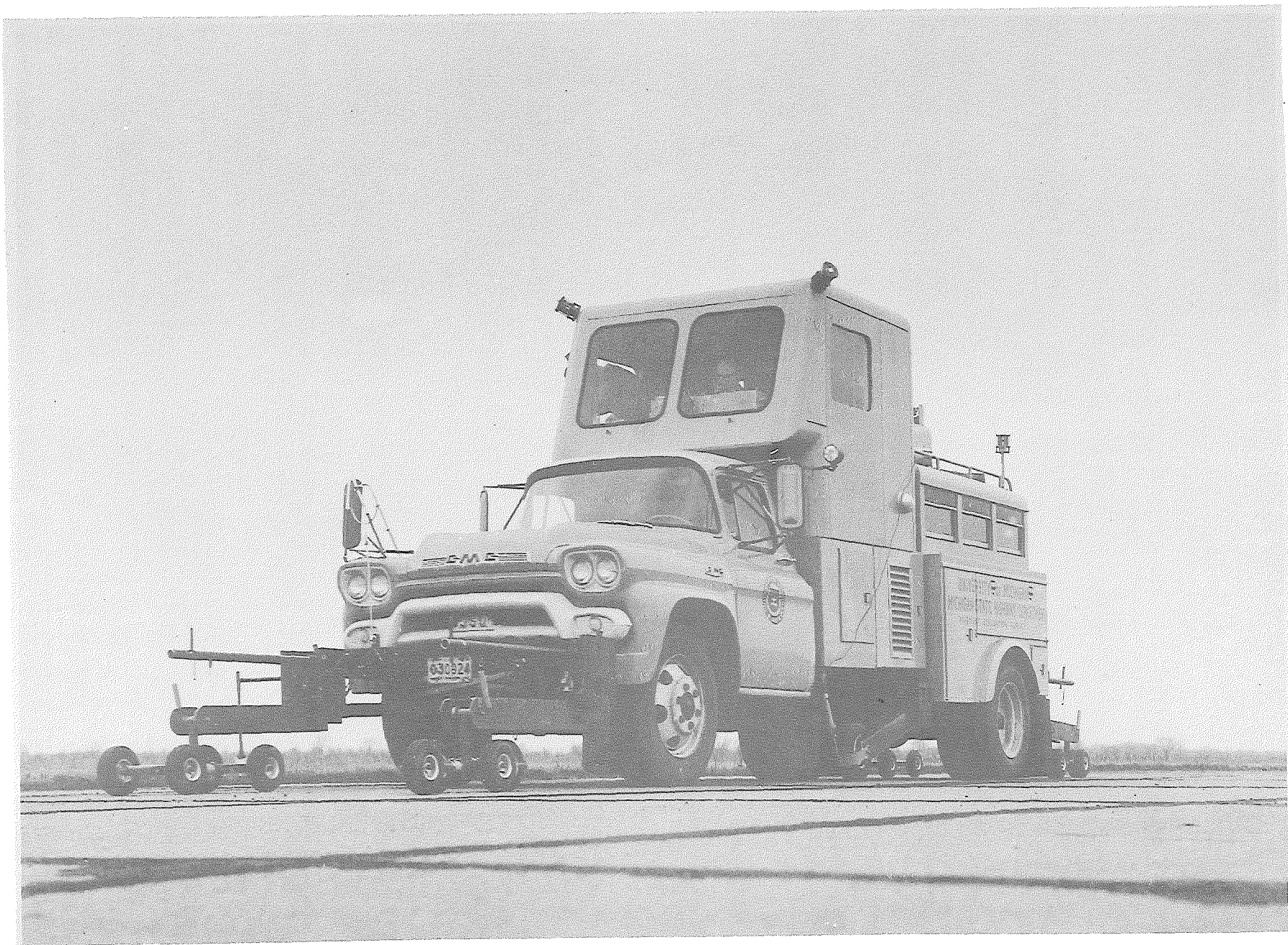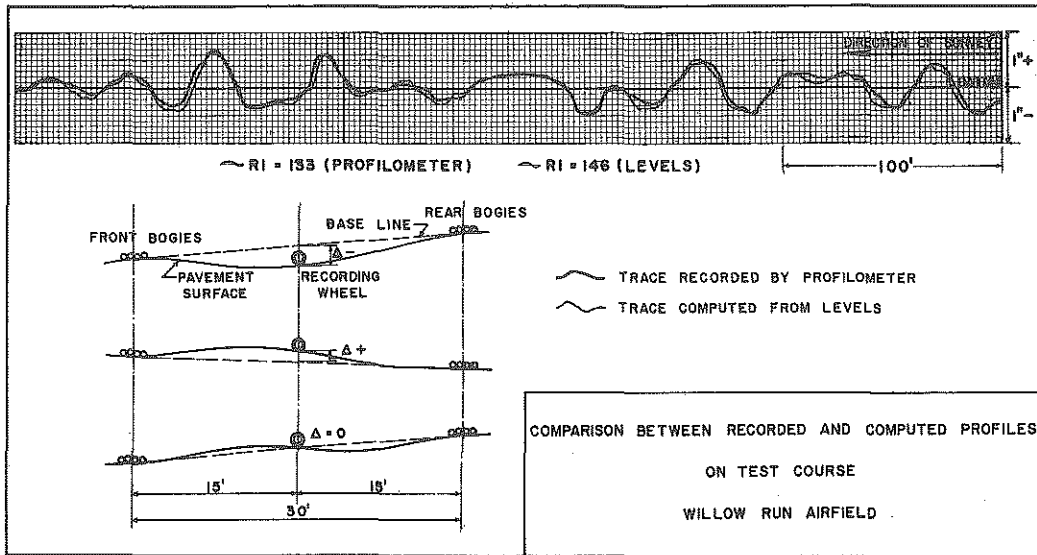
FIG. 7

