

MICHIGAN PAVEMENT PERFORMANCE STUDY

FIVE YEAR SUMMARY

The following papers based on the Michigan Pavement Performance Study present the most concrete results of that study. Combined under one cover, they present the essential findings of the five year cooperative research program. As such they are presented as an integral part of the final report which is being submitted at the end of the last contract period. Normally, research reports are first written for internal review and consumption and are then judged on the basis of acceptability for publication or other public presentation. In this case, the normal procedure has been reversed as the reports have already been published and received with widespread interest and recognition.

LIST OF PUBLICATIONS AND PAPERS

1. "Pavement Profile Surveys to Correlate Michigan Design Practice with Service Behavior", Proceedings, Highway Research Board, Vol. 38, 1959. (Co-author, O. L. Stokstad)
2. "Legal Weight Limitations on Motor Vehicles", Paper presented at 44th Annual Michigan Highway Conference, Grand Rapids, Michigan, March, 1959.
3. "Service Behavior as a Criterion for Pavement Design", Paper presented at 48th Annual Meeting of Western Petroleum Refiners Association, San Antonio, Texas, March, 1960.
4. "Cumulative Changes in Rigid Pavement with Age in Service", Highway Research Board Bulletin No. 328, 1962. (Paper presented at 41st Annual Meeting of Highway Research Board, Washington, D. C., January, 1962)

5. "Design, Maintenance, and Performance of Resurfaced Pavements at Willow Run Airfield", Highway Research Board Bulletin No. 322, 1962. (Paper presented at 41st Annual Meeting of Highway Research Board, Washington, D. C., January, 1962)
6. "The Michigan Pavement Performance Study for Design Control and Serviceability Rating", Proceedings, The International Conference on The Structural Design of Asphalt Pavements, University of Michigan, 1962.

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PAVEMENT PROFILE SURVEYS TO CORRELATE

MICHIGAN DESIGN PRACTICE WITH

SERVICE BEHAVIOR

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MICHIGAN (M.T.A.) PAVEMENT SURVEY

IN COOPERATION WITH

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Michigan Pavement Performance Surveys

Pavement performance surveys have been used in Michigan for many years to correlate pavement design and performance under actual conditions of service. In 1925 a group of highway engineers and pedologists including the late V. R. Burton assisted by Benkelman, Stokstad and Kellogg, started making state-wide surveys of pavement condition as related to soil types and climatic environment. Some of the results of these early surveys are reported in Highway Research Board Proceedings (1,2,3) and indicate that these investigators early found significant correlation between pavement performance and environmental factors including soil type, drainage and climatic influences. This general approach has continued to be used in Michigan as the primary basis of pavement design, utilizing subgrades of high bearing capacity, providing good drainage and eliminating frost-susceptible soils.

Since 1940 the Department's road design engineers have designed new roads in the state trunk line system to carry legal axle loads at all seasons of the year. Along with this design objective every opportunity has been taken to rebuild the old roads and bring them up to the same standards. During the development of the Michigan pavement design procedures, it has been recognized that more precise methods of evaluating pavement performance and measuring the effect of various design conditions were desirable and in more recent years several re-

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search studies have been directed toward the development of such improved procedures. During the period from 1952 through 1956 the University of Michigan, the Michigan State Highway Department and the Wire Reinforcement Institute collaborated in a study to evaluate the effect of steel reinforcement in concrete pavements under actual service conditions. Pavement performance surveys were conducted on selected projects in southern Michigan which had been in service for periods varying from ten to twenty-five years. Each project included comparable reinforced and unreinforced sections which were observed over a period of approximately five years. Analysis of these projects indicated that performance of rigid pavements could be measured in terms of two basic factors -- structural continuity and riding quality, or their counterparts, cracking and pavement roughness. Loss of structural continuity is reflected in pavement cracking with relative performance being shown in progressive changes in the cracking pattern. Similarly progressive changes in the pavement profile or pavement roughness give evidence of continued displacement in the supporting subgrade.

Analysis of pavement condition in these selected projects showed a significant difference inasmuch as pavements with steel reinforcement were measurably smoother and the cracking was measurably less than in unreinforced pavements. While these studies gave positive evidence on the effect of steel reinforcement as a factor in pavement design, the importance to the present discussion lies in the demonstration that the behavior of existing pavements under actual service conditions could be measured quantitatively in terms of pavement roughness and structural continuity. The results obtained which have been reported (4,5) encouraged the continuation of such studies and when the opportunity came, a state-wide survey of the Michigan trunk line system was undertaken.

First Pavement Evaluation of January 1, 1958

This work was organized as a cooperative research program in which the Transportation Institute of the University of Michigan collaborated with the

Michigan State Highway Department. The first phase of this program was the review and **recapitulation** of pavement adequacy on the state trunk line system, an objective **that was** accomplished with the publication by the Highway Department of the **First Pavement Evaluation Survey** of January 1, 1958, shown in Fig. 1. This first state-wide evaluation provides the foundation for future surveys and will be the basis for reclassification of highways and for future improvements that will be made from time to time. In this road classification for legal axle loads, four levels of adequacy have been selected which will subsequently be referred to as Class 1, 2, 3 or 4, and are defined as follows:

Class 1 (Solid black)

No seasonal restriction; pavement and subgrade adequate for year-round service as represented by natural sand and gravel subgrades with superior natural drainage.

Class 2 (Dot-dash black)

No seasonal restriction; 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 (Dot-dash red)

Spring load restriction 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 (Solid red)

Spring load restrictions required; pavement design 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 results of the first pavement evaluation covering some 9300 miles of trunk line highway may be summarized as follows in terms of the four levels of adequacy represented by the road classification used, with approximate percentages

on a state-wide basis:

Adequate	--	Class 1:	30.5 %	Class 2:	23.8 %	54.3 % Total
Inadequate	--	Class 3:	31.5 %	Class 4:	14.2 %	45.7 % Total

Combining Classes 1 and 2 into roads adequate for year-round service, the percentage in round figures is 55 per cent as compared to 45 per cent of Class 3 and 4 roads which are inadequate for legal axle loads. There are several other interesting points in connection with the map shown in Fig. 1. In the ten districts in the state, District 1, the western portion of the Upper Peninsula, has the lowest percentage of adequate roads amounting to only 35 per cent as compared to 65 per cent of inadequate roads. This is an area of highly susceptible soils, frequently inadequate drainage and severe frost action. On the other hand, there are several locations in the state where soil conditions are exceptionally good. Thus in District Nos. 3 and 7, the percentage of adequate roads is approximately 73 per cent, while the inadequate roads are only 27 per cent. In the central portion of the state there is a large area including Kalkaska and Crawford Counties where natural sand subgrades have provided Class 1 roads on all state trunk lines in the area. Similar areas are found in District 7 in the southwest corner of the state. Another feature shown on this map is the network of divided lane expressways radiating from the metropolitan centers of Detroit, Lansing and Grand Rapids, representing the accomplishment of the early years of modern expressway or turnpike construction characteristic of the interstate highway system.

The first pavement evaluation brought together a great volume of basic design data available in the Department's records, as well as other pertinent information on soil types and drainage, maintenance experience and pavement performance data going back to the previously mentioned condition surveys of many years ago. Through the collaboration of the District Soil Engineers and other department personnel, much of this classification reflects the unrecorded knowledge of the men who have lived with these roads for many years. The pavement evaluation

map brings together this great store of working knowledge and for the first time presents it in a completely integrated form as a matter of definite record. It will thus provide a foundation and starting point for future reclassification in terms of pavement adequacy and provide a basis for future improvements in the highway system.

Pavement performance surveys which are now in progress have as their objective more precise measurement of pavement adequacy leading to improvement in the accuracy of this first classification. It is expected that weak sections will be found in roads now classified as adequate and strong sections in those classified as inadequate. In the case of weak sections, more extensive field investigations will be made to pinpoint the sources of weakness and determine the most practicable corrective measures. This leads directly to the reconstruction or betterment of such roads, and already preliminary study of the problems involved has led to including several strategic sections in early construction programs. Guided by experience from previous surveys, the methods of measuring pavement condition are predicated on the idea that the ability of a pavement to carry the loads of present day traffic are shown by the two types of pavement behavior which in inadequate pavements result in progressive changes in pavement cracking and pavement roughness.

Elimination of Spring Load Restrictions in 1958

The first concrete result of the Michigan Pavement Performance Study was the lifting of Spring load restrictions on a selected network of state trunk lines which were opened for full legal axle loads during the Spring of 1958. The so-called 1958 Frost-Free Network is shown in Fig. 2 and originally included some 3720 miles opened to full legal axle loads. Before the state-wide Spring load restrictions were invoked, some 825 miles were added to the original network, making a total in the frost-free network of some 4545 miles or almost 50 per cent of the state trunk line mileage. Included in this network was some 27 per cent of

roads rated inadequate on which restrictions were lifted in order to provide a reasonable number of through routes, both east and west and north and south, and certain access roads to important industrial areas.

This first year's experience was considered satisfactory with surprisingly little apparent damage due to lifting of restrictions and only a few isolated cases of roads which had to be taken off the frost-free network because of abnormally weak sections not practical to keep in service by heavy maintenance. Judged in terms of public benefit, there has been little question of the justification of the forward move represented by the raising of load limitations during the Spring period under the controlled conditions represented by this action. In this connection it can be pointed out that the cost of Spring load restrictions to industry and agriculture in the state of Michigan was estimated to be some \$20,000,000 a year, a substantial part of which was saved during the 1958 Spring period by providing for the free movement of legal axle loads on the frost-free network.

Michigan Standards of Pavement Design

As indicated by the road classification used in the first state-wide pavement evaluation survey and in the frost-free network, the Michigan standards of pavement design depend on a correlation of pavement performance in actual service with associated conditions of environment. The important environmental factors are soil conditions as reflected in texture, susceptibility to moisture change and frost action, drainage as affected by grade line and drainability of the subgrade soils, and climatic influences of rainfall and temperature. At the risk of some over-simplification, the design procedure can be described as one of providing an adequate foundation to carry the applied loads but primarily capable of resisting the destructive effects of environment. In such a procedure the pavement surface becomes a protective cover of sufficient thickness and structural continuity to distribute concentrated loads to the supporting subgrade, but its

major function is to confine that subgrade to preserve its internal stability and protect it from the wear and surface deterioration caused by rapidly moving loads.

As a matter of design procedure it was indicated in the road classification that there are many miles of highways built on natural granular materials perfectly capable of carrying legal axle loads throughout the year regardless of the effect of environment. Where soil conditions and environment require measures of improvement, the most effective design compensations consist of employing a high grade line for improved drainage, removal of the frost-susceptible soils (silt and very fine sand) to the depth of frost penetration, and finally the use of free draining sand subbases over clay soils for pumping control, compensation for volume changes and for load distribution.

In terms of pavement cross-section, the result of the Michigan method of pavement design has been summarized in Fig. 3 in a table of recommended designs for the several classifications of traffic in terms of volume and vehicle size and weight. In this table are represented Michigan's present design standards, so formulated as to indicate the corrective measures required to compensate for subgrade deficiencies where they occur. Thus it is assumed that Class 1 roads require no subgrade compensation and the conventional paved surfaces may be laid directly on the natural subgrade. It is also assumed that in all cases inadequate surface drainage will be met by normal grade raises and frost-susceptible soils will be removed to the depth of frost penetration, or in lieu of such provisions equivalent alternates will be provided.