TENTATIVE   PAVEMENT   ROUGHNESS   RATING

VERTICAL   DISPLACEMENT - INCHES   PER   MILE

U.S. BUREAU OF PUBLIC ROADS[1] SINGLE WHEEL ROUGHOMETER	ROUGHNESS RATING	U. OF M. PROFILE TRUCK[2]
LESS THAN 100	EXCEPTIONALLY SMOOTH	LESS THAN 50
100 - 125	VERY GOOD	50 - 75
125 - 150	GOOD	75 - 100
150 - 175	FAIR	100 - 125
175 - 200	ACCEPTABLE	125 - 150
200 - 225	POOR	150 - 175
225 - 250	VERY POOR	175 - 200
MORE THAN 250	EXTREMELY ROUGH	MORE THAN 200

(1) OPERATED AT 20 MILES PER HOUR

(2) OPERATED AT 4 TO 5 MILES PER HOUR   FIG. 8

2½" BITUMINOUS AGGREGATE-1955
¼" BITUMINOUS SEAL COAT-1954
7" GRAVEL BASE (22A)-1954
3"-5" GRAVEL
SUBGRADE: SAND

DRAINAGE: EXCELLENT

FIGURE-9A
CLASS-I FLEXIBLE
M-37, 1955   SERVICE 3 YEARS
TRAFFIC: TOTAL 500, COMMERCIAL 120, DHV 95
DATE OF SURVEY: SEPT. 11, 1958

2¼" BITUMINOUS CONCRETE-1956
2½" BITUMINOUS CONCRETE-1956
8" GRAVEL-1932
2" EARTH FILL
SUBGRADE: SAND

DRAINAGE: EXCELLENT

FIGURE-9B
CLASS-I FLEXIBLE
M-55, 1936   SERVICE 22 YEARS
TRAFFIC: TOTAL 900, COMMERCIAL 200, DHV 125
DATE OF SURVEY: SEPT. 12, 1958

FIG. 9

½" SURFACE TREATMENT-1950 & 54
2 ½" OIL AGGREGATE- 1939

8 ½" SUBBASE: GRAVEL

SUBGRADE: SAND

DRAINAGE: EXCELLENT

FIGURE -10 A
CLASS-IV FLEXIBLE
M-36, 1939   SERVICE 19 YEARS
TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
DATE OF SURVEY: DEC. 5, 1958

½" SURFACE TREATMENT-1950 & 54
2 ½" OIL AGGREGATE -1939

11" SUBBASE: GRAVEL

SUBGRADE: SANDY CLAY

DRAINAGE: POOR

FIGURE -10 B
CLASS-IV FLEXIBLE
M-36, 1939   SERVICE 19 YEARS
TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
DATE OF SURVEY: DEC. 5, 1958

FIG. 10

2½" BITUMINOUS AGGREGATE-1953
5" GRAVEL BASE-1953
12" SUBBASE: SAND
4" OLD GRAVEL ROAD (1917-1946)
SUBGRADE: SANDY CLAY
DRAINAGE: POOR

FIGURE- II A
CLASS-Ⅲ FLEXIBLE
M-100, 1953 SERVICE 5 YEARS
TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
DATE OF SURVEY: DEC. 4, 1958

2¼" BITUMINOUS AGGREGATE-1953
4" GRAVEL-1953
12" SUBBASE: SAND & GRAVEL
4" ORIGINAL GRAVEL ROAD
2'-4' SANDY CLAY
PEAT
DRAINAGE: FAIR TO POOR

FIGURE- II B
CLASS-Ⅲ FLEXIBLE
M-100, 1953 SERVICE 5 YEARS
TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
DATE OF SURVEY: DEC. 4, 1958

FIG. II

FIGURE-12 A
CLASS-I RIGID
US-112, 1926   SERVICE 32 YEARS
TRAFFIC: TOTAL 4800, COMMERCIAL 1525, DHV 675
DATE OF SURVEY: JULY 28, 1958

2⅛" BITUMINOUS CONCRETE-1948
9"-7"-9" PLAIN CONCRETE-1926
SUBGRADE: SANDY CLAY LOAM
DRAINAGE: FAIR

FIGURE-12 B
CLASS-IV RIGID
US-112, 1926   SERVICE 32 YEARS
TRAFFIC: TOTAL 4300, COMMERCIAL 1400, DHV 675
DATE OF SURVEY: AUGUST 4, 1958

3" BITUMINOUS CONCRETE-1958
2" BITUMINOUS CONCRETE-1943
9"-7"-9" PLAIN CONCRETE-1926
SUBGRADE: SANDY CLAY LOAM
DRAINAGE: POOR

FIG. 12

FILL = 2' to 3'

RI = 233

DIRECTION OF SURVEY

RI = 225

STATION
CRACKS CR = 1.33 to 1.29
JOINTS

9" PLAIN CONCRETE - 1942

18"- 24" SAND SUBBASE

HEAVY CLAY SUBGRADE

DRAINAGE: POOR

FIGURE - 13 A

CLASS - II   RIGID

US - 24A, 1942, SERVICE 16 YEARS

TRAFFIC: TOTAL 10,000, COMMERCIAL 3025, DHV 575

CONTRACT: 30-C3 & C4   DATE OF SURVEY: JUNE 19, 1958

FILL

RI = 236

DIRECTION OF SURVEY

RI = 105

STATION
CRACKS CR = 6.60
JOINTS

9" REINFORCED CONCRETE
3" SELECTED SUBBASE
9" SAND SUBBASE

SAND FILL

DRAINAGE: GOOD

FIGURE - 13 B

CLASS - I   RIGID

US - 131, 1956   SERVICE 3 YEARS

TRAFFIC: TOTAL 8700, COMMERCIAL 3000, DHV 2000

CONTRACT: 86-C1   DATE OF SURVEY: NOV. 7, 1959

FIG. 13

FIGURE –14A
CLASS – II FLEXIBLE
M – 57, 1952 SERVICE 6 YEARS
TRAFFIC: TOTAL 1000, COMMERCIAL 200, DHV 130
SURVEY DATES: DEC. 5, 1958, FEB. 20 & MAY 6, 1959