The important provisions in the pavement sections shown in Fig. 3 come in the cases where normal grade lines and removal of objectionable soils provide only partial protection. Thus in fine-grained undrainable subgrade soils subject to concentration of soil moisture, the most effective compensation for seasonal loss of strength and certain other deficiencies such as pumping is the use of free draining granular subbases. In this connection it is significant that the

design standards shown in Fig. 3 require substantially the same foundation treatment through out the entire range of traffic volume and classifications of vehicle size and weight. In terms of what is commonly called the "basic road" or the first increment of pavement required only for light vehicles, Michigan design standards would provide the same drainage and substantially the same subbase and base construction for all classifications of road use to protect the pavement surface against the destructive effects of environment. From this it follows that the Michigan standards are more closely related to soil conditions, drainage and climatic environment than they are to vehicle size and weight. Consequently the variation in thickness as related to vehicle size and weight is limited to rather nominal increases except in the case of heavy duty flexible pavements where greater increase in total thickness is required for load distribution.

Pavement Profiles to Measure Service Behavior

The second phase of the Michigan Pavement Performance Survey reported herein has to do with the more detailed measurement and recording of pavement profiles and structural continuity of the pavement surface. This part of the project has been carried on with the cooperation and support of three industry associations, the Michigan Trucking Association, the American Trucking Associations, Inc., and the Motor Truck Division of the Automobile Manufacturers Association. These three sponsors financed the research now being carried on by the Transportation Institute of the University of Michigan whose representatives work directly with the Michigan State Highway Department.

Active work on this project began in December, 1957, with the building of a truck-mounted profilometer or profilograph shown in Fig. 4. This truck is specially equipped to trace and record an accurate profile in each wheel track of a pavement. Two sets of so-called "bogie wheels" located in front and back of the truck thirty feet apart provide reference points on the pavement surface from which vertical displacement is measured by the recording wheel midway between

these reference points. Pavement profiles are recorded on a continuous chart and may be retraced after a designated period of service to measure any progressive changes that may have taken place. The tracing and recording mechanism is connected to electronic integrators which measure the cumulative vertical deflection of the pavement in inches per mile for any selected length of pavement. This cumulative vertical deflection in inches per mile is used as a roughness index in the subsequent pavement classification reported herein. The roughness is stamped out on the profile chart for each quarter mile to aid in selecting critical sections which may be isolated for further investigation, or possible defects in design or construction.

The University of Michigan equipment is modeled after that designed and used by the California State Highway Department. F. N. Hveem, Materials and Research Engineer for California, provided much useful information including plans and specifications for the design of their equipment and made California's experience available to the authors and their associates in designing and building the University of Michigan profile truck. In addition to recording the cumulative vertical displacement, equipment is provided for measuring and recording the location of cracks and joints, to provide a measure of the structural continuity of the pavement surface. Photographic equipment is also being developed to take a continuous film strip of the pavement coordinated with the recorded profile, but this has not been available for the surveys completed up to the present time. Means are also provided to measure and record the relative elevation in both wheel tracks, a device which is called a "roll indicator" with a recording pen in the center of the profile chart.

The importance attached to pavement profiles arises from the fact that even an infinitesimal amount of permanent displacement in the pavement structure will under many repetitions of load lead to increasing roughness and eventual destruction. When the pavement is first built, assuming a reasonably good job, the surface irregularities will be small and the riding quality will be good or excellent.

If the subgrade is of potentially high capacity such as sand or gravel, it should be capable of supporting the pavement surface without continued deformation or displacement. If the subgrades are poorly compacted and non-uniform, they may nevertheless consolidate under traffic and produce a rough-riding pavement. However, after a period of conditioning under service, they become stable and the riding quality of the pavement can be recovered by resurfacing. It would then seem reasonable to expect that this riding quality could be preserved for an indefinite period of service.

On the other hand if the subgrade is weak and actually deficient in supporting capacity, such as plastic clay, there may be continued permanent deformation under applied loads. Such pavements not only become rough but continue to get rougher and eventually must be rebuilt or periodically resurfaced without any expectation of ever being adequate for modern traffic. In the same category must be placed those pavements built on soil susceptible to moisture fluctuation or frost action. Such subgrade soils lose their stability periodically and also will be subjected to pumping action in pavements not properly protected against this phenomenon.

Progressive changes of this character would be revealed by subsequent profiles of a selected pavement taken after some significant period of service. Measurement of differences of this kind represents the normal use of the truck-mounted profilometer in the planned research program. This plan requires an initial or basic profile taken preferably as soon as the road is built to which all future profiles could be compared. This would mean that data of the kind being gathered would not become of value for some period of years during which successive pavement profiles were being recorded. While such long range objectives are very much a part of the present program, the need for immediate information in determination of pavement adequacy and reclassification of Michigan trunk lines has focused attention on possible value of the roughness measurements that have been made during the first year's operation of the profile truck.

Up to the present time more than 1800 miles of pavement profiles have been accumulated covering some of the main traffic routes within the state and certain projects of special interest from the standpoint of research and design procedure. These data show that there is a wide variation in relative roughness on various roads throughout the state with differentials which appear to be great enough to serve as a means of identifying deficiencies in design or in the natural conditions to which the pavement is subjected. However, the use of a specific figure in terms of vertical inches of displacement per mile of pavement as a roughness index poses a somewhat different problem from relying upon the differential between two successive pavement profiles. In this connection it would be desirable to record actual vertical displacement that could be checked by precise level measurements or some other means so that it represents a definite physical condition rather than simply a relative roughness in empirical terms. Looking at it from this viewpoint, one of the first questions which arises in connection with the recorded pavement profiles is just what it is that the profile truck is measuring. The answer to this question depends primarily on the selection of a datum or zero elevation to which the measured vertical deflections may be referred. For the purpose of explanation, several pavement profiles taken on a test course at Willow Run Airport have been plotted for comparison in Fig. 5.

The profile shown as a heavy line is the trace recorded by the profilometer truck. The profile shown as a thin line has been computed from elevations taken from levels at intervals of 2-1/2 feet along the test course. The latter profile represents the vertical difference at the recording station at the center of the truck with respect to the average elevation of the bogie wheels in front and back of the truck. In other words, the reference plane for vertical displacement is represented by a straight line drawn from the center of gravity of the two sets of bogie wheels.

This computed trace is considered to be most representative of the vertical

displacements recorded by the profile truck which is actually measuring deflection at the center of a floating base line thirty feet in length. By visual inspection there is reasonably good agreement between the recorded profile and that constructed from the level survey. This does not mean that the cumulative vertical displacement previously defined as the roughness index will necessarily be the same for both of the profiles shown as there are at least two basic differences in the two methods of measurement which cannot be readily resolved.

In the first place the elevation at reference points at the center of gravity of the bogie wheels is taken with the level rod at a specific point and is subject to pavement irregularities at that point. In the machine recorded profile the bogie wheels cover an area approximately one foot by three feet and will reflect the average elevation of four points within that area. Local high and low spots will in effect be leveled off. The bogie wheels may be described as selecting a flat surface as a reference plane which would presumably be roughly parallel to the grade line at which the pavement was laid. By introducing more pairs of bogie wheels and extending the area they cover, the more completely would local irregularities be damped out and the more closely this reference plane would approach the pavement grade.

The problem is one of extending the area of reference enough to produce profiles with sufficient accuracy to be reasonably reproducible and reflect significant differentials in position of the pavement surface and still keep the equipment itself within practical limits. Data from pavement profiles analyzed to date indicate that the profilometer equipment as now designed is capable of achieving this objective within practical limits. The second source of difference in recorded vertical displacements and those taken by levels results from the fact that the recorded displacements are those taken with wheels equipped with pneumatic tires. The inflation pressure of these tires is kept constant at 60 p.s.i. but obviously the tires will deflect and further damp out any sharp discontinuities which may be reflected in the level measurements.

In spite of these differences, it is surprising to find that the cumulative vertical displacement recorded by the profilometer is 12.45 inches in 492.5 feet or a roughness index of 133 inches per mile as compared to 13.45 inches in 485 feet or a roughness index of 146 inches per mile. The question to be answered here with respect to accuracy or validity of results is not which is the more accurate in absolute units of measurement but which is the more realistic as the measure of pavement roughness. It is considered that either type of measurement is sufficiently accurate to be reproducible within practical limits and that the cumulative vertical displacement recorded by the profilometer is the more representative of pavement roughness experience by pneumatic-tired vehicles using these pavement surfaces.

TYPICAL PAVEMENT PROFILES

The primary objective of this report is to present recorded data from projects selected from more than 1800 miles of available profiles to illustrate the various conditions of service for which the Michigan design standards have been formulated. Examples of pavements have been selected in all four classifications of adequacy and their performance measured in terms of the roughness index and the structural continuity where applicable. In this way it may first be possible to find significant evidence on the relative importance of soil type, drainage, frost action and other environmental conditions in pavement performance as compared to the effect of vehicle size and weight and traffic volume.

In the second place the examples should provide evidence of the reliability and selectivity of pavement profiles as a means of differentiating between adequate and inadequate roads on the basis of service behavior. The data should also provide some idea of the validity of pavement roughness and structural continuity as measures of pavement adequacy and further show their value as a means of identifying the factors responsible for variations in behavior.

With these introductory remarks attention may now be given to the examples selected for illustration. In reviewing these examples there are several things

to be kept in mind as a means of correlating the data on some comparative basis. In Table 6 is shown a tentative roughness rating to associate with the measured roughness indexes as representative of varying degrees of riding quality from exceptionally smooth to extremely rough. Roughness indexes in excess of 200 inches per mile are considered unacceptable, though there are many examples of such roads in service. For those familiar with the more widely used single-wheel roughometer, comparative roughness indexes have been established on the basis of a relatively small number of projects where both types of equipment have been used.

Pavement profiles are presented in a series of charts that follow a definite format which will be explained at the outset with no further reference thereafter. Each chart presents 600 feet of profile in the outer and inner wheel path of the traffic lane. The outer wheel path is generally at the top of the sheet on the right hand side of the lane with the direction followed by the profile truck generally being to the left. Cut and fill in feet is indicated at the top of the sheet. 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. It should be kept in mind that vertical displacements are recorded full scale (1" = 1") while the horizontal distances are to a scale of 1" = 25'. This results in a 1 to 300 magnification of vertical displacements, thus exaggerating the roughness in terms of pavement grade or slope.

Pavement joints and cracks are shown along a 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 this connection it should be noted that the continuity ratio is defined in terms of a standard slab length of 15 feet representing a continuity ratio of 1. Thus an uncracked slab between hundred 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 of 15 feet.

To the right of the pavement profiles are given pertinent data on the pavement section including pavement type and thickness of various components, subgrade classification, drainage, road classification from the standpoint of adequacy, project identification, traffic based on 1957 surveys, and date of survey.

Rigid Pavements

The first series of charts to be presented are for rigid pavements consisting of Portland cement concrete, both plain and reinforced, and in the case of some of the older pavements covered by a bituminous surface.

Fig. 7, Class I and Class IV, Rigid. The first set of two charts shown in Fig. 7 are taken from one of the older heavily traveled roads in Michigan, US-112. The pavement profiles in Fig. 7-A with roughness indexes of 72 in the outer wheel path and 75 in the inner would still be rated very good from the standpoint of riding quality after some thirty-two years of service insofar as the original concrete pavement is concerned with ten years of service on the bituminous resurfacing laid in 1948. The traffic is 4800 vehicles per day of which 1525 are commercial type. There are relatively few cracks between the hundred foot joints with a reduction in continuity ratio of something less than 50 per cent since construction in 1926.

The pavement section in Fig. 7-B which is only a short distance from the first section with which it is compared has been in almost continuous trouble since it was built, requiring continuous maintenance with an unusual amount of concrete patches and continued cracking of the old pavement. After having been resurfaced with bituminous concrete in 1943, it developed the rough profile shown by a heavy line with a roughness index of 291 in the outer wheel path and 395 in the inner wheel path. It was resurfaced in 1958 with the pavement profile indicated by the finer line superimposed on the profile before resurfacing. In connection with this resurfacing, it is interesting to note the degree to which slab displace-

ments are ironed out and the riding quality recovered. There is a much greater reduction in continuity ratio in Example 7-B but it should be noted that these data are not completely satisfactory. In a pavement that has been resurfaced as much as this it is difficult to locate all of the cracks in the structural slab as many of them do not show through the resurfacing. The extensive amount of concrete patching also has been indicated as joints producing the irregular spacing of joints that were originally 100 feet apart.

The significant difference in these two sections of pavement lies in the subgrade type and drainage. The pavement giving very good service in Fig. 7-A falls within a relatively small area of glacial outwash which is predominantly sandy with fair internal drainage. This short stretch of pavement was originally included in a much larger area of Class IV road rated inadequate at all times of the year. Its performance as indicated by the pavement profile and roughness index suggested reclassification and this was confirmed by field investigation. The example of Class IV pavement in Fig. 7-B is also a reclassification on the basis of the pavement profile, this section having been included in an area of Class I pavement due to an error in transposing soil survey data on the pavement evaluation map. While the classification of subgrade soil accompanied by poor drainage as Class IV is obvious, the error was discovered by a study of the pavement profiles taken over this portion of US-112.

Fig. 8-A and 8-B, Class I and Class IV, Rigid. The comparisons shown in Fig. 8-A and 8-B again show the dominant influence of subgrade with good performance over the sand subgrade with excellent drainage and poor performance over the lake bed clay with poor drainage. Other contributing factors may have entered into this difference in behavior in that the good performance is on a reinforced concrete pavement with lighter traffic and a service period of twenty-eight years as compared to the plain concrete pavement over a heavy lake bed clay with poor drainage and thirty-six years of service under substantially heavier traffic. The poor performance in the latter case is also associated with a marked reduction in

continuity ratio even though the older pavement has the advantage of being laid over an old road of four to five inches of water bound macadam. It is felt that the extra support of this old road is a factor which has enabled this pavement to last over this long period of years under otherwise rather unfavorable conditions. One thing that may be noted in connection with most of the profile charts is the fact that the roughness index of the outer wheel path is generally greater than the inner wheel path, a condition which may be either built in at the time of construction or the result of weakness along the free edge of the pavement associated with unpaved shoulders.

Fig. 9-A and Fig. 9-B, Class I and Class II, Rigid. The Class I pavement in Fig. 9-A is an example of a long period of service on a heavily traveled route where the condition of the pavement is still fair, a fact that is credited to the superior subgrade of outwash sand with excellent drainage. While the continuity ratio of this pavement has been substantially reduced by cracking, there is little evidence of faulting at the joints and cracks. The Class II pavement in Fig. 9-B is an example of the effectiveness of a 12-inch sand subbase in compensating for a natural subgrade that would be rated inferior as a heavy clay with inadequate drainage.

One of the reasons for including the Class II pavement with a service of only nine years was to show the results of two successive profiles taken several months apart with a considerable differential in air temperature. At the time the first profile was run in early October, the weather was mild and the air temperature was 77° F. When the second profile was run on December 4, 1958, there had been a considerable period of freezing weather and at the time the profile was taken the air temperature was 38° F. A comparison of the two profiles given by the heavy line and the light line indicates in the first place excellent reproducibility insofar as the profilometer truck is concerned. From the standpoint of the pavement performance, it would indicate that the temperature differential had produced very little difference in the roughness index although there was a

measurable change along the inner wheel path. While much more data would be required to establish the significance of such a small differential in roughness, it may be that the greater roughness at the time of the last survey and lower temperature could have been caused by curling and greater upward displacement at the joints. A close examination of the two profiles along the inner wheel path would indicate that there is some evidence of greater vertical displacement at the joints and cracks at the time of the second survey.

Fig. 10-A and 10-B, Class II Rigid, Plain and Reinforced Concrete. The two pavement sections shown in Figs. 10-A and 10-B provide a direct comparison between plain and reinforced concrete pavements with other design factors substantially the same. This pavement represents one designed up to Michigan's present standards with 12" sand subbase over a clay subgrade, good drainage and subjected to fairly heavy traffic. Both sections are in excellent condition and would be rated very good or exceptionally smooth with the slightly better performance in favor of the reinforced concrete section. Eleven years of service is rather short but long enough to reveal any serious weakness in the pavement. Structural continuity is still high with the absence of cracking in the plain concrete a function of the 50 foot slab length as compared to the 100 foot slab length in the reinforced section. In both cases the high capacity foundation with the 12" well compacted sand subbase is the major factor in the excellent performance of both plain and reinforced sections. The differential in roughness index is not large but is probably a measurable difference being somewhat more than the probable error in recording roughness, although not enough to indicate any marked difference in behavior.

Fig. 11-A and 11-B, Class III and Class II, Rigid. The two pavements in Fig. 11-A and 11-B differentiate between comparable reinforced concrete pavements on poorly drained clay in Fig. 11-A and in a deep cut through a sandy glacial moraine in Fig. 11-B. On the basis of the roughness index the Class III pavement would be rated poor to very poor and the Class II pavement would be rated good.

There is a considerable difference in the period of service and traffic but not considered sufficient to be the major factor in service behavior. These two sections were selected as representative of concrete pavements subjected to fairly heavy traffic on important truck routes. The reductions in structural continuity parallel the differences in pavement roughness. (Note that there is probably a reclassification involved in the above in that the pavement in Fig. 11-B is on a natural sand subgrade and not a 12" sand subbase. M-59 - Class II both sides of cut.)

Fig. 12-A and 12-B, Class III, Rigid. The two pavement sections shown in Fig. 12-A and 12-B make one of the most interesting comparisons provided by the pavement profiles presented in this report. Both sections are taken from US-41, running north out of Menominee into the Upper Peninsula and are only a few miles apart. This road, incidentally, is known as one of the roughest riding pavements in the state and the two sections selected represent the roughest and the smoothest sections. There is a startling differential in roughness index in these two sections with the rough section entirely outside the rating scale, while the smooth section would be rated good or very good. This contrast is directly the result of a difference in subgrade soil and drainage. The rough section is a swamp fill following peat excavation with the backfill material being described as a heavy sandy loam. The water table is close to the surface and in this area of deep frost penetration the combination of circumstances promote severe frost action. The smooth section, on the other hand, is on a side hill cut and fill, with the subgrade material being described as a light sandy loam sufficiently granular to supply more adequate internal drainage. This in combination with the side-hill surface drainage has produced very good performance over a service period of 22 years. Similar differentials are shown in the change in structural continuity, although to some extent the small amount of cracking in the good section is the result of the 30 foot joint spacing.

Flexible Pavements

As part of the pavement performance surveys conducted during the past year it was arranged to obtain typical profiles of the four different classifications of flexible pavement for comparison with the rigid type pavement and to test the applicability of pavement profile surveys and pavement roughness as measures of performance. The next series of examples will, therefore, be devoted to flexible pavements and their performance.

Fig. 13-A and 13-B, Class I, Flexible Pavement. The two pavement sections shown in Fig. 13-A and 13-B are representative of high type bituminous construction, which, on the basis of performance is being given increased consideration in the state. Fig. 13-A has only a service period of three years but the subgrade and structural components of the pavement are so closely similar to the pavement section in Fig. 13-B that they should give comparable performance over longer periods of time. Both are on superior type subgrades 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 would be rated as exceptionally smooth, indicating excellent construction practice as well as completely adequate carrying capacity. It may also be pointed out that subgrade stabilization in one case with earth fill and the other case with stabilized gravel were 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 foundation are important factors in the superior riding quality that has been produced.

One other special circumstance should be noted; the pavement in Fig. 13-B with a service period of twenty-two years was resurfaced in 1956 because over a period of twenty years, surface abrasion and hardening of bituminous material had

produced a fragile surface which either had to be 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, but insofar as the structural capacity of the pavement is concerned, there was no indication before resurfacing of loss in load carrying capacity reflected in increased pavement roughness.

Fig. 14-A and 14-B, Class II, Flexible. The two pavement sections in the Class II category have been selected from those roads built in comparatively recent years under the design practice of supplying a minimum 12" sand subbase over inadequate subgrades. With the raise in grade and the good internal drainage of the sand subbase, drainage conditions were changed from inadequate to good or fair. The service behavior of these pavements as measured by the roughness index is exceptional to very good. Traffic is fairly heavy on both routes and the service periods of six and nine years are rather short but sufficient to expose any serious weakness in these pavements. The section of M-57 was part of a betterment program of an old road which had proved to be completely inadequate under increasing traffic to which it was being subjected. The section of M-50 is also an old road which was going through a similar experience but the selected profile happens to be on a relocation, so has been built over the original natural subgrade. While it has maintained very good riding quality there is more evidence of permanent displacement and considerable amount of patching in the latter pavement, probably due to the longer service period and the substantially heavier traffic.

Fig. 15-A and 15-B, Class III, Flexible. The two sections of pavement in Fig. 15-A and 15-B are taken from the same road, M-100, in a section which has presented something of a problem over the years of increasing traffic. Because of weak sections in the road it has been rated as Class III, inadequate during the Spring breakup period but the pavement profile surveys generally indicate that some reclassification would be justified. The addition in 1953 of a 12" subbase and a flexible pavement with a bituminous wearing course over the old road would appear to justify at least a Class II rating and the performance over a service

period of five years with a roughness index of 34 and 40 would, in terms of riding quality, be rated as exceptionally smooth. One of the weak sections that had been the source of difficulty was selected as the example in Fig. 15-B and represents the poor performance in a comparatively short stretch. The reason for this is obvious as this section was built over a peat swamp under the old road which has never been removed and has resulted in surface subsidence, followed by extensive bituminous patching, a combination which falls far short of producing the satisfactory riding quality. The basic difficulty is permanent and as long as the unstable peat is not removed, no amount of strengthening of the pavement will correct the defect. However, with the greater portion of this road giving quite satisfactory service, it would appear that reclassification is justified, accepting that the peat swamp will always have to be provided with whatever maintenance is necessary to maintain a usable pavement.

Fig. 16-A and 16-B, Class IV, Flexible. The pavement sections shown in Fig. 16-A and 16-B supply another study in contrast of two sections taken from the same road just a few miles apart. Through this area this road has been rated Class IV, inadequate at all times of the year on the basis of visual inspection and general performance. It is an old road having been built over a period of years by County forces and later State Highway Department when the road was taken over as a trunk line. The comparison is mainly of interest as a demonstration of the dominant influence of a good sand subgrade over a comparatively limited stretch as compared to a poorly drained sandy clay subgrade. In the good section after nineteen years of service the riding quality is still rated as good, whereas on the weak section some new words would have to be added to the rating table for adequate description. These two examples would also seem to be another illustration of the value of the pavement profilometer in differentiating between those sections which would eventually have to be rebuilt and those which would not require rebuilding, thus adding a quantitative tool to supplement good engineering judgment.

Special Sections.

Three final examples have been selected to present some special pavement sections for which the profile data has particular interest.

Fig. 17 and 12-B. The pavement in Fig. 17 is of particular interest because it represents the effect of short 20 foot slabs without load transfer at the joints in what is presumably an effort to control pavement cracking. The subgrade was a heavy lake bed clay with a fill of several feet produced by a 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" sand subbase was added on top of the grade to protect the 9" plain concrete from quite certain pumping action. The small reduction in continuity ratio indicates that the crack control was excellent but the riding quality produced was still outside the tentative roughness scale. However, the most interesting feature on this pavement is the characteristic saw-tooth pattern that has been produced by the tilting and faulting of the short slabs, illustrating another valuable type of information from pavement profiles. From the standpoint of design the absence of load transfer at the joints is probably the critical weakness. For purposes of comparison the results shown in Fig. 17 can be compared to Fig. 12-B, the smooth section in US-41 where there are 30 foot joints, but with load transfer provided. Expansion joints at 60 feet are supplied with a trans-lode expansion joint base and the reinforcing is carried through the 30 foot dummy joints.