2½" BITUMINOUS AGGREGATE–1952
6" GRAVEL BASE–1952
12" SUBBASE: SAND
4" ORIGINAL GRAVEL SURFACE
SUBGRADE: SANDY CLAY
DRAINAGE: GOOD

FIGURE –14B
CLASS – III FLEXIBLE
M – 36, 1939 SERVICE 19 YEARS
TRAFFIC: TOTAL 2000, COMMERCIAL 388, DHV 260
SURVEY DATES: DEC. 5, 1958, FEB. 20 & MAY 6, 1959

½" SURFACE TREATMENT – 1950 & '54
2½" OIL AGGREGATE – 1939
11" SUBBASE: GRAVEL
SUBGRADE: SANDY CLAY
DRAINAGE: POOR

FIG. 14

CULVERT

DIRECTION OF SURVEY

DATE OF SURVEY	ROUGHNESS INDEX (R.I.) IN./MILE
OUTER WHEEL PATH	
9-25-58	53
3- 7-59	68
6-21-59	36
10- 2-59	43
11-24-59	34
3- 8-60	36
9- 2-60	41
3-21-61	47
6-21-61	45
9-15-61	36
10- 5-61	37
12-27-61	41
3-13-62	68
4-17-62	42
INNER WHEEL PATH	
9-25-58	23
3- 7-59	67
6-21-59	26
10- 2-59	23
11-24-59	28
3- 8-60	27
9- 2-60	30
3-21-61	33
6-21-61	37
9-15-61	29
10- 5-61	33
12-27-61	34
3-13-62	61
4-17-62	33

THE ABOVE ROUGHNESS INDICES ARE FOR THE PRESENTED
SECTION, LOCATED BETWEEN 915' TO 465' SOUTH OF
BERG ROAD.

4 1/2" BITUMINOUS CONCRETE
4" AGGREGATE BASE (58 % CRUSHED)
4" AGGREGATE BASE (58 % CRUSHED)] SPREAD IN ONE
5" SELECTED AGGREGATE SUBBASE] OPERATION