Fig. 18-A and 18-B. The last two pavement sections in Fig. 18-A and 18-B are examples of new pavements constructed in 1958 with profiles that give some measure of the high quality of the pavement construction being produced. Fig. 18-A is on a divided line lane highway, M-20, between Bay City and Midland, with a 12" granular subbase consisting of 9" of sand and 3" of selected gravel used as subgrade stabilization. This pavement would be rated as exceptionally smooth with roughness indexes of 40 and 27 in the two wheel tracks. Fig. 18-B is an example of a heavy duty flexible pavement constructed on the Muskegon-Grand Haven

expressway and is representative of the type of heavy duty flexible pavement construction proposed for use on the interstate highway system on certain selected projects. The riding quality of the pavement is exceptional with roughness indexes of 18 and 15 in the two wheel paths of the pavement. Having obtained the basic profile immediately after the pavements were constructed, these projects will be watches with considerable interest over the next few years and will provide the type of data needed to record the complete history of pavements during their useful period of service.

CONCLUSION

The studies described in this paper were undertaken in the belief that some of the perplexing problems in pavement design could be most effectively resolved by a searching examination and careful analysis of existing roads in their natural environment and under normal conditions of traffic and loading. It was further felt that these roads already subjected to years of actual service would reflect relative strength or weakness of the total pavement structure in a way that could not be duplicated in any short period of time under simulated service conditions.

The major problems presented by such an investigation were first to select some basis for evaluating pavement adequacy, second, devise procedures, design and build equipment for observing and recording whatever quantitative measures had been selected. Guided by the results of previous investigation the present project has now been in progress for a little more than a year. The truck profilometer has been designed and built. More than 1800 miles of pavement profile have been run. A portion of the data obtained has now been analyzed and from their analysis the examples presented in this report have been selected.

Another phase of the investigation the thirty year development of Michigan pavement design standards from pavement performance surveys has been reviewed and recapitulated in an integrated adequacy evaluation of the State trunk

line system. As a direct result of this evaluation, Spring load restrictions were relieved in 1958 on a network of about 50 per cent of the state trunk line system. This action saved for industry and agriculture a substantial part of an estimated annual cost of load restriction of some \$20,000,000. The studies of pavement design standards and performance surveys have been combined to focus the objective of the program on more accurate classification and more precise measures of pavement performance.

The examples which have been presented seem to indicate that a reasonably accurate pavement profile does reveal the direct correlation between service behavior under normal traffic and loading and soil conditions, drainage and climatic environment with quite unmistakable clarity and serves to identify those factors responsible for differences in behavior.

The measurement of pavement roughness by the profilometer equipment used appears to be sufficiently accurate to provide a satisfactory degree of reproducibility and measures significant differentials in the pavement profile as distinguished from normal experimental error.

The correlation between pavement roughness and the structural continuity of rigid pavements appears to be consistent and mutually supporting as measures of pavement performance and the quantitative values associated with these two factors do reflect the influence of controlling variables in service behavior.

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- (2) Survey of Soils and Pavement Condition in Progress in Michigan -- V. R. Burton. Public Roads, Vol. 7, No. 4, August, 1926.
- (3) Design of Flexible Surfaces in Michigan -- W. W. McLaughlin and O. L. Stokstad. Proceedings, Twenty-Sixth Annual Meeting, Highway Research Board, 1946.
- (4) Report on the Evaluation of Service Behavior of Plain vs. Reinforced Concrete Pavement -- W. S. Housel. University of Michigan Research Institute Report No. 2053-1-S, for Wire Reinforcement Institute, May, 1954.
- (5) Pavement Performance as Related to Design -- W. S. Housel. Proceedings of the Fourth Annual Michigan Highway Conference, University of Michigan, 1955.

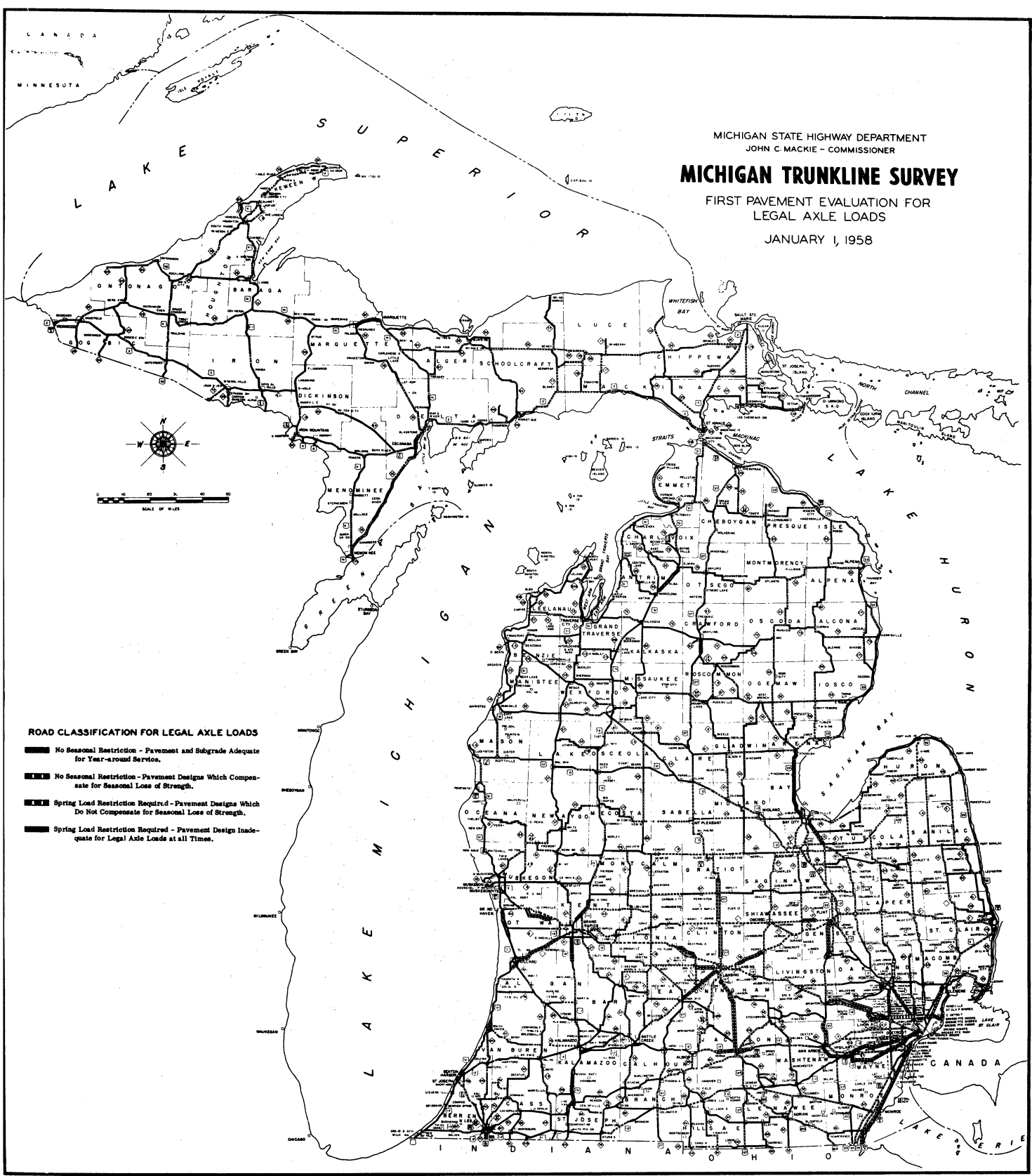


FIGURE - 1

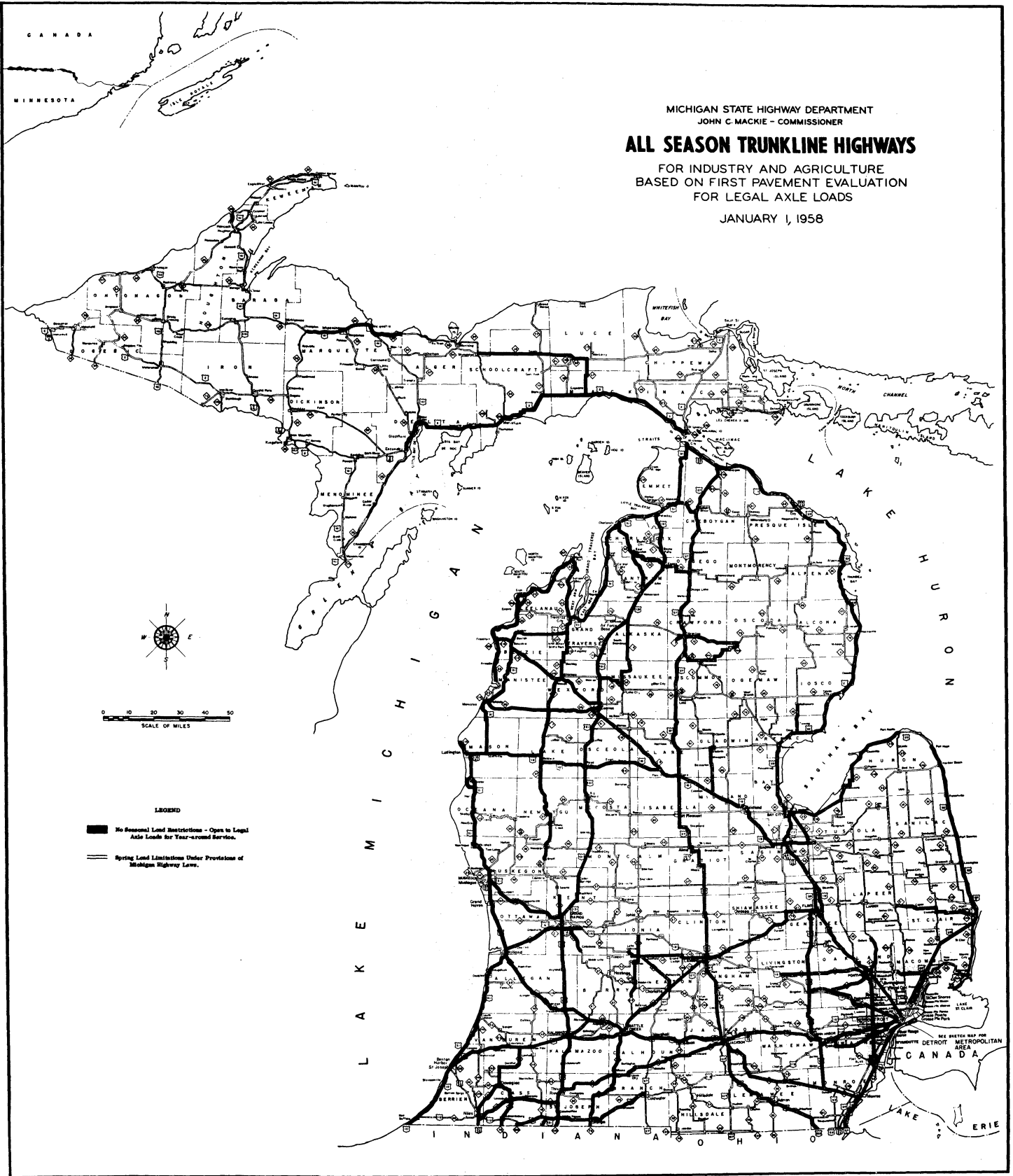


FIGURE-2

INCREMENTAL COST STUDIES
RECOMMENDED DESIGNS

RANGE IN TWO-LANE TRAFFIC VOLUMES
(VEHICLES PER DAY)

AXLE LOADS IN KIPS	RANGE IN TWO-LANE TRAFFIC VOLUMES (VEHICLES PER DAY)				HIGH 1000 & Over
	LOW 0-100	LOW-MEDIUM 100-400	MEDIUM-HIGH 400-1000		
BASIC 0-4	6 - 22 - 6 6" GRAVEL	8 - 22 - 8 3/4" BIT. SURF. 6" GRAVEL 12" GRAN. SUBBASE	8 - 22 - 8 2" BIT. SURF. 6" GRAVEL BASE 12" GRAN. SUBBASE	10 - 24 - 10 2-1/2" BIT. SURF. 6" GRAVEL BASE 14" GRAN. SUBBASE OR 6" REINF. CONCR. 14" SAND SUBBASE	
LIGHT 4-10		8 - 22 - 8 1-1/2" BIT. SURF. 6" GRAVEL 12" SUBBASE	8 - 22 - 8 2" PLANT MIX 22" GRAN. BASE & SUBBASE 2" BIT. SURF. 6" BASE 14" SUBBASE	10 - 24 - 10 8" REINF. CONCR. 14" GRAN. SUBBASE OR 3" BIT. SURF. 7" BASE 18" SUBBASE	
MEDIUM 10-18			8 - 22 - 8 8" REINF. CONCR. 14" GRAN. SUBBASE OR 3" BIT. SURF. 7" BASE 18" SUBBASE	10 - 24 - 10 9" REINF. CONCR. 14" GRAN. SUBBASE OR 3" BIT. SURF. 8" BASE 18" SUBBASE	
HEAVY 18-22				10 - 24 - 10 9" REINF. CONCR. 14" GRAN. SUBBASE OR 4" BIT. SURF. 8" BASE 20" SUBBASE	
LOADS HEAVIER THAN NOW PERMITTED 22-30			NOTE: MINIMUM DESIGN FOR P.C. CONCRETE 6" SURFACE 14" SUBBASE	9" to 11" REINF. CONC. PAV'T. 14" GRAN. SUBBASE OR 4" BIT. CONCR. 8" BASE 28" SUBBASE	



FIGURE-4 MICHIGAN TRUCK-MOUNTED PROFILOMETER

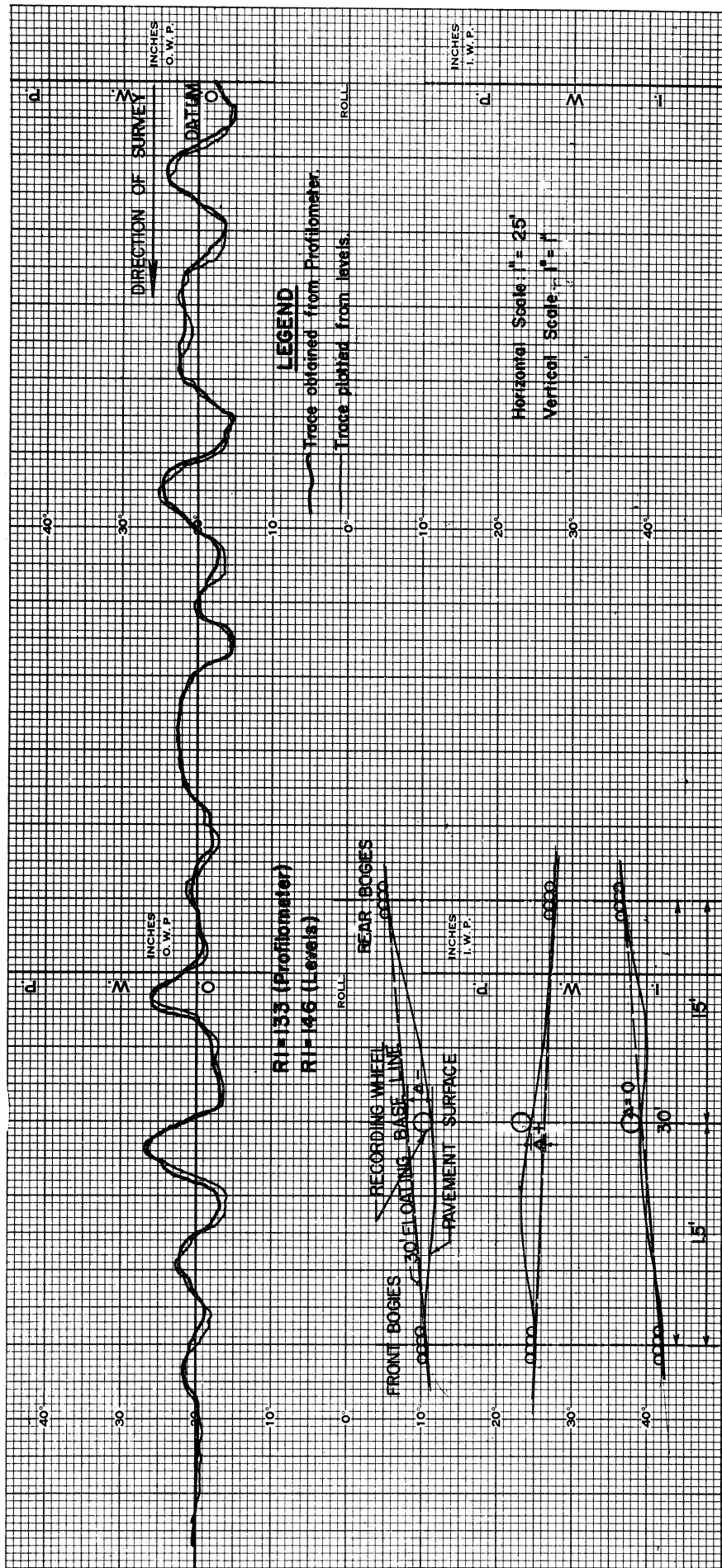


FIGURE-5
COMPARATIVE PAVEMENT PROFILE
ON TEST COURSE - WILLOW RUN AIRPORT

**TENTATIVE
PAVEMENT ROUGHNESS RATING
VERTICAL DISPLACEMENT - INCHES PER MILE**

U.S. BUREAU OF PUBLIC ROADS⁽¹⁾ <u>SINGLE WHEEL ROUGHOMETER</u>	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 4-5 MILES PER HOUR.

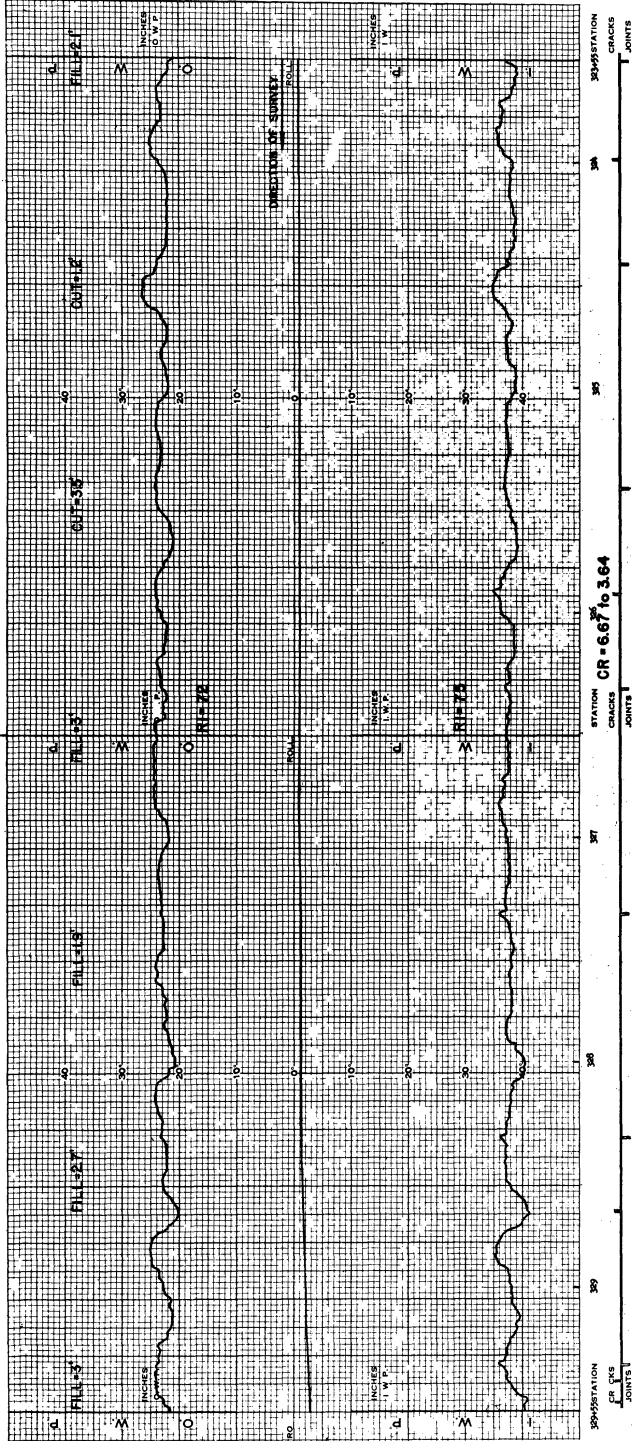
FIGURE - 6

2 1/2" BITUMINOUS CONCRETE-1948
 9'-7" 9" PLAIN CONCRETE-1926
 SUBGRADE SANDY CLAY LOAM

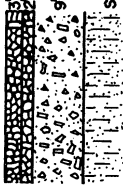


DRAINAGE: FAIR

FIGURE-7 A
 CLASS-I RIGID
 US-112, 1926 SERVICE 32 YEARS
 TRAFFIC TOTAL 4800, COMMERCIAL 1525, DHV 675
 DATE OF SURVEY: JULY 28, 1958