25"(MINIMUM) SAND SUBGRADE

ORIGINAL SOIL - SAUGATUCK SAND

DRAINAGE: EXCELLENT

US-31 MUSKEGON - GRAND HAVEN EXPRESSWAY

CLASS-I FLEXIBLE PAVEMENT

TRAFFIC: TOTAL 8000, COMMERCIAL 1250, DHV 920

PROJECT BM 61074-C3RN, 1958
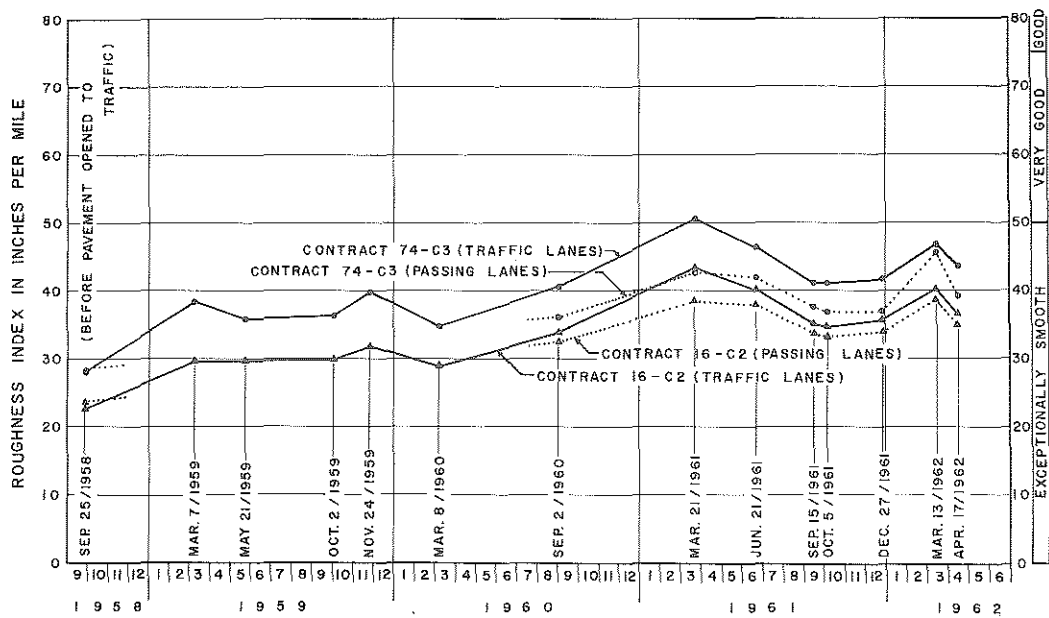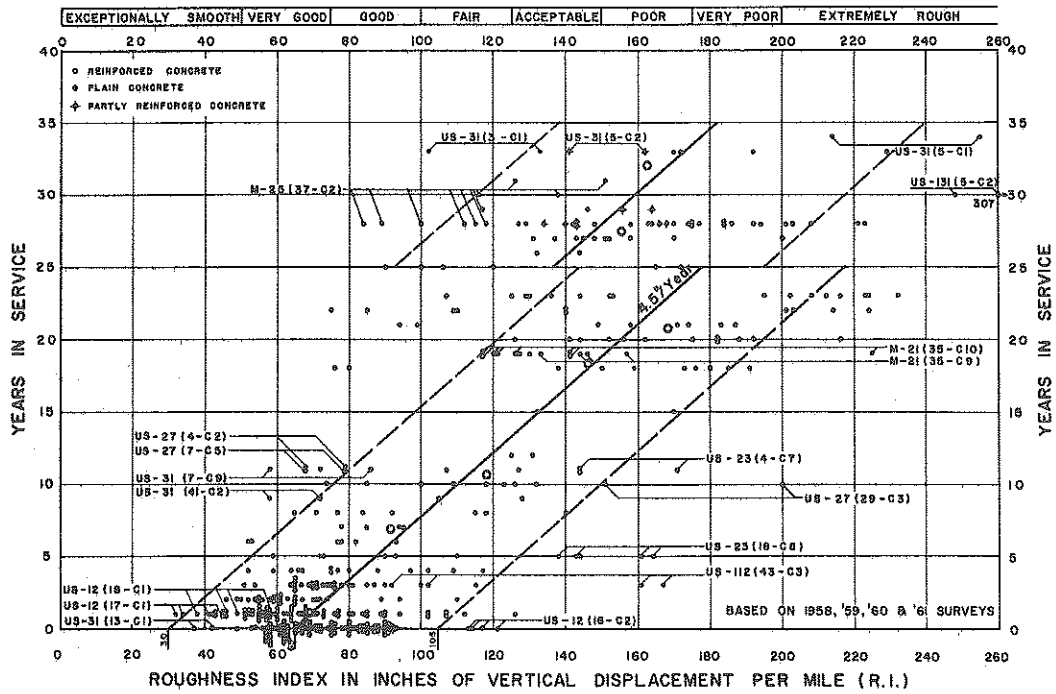
NORTHBOUND TRAFFIC LANE