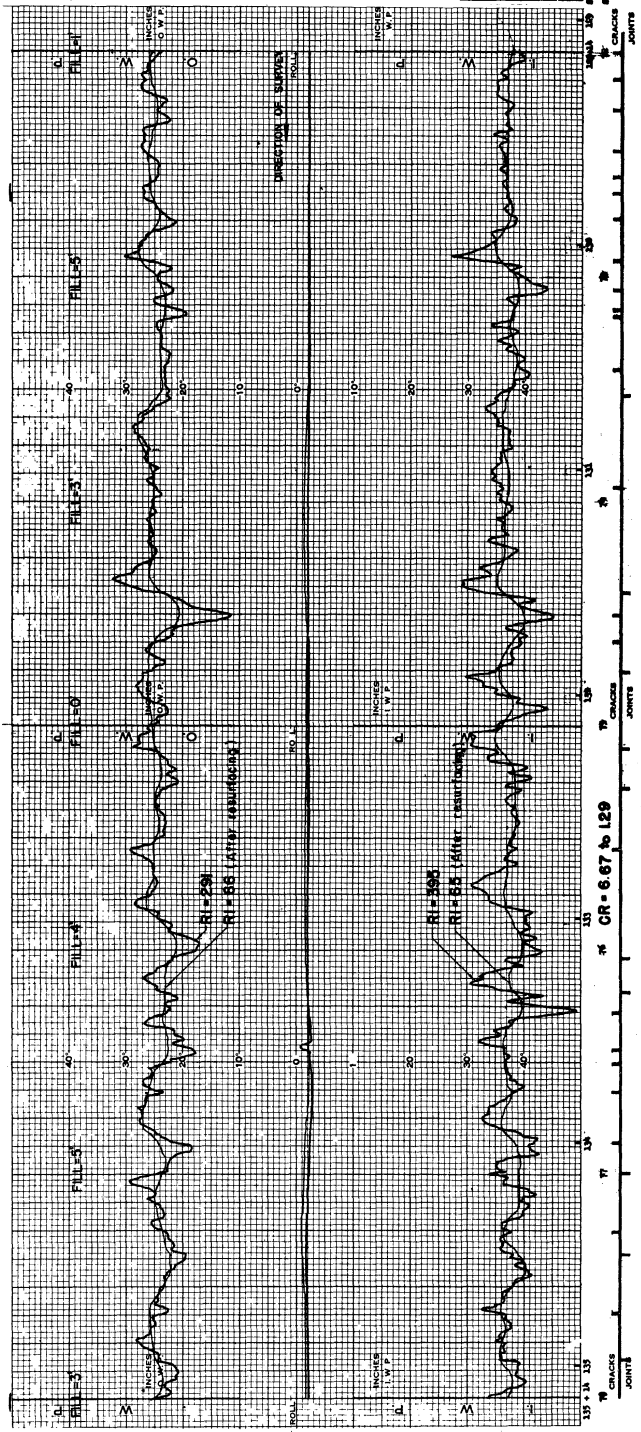


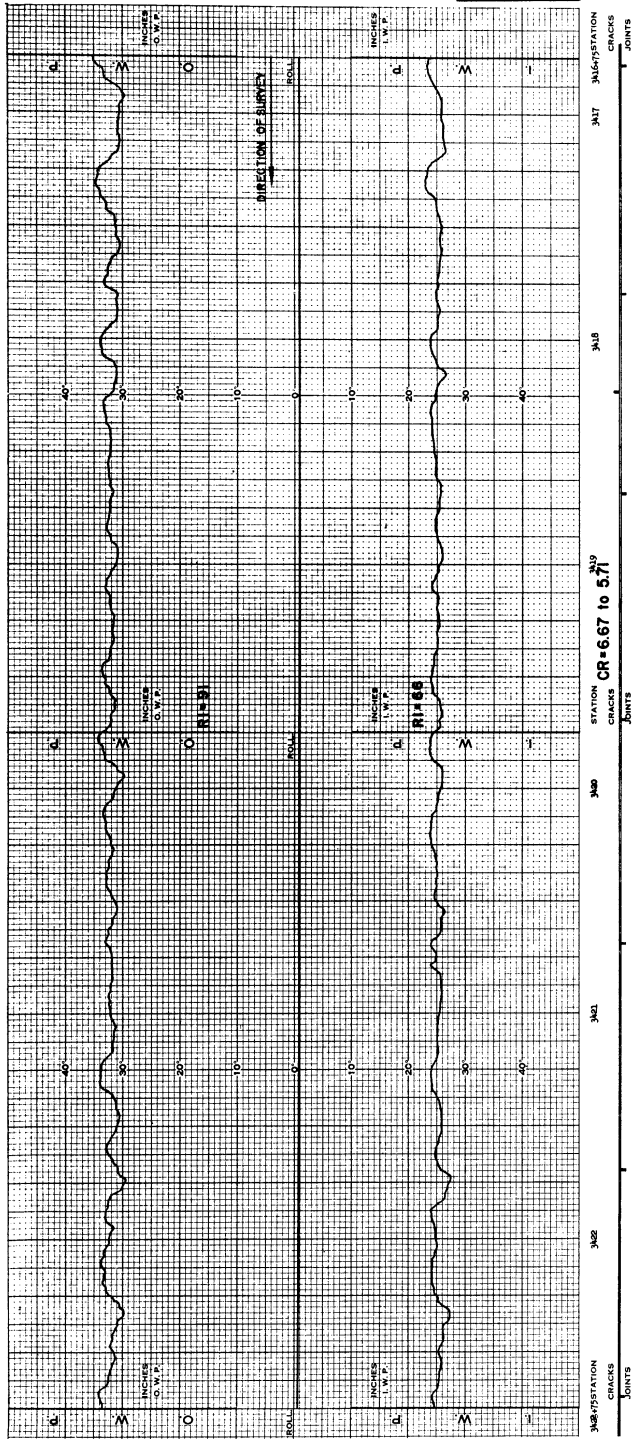
BITUMINOUS CONCRETE-1958
 BITUMINOUS CONCRETE-1943
 9'-7" 9" PLAIN CONCRETE-1926
 SUBGRADE SANDY LAY LOAM



DRAINAGE: POOR

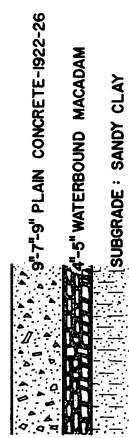
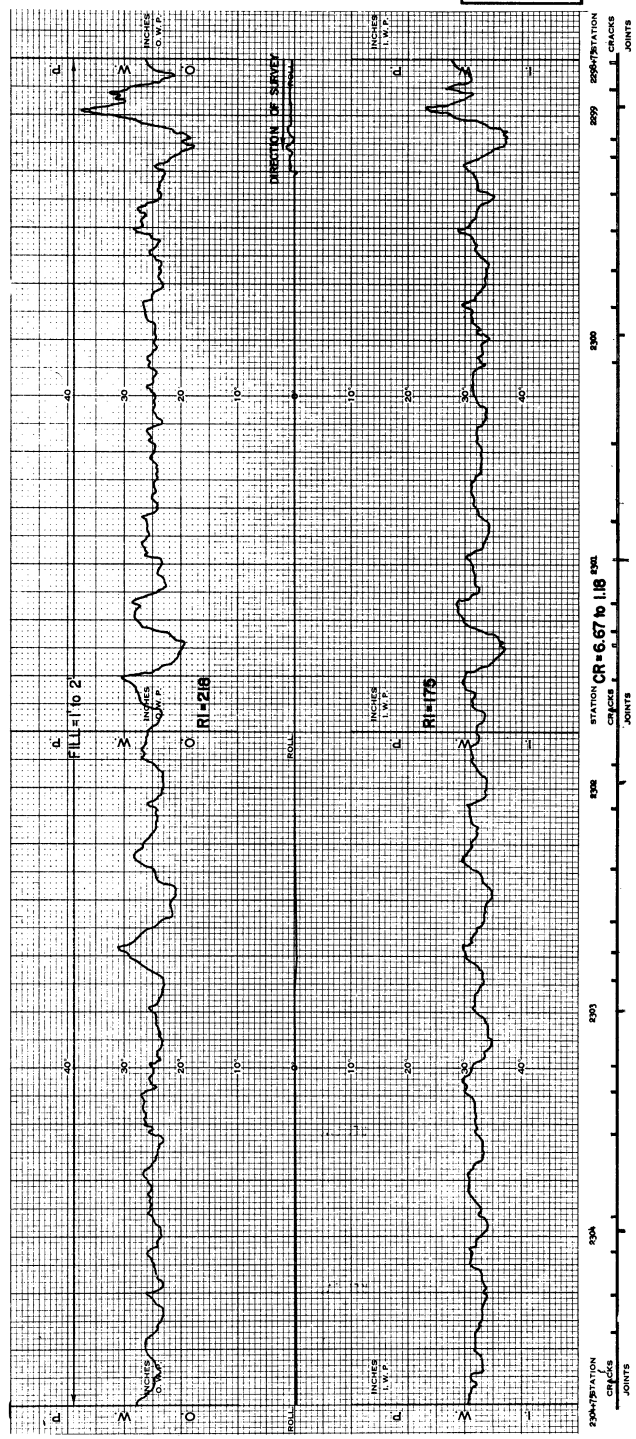
FIGURE-7 B
 CLASS-IV RIGID
 US-112, 1926 SERVICE 32 YEARS
 TRAFFIC TOTAL 4300, COMMERCIAL 1400, DHV 972
 DATE OF SURVEY AUGUST 4, 1958





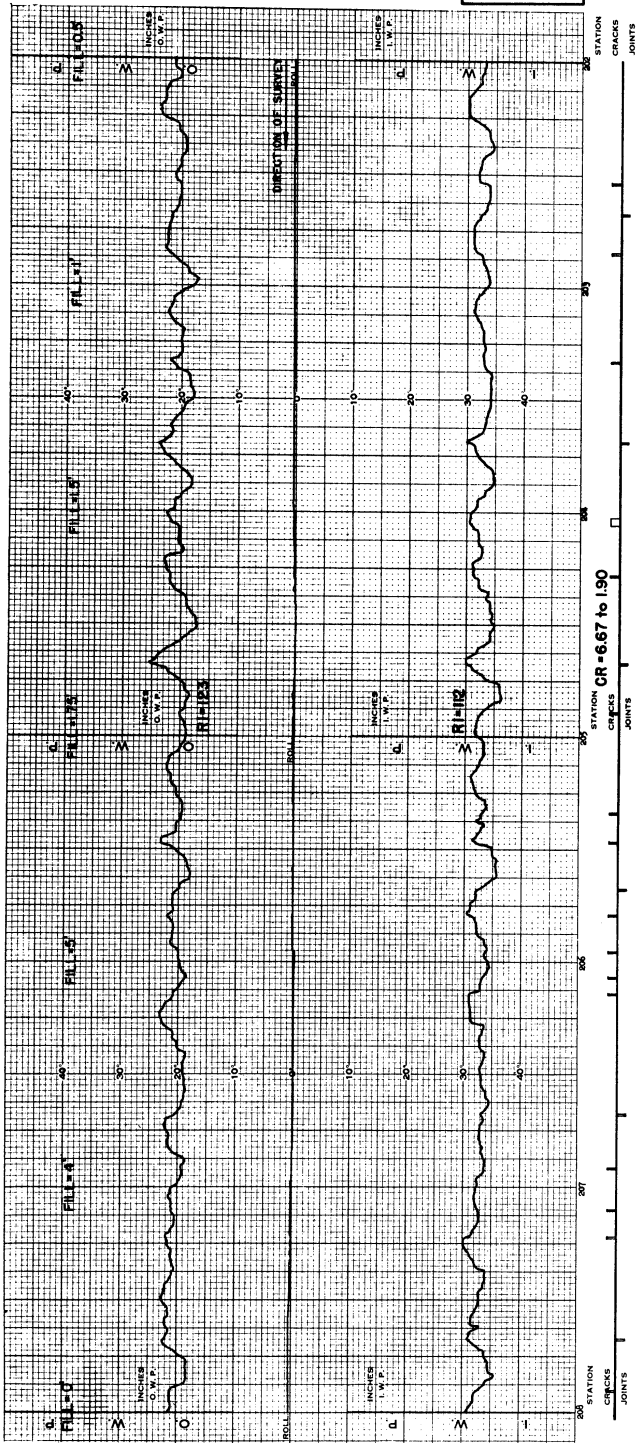
DRAINAGE: EXCELLENT

FIGURE - 8 A
 CLASS - I RIGID
 M - 25 , 1930-31 SERVICE 28 YEARS
 TRAFFIC: TOTAL 700, COMMERCIAL 50, DHV 140-150
 DATE OF SURVEY: JUNE 5, 1959



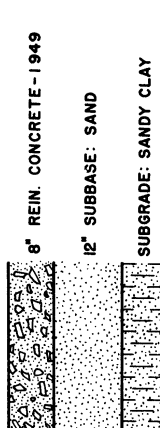
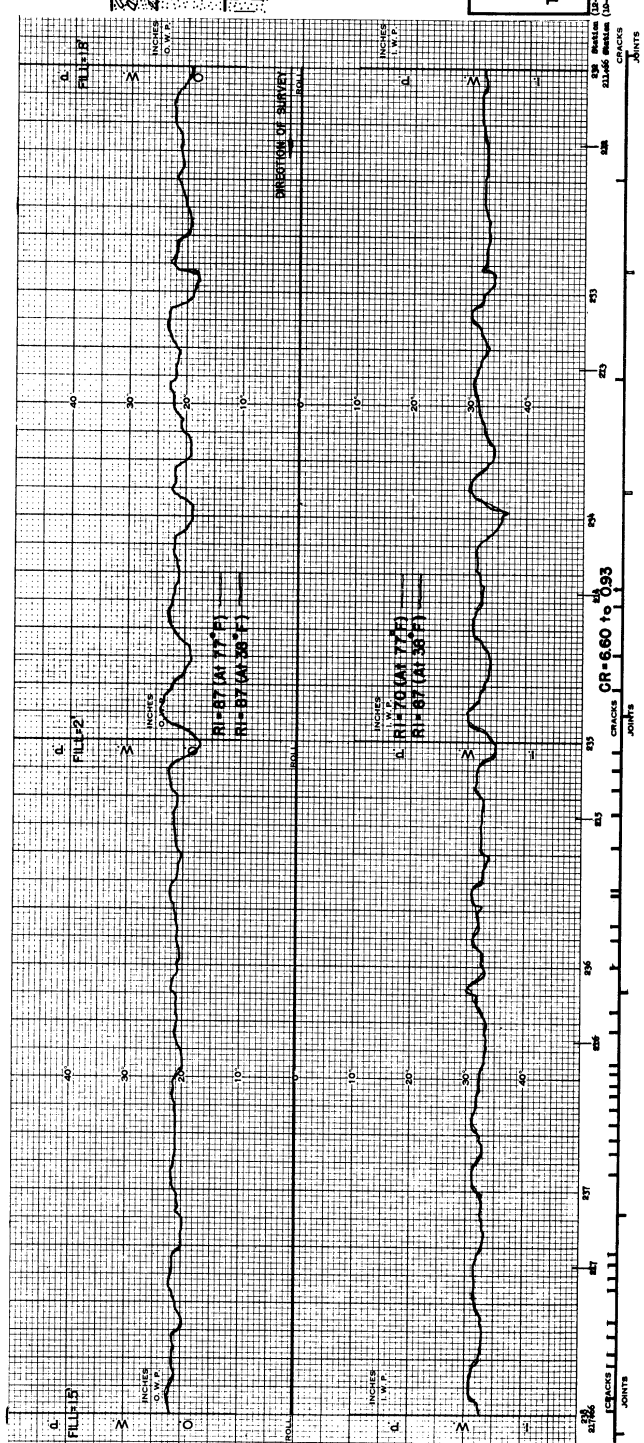
DRAINAGE: POOR

FIGURE - 8 B
 CLASS - IV RIGID
 M - 25 , 1922-26 SERVICE 36 YEARS
 TRAFFIC: TOTAL 1800, COMMERCIAL 310, DHV 350
 DATE OF SURVEY : JUNE 6, 1958



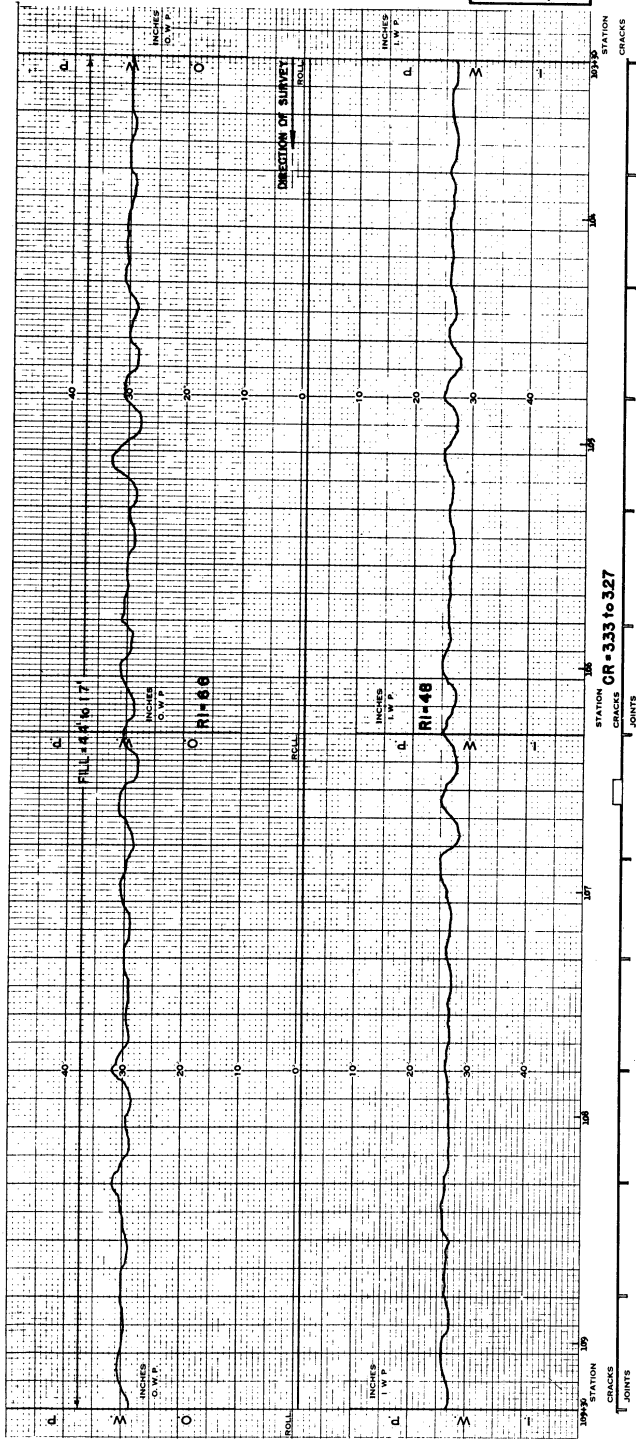
DRAINAGE: EXCELLENT

FIGURE - 9 A
 CLASS - I RIGID
 US - 23, 1930 SERVICE 28 YEARS
 TRAFFIC: TOTAL 8700, COMMERCIAL 1000, DHV 825
 DATE OF SURVEY: DEC. 2, 1958



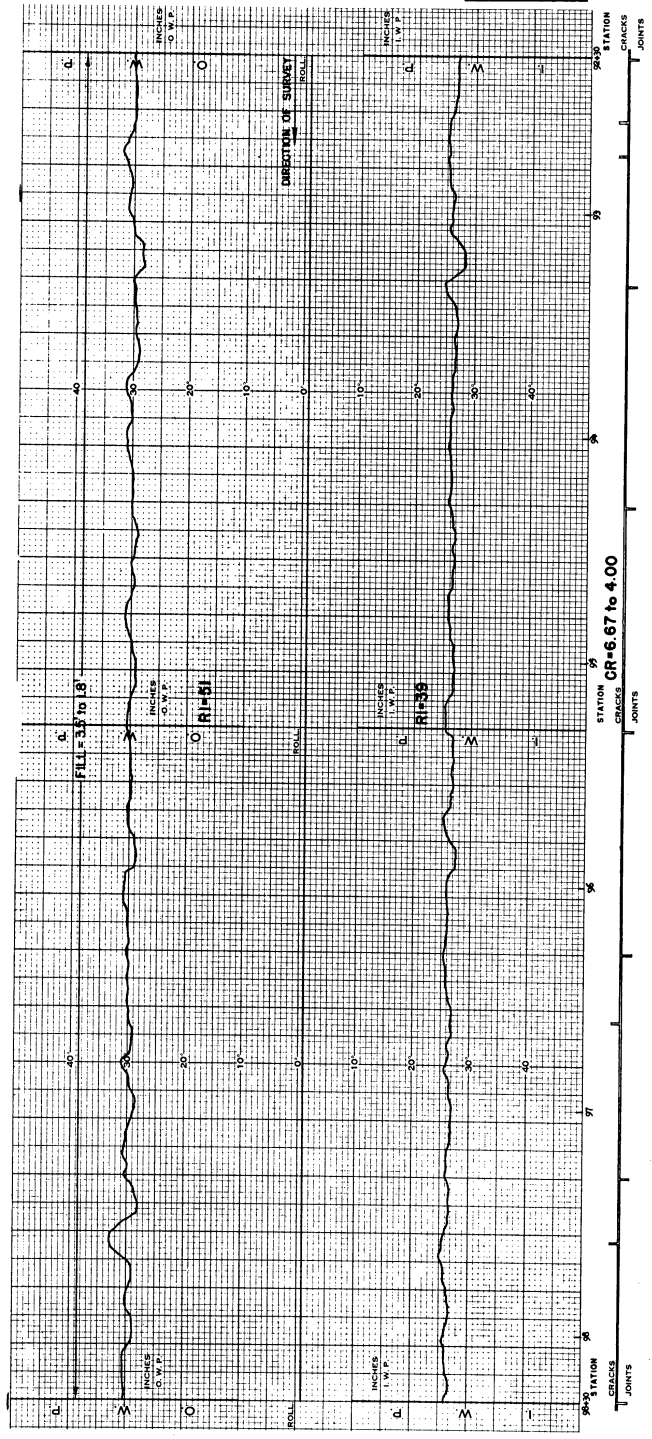
DRAINAGE: GOOD

FIGURE - 9 B
 CLASS - II RIGID
 M - 43, 1949 SERVICE 9 YEARS
 TRAFFIC: TOTAL 1500, COMMERCIAL 225, DHV 300
 DATE OF SURVEY: OCT. 7, 1958 & DEC. 4, 1958