FIG. 15

CUMULATIVE CHANGES IN ROUGHNESS

US-31 MUSKEGON - GRAND HAVEN EXPRESSWAY
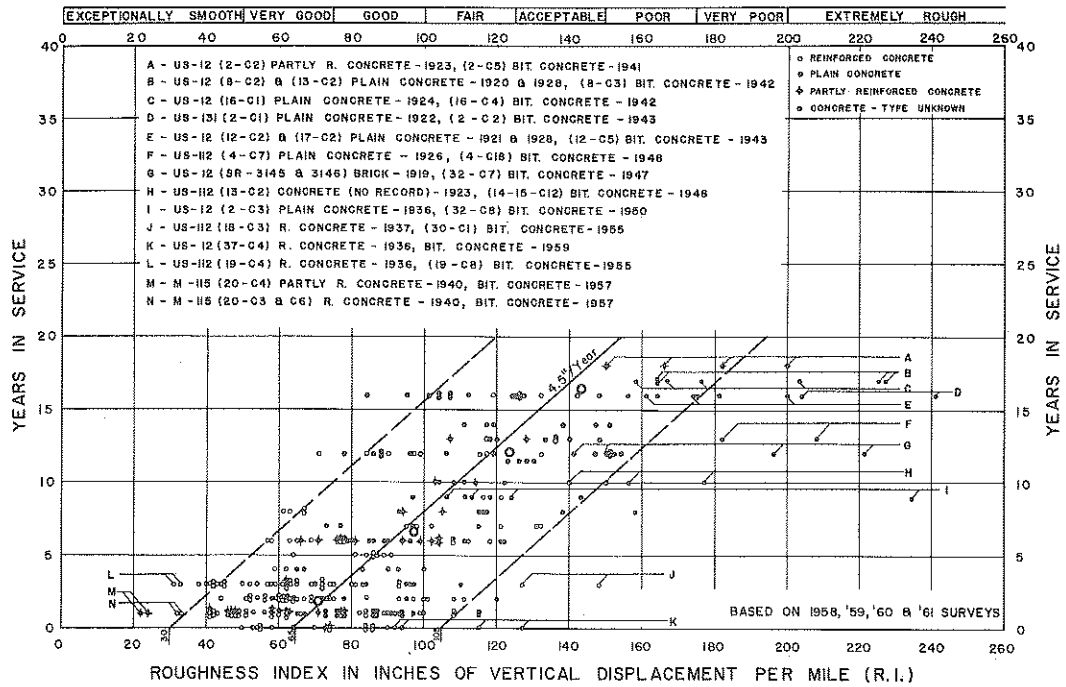
CLASS-1 FLEXIBLE PAVEMENT

FIG. 16

CUMULATIVE CHANGES WITH AGE IN SERVICE

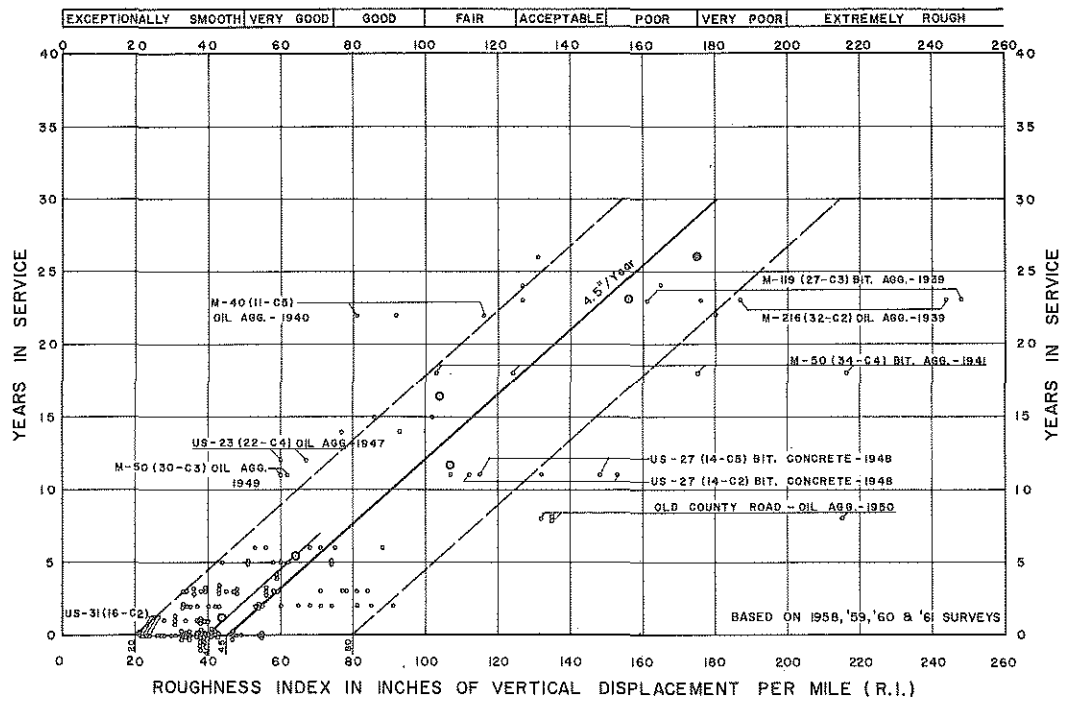RIGID PAVEMENT – CLASS – I

TRAFFIC LANE

FIG. 17

CUMULATIVE CHANGES WITH AGE IN SERVICE

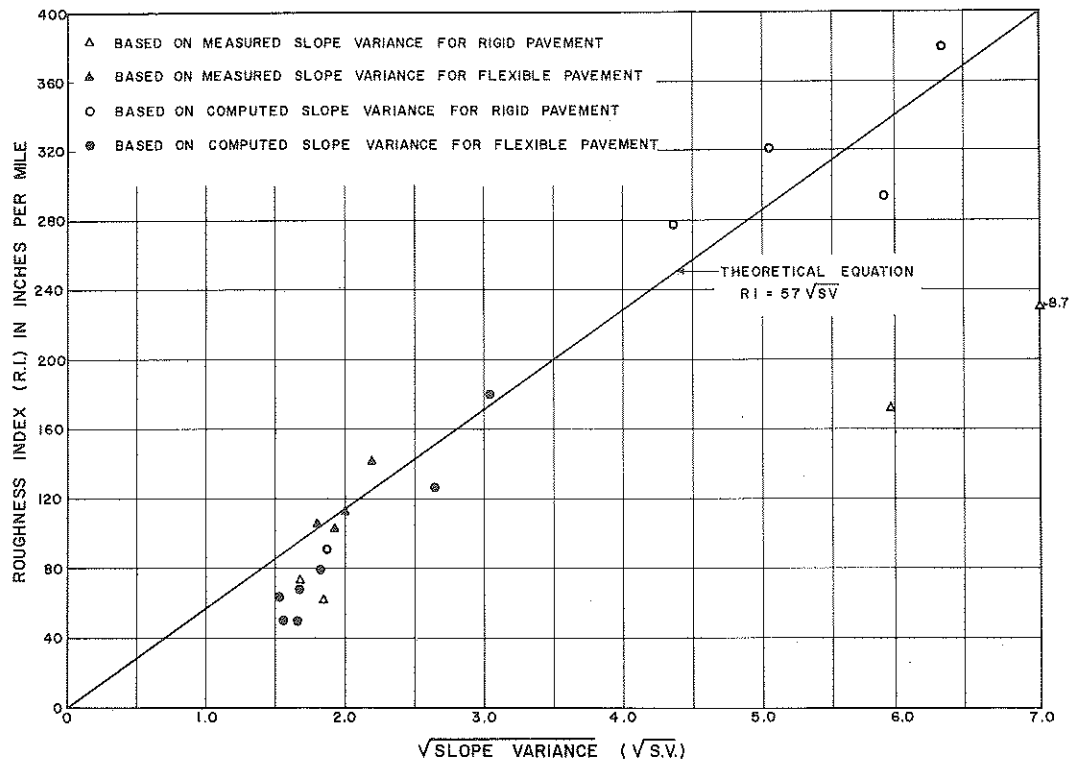RECAPPED CONCRETE PAVEMENT - CLASS - I

TRAFFIC LANE

FIG. 18

CUMULATIVE CHANGES WITH AGE IN SERVICE
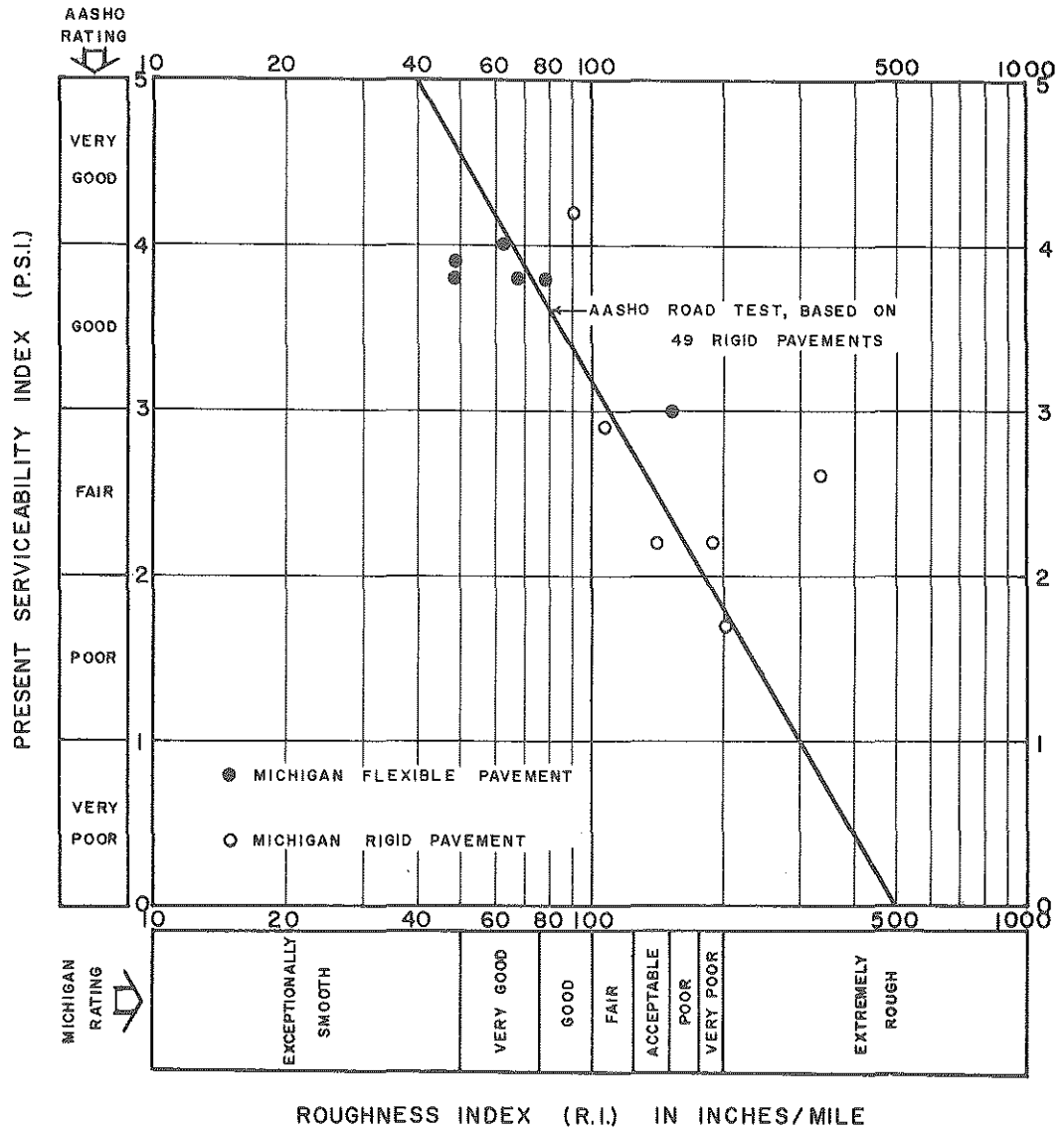
FLEXIBLE PAVEMENT - CLASS - I

TRAFFIC LANE

FIG. 19

MICHIGAN ROUGHNESS INDEX
VS.
AASHO SLOPE VARIANCE

FIG. 20

MICHIGAN ROUGHNESS INDEX
VS.
AASHO PRESENT SERVICEABILITY INDEX
FIG. 21