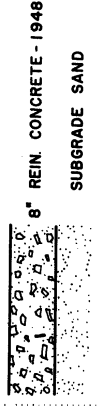
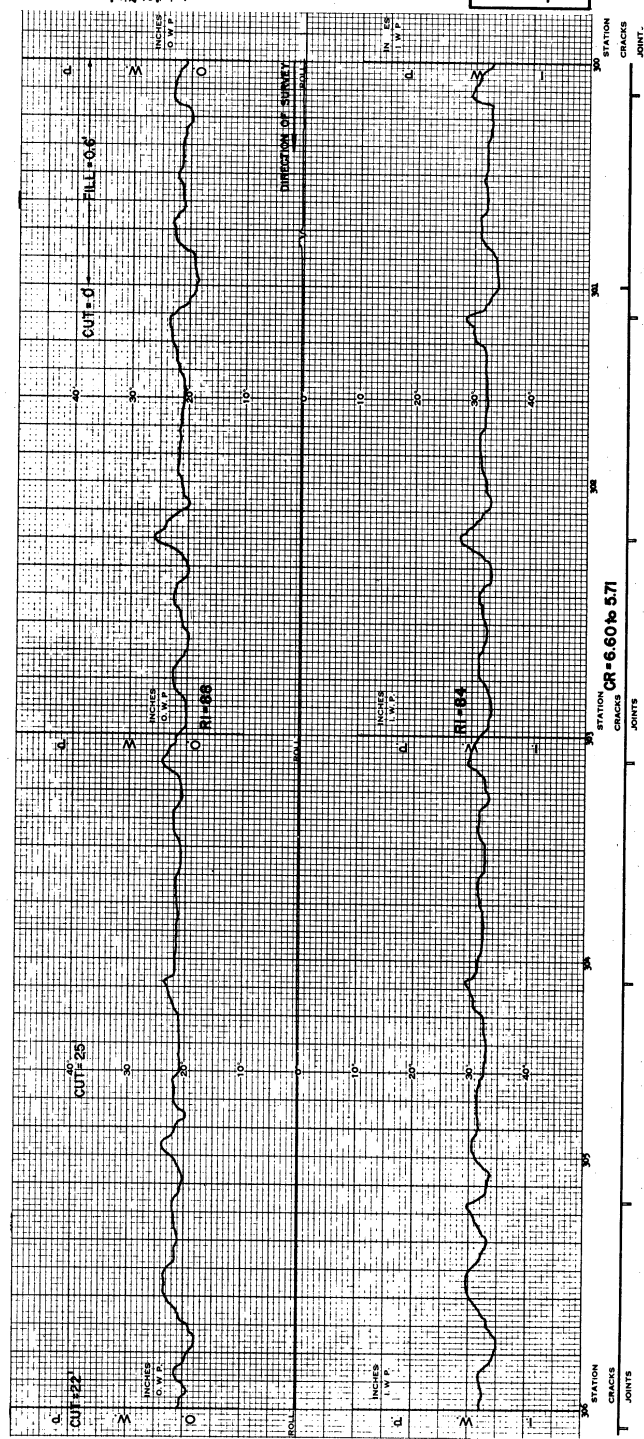
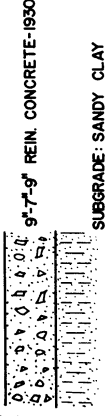
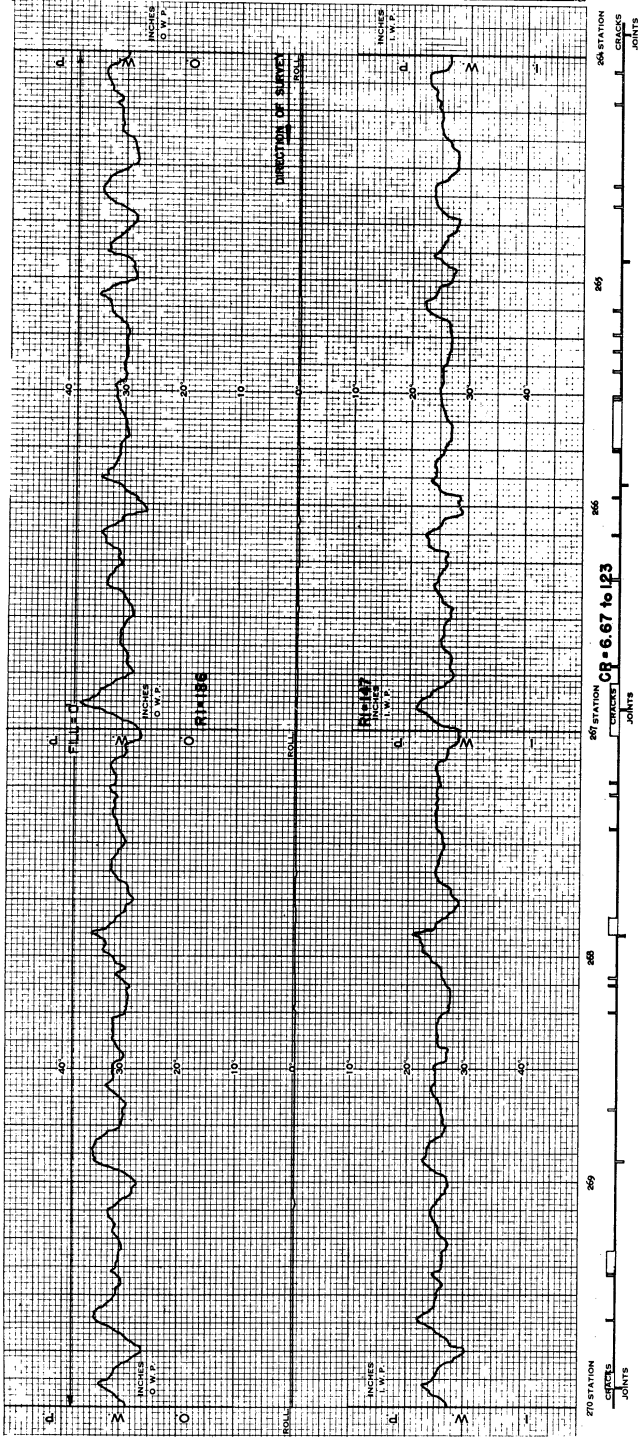
8" PLAIN CONCRETE - 1947
 12" SUBBASE: SAND
 SUBGRADE: SANDY CLAY
 DRAINAGE: GOOD

FIGURE -10 A
 CLASS-II RIGID
 M-43, 1947 SERVICE II YEARS
 TRAFFIC: TOTAL 1500-3300, COMMERCIAL 280, DHV 400
 DATE OF SURVEY: OCT. 7, 1958



8" REIN. CONCRETE-1947
 12" SUBBASE: SAND
 SUBGRADE: SANDY CLAY
 DRAINAGE: GOOD

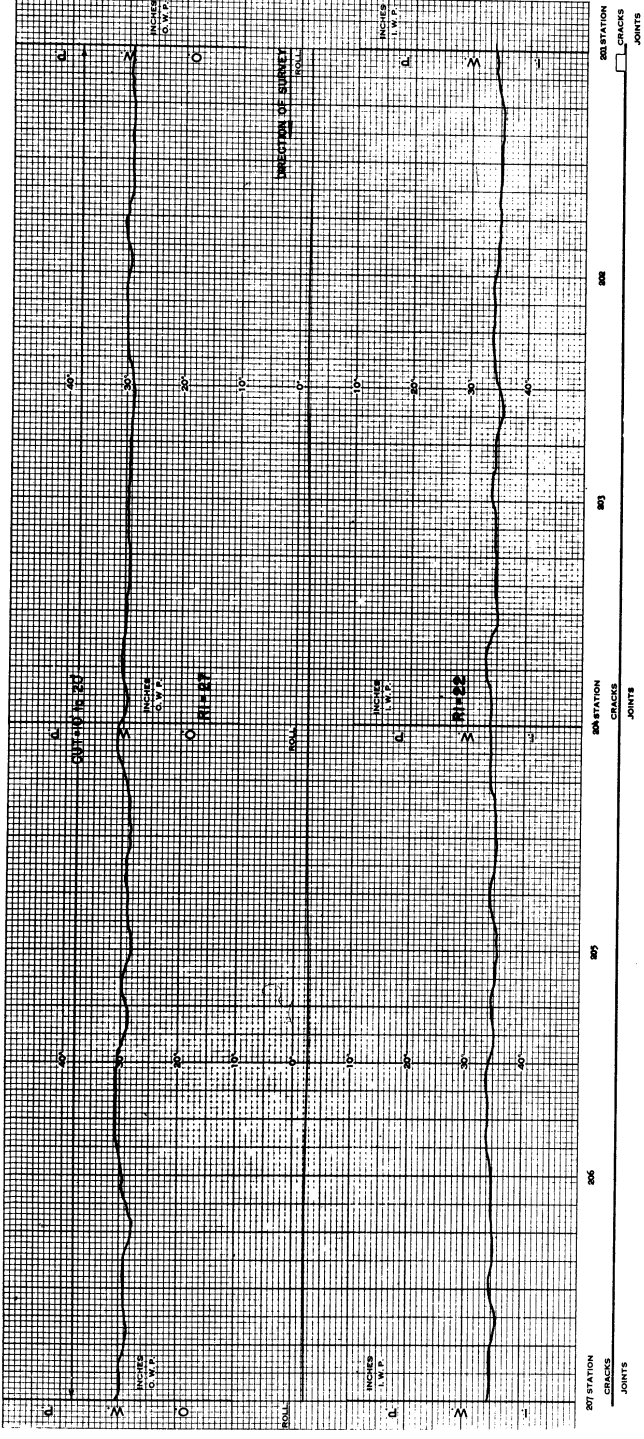
FIGURE -10 B
 CLASS-II RIGID
 M-43, 1947 SERVICE II YEARS
 TRAFFIC: TOTAL 1500-3300, COMMERCIAL 280, DHV 400
 DATE OF SURVEY: OCT. 7, 1958



2 1/2" BITUMINOUS AGGREGATE-1955
 1/4" BITUMINOUS SEAL COAT-1954
 7" GRAVEL BASE (22A)-1954
 3"-5" GRAVEL
 SUBGRADE: SAND

DRAINAGE: EXCELLENT

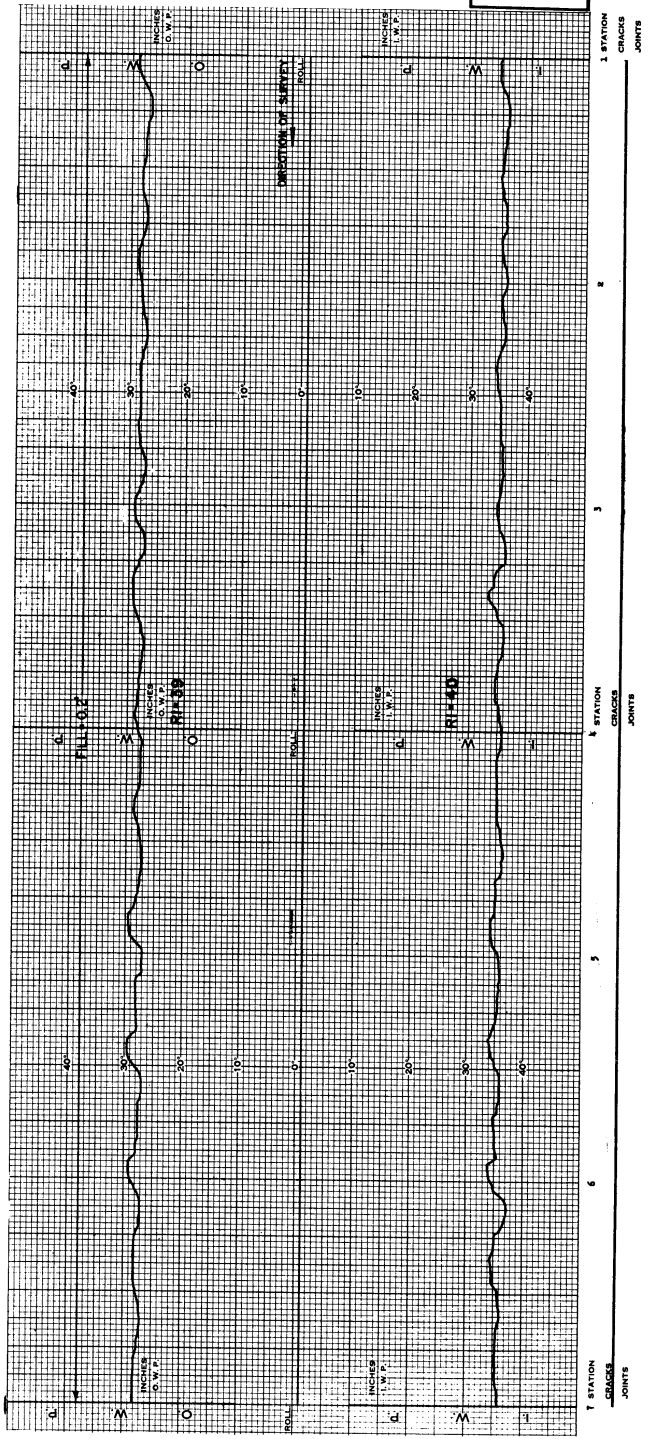
FIGURE-13A
 CLASS-I FLEXIBLE
 M-37, 1955 SERVICE 3 YEARS
 TRAFFIC: TOTAL 500, COMMERCIAL 120, DHV 95
 DATE OF SURVEY: SEPT. 11, 1958

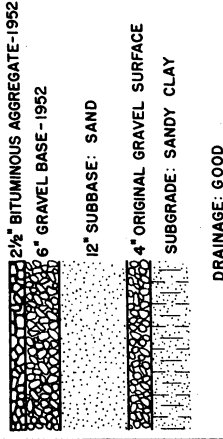


2 1/2" BITUMINOUS CONCRETE-1956
 2 1/2" BITUMINOUS CONCRETE-1936
 8" GRAVEL-1932
 2" EARTH FILL
 SUBGRADE: SAND

DRAINAGE: EXCELLENT

FIGURE-13B
 CLASS-I FLEXIBLE
 M-55, 1936 SERVICE 22 YEARS
 TRAFFIC: TOTAL 900, COMMERCIAL 200, DHV 125
 DATE OF SURVEY: SEPT. 12, 1958





DRAINAGE: GOOD

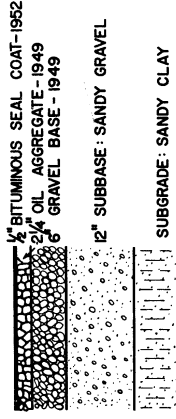
FIGURE-14 A

CLASS-II FLEXIBLE

M-57, 1952 SERVICE 6 YEARS

TRAFFIC: TOTAL 1000, COMMERCIAL 200, DHV 130

DATE OF SURVEY: DEC. 5, 1958



DRAINAGE: FAIR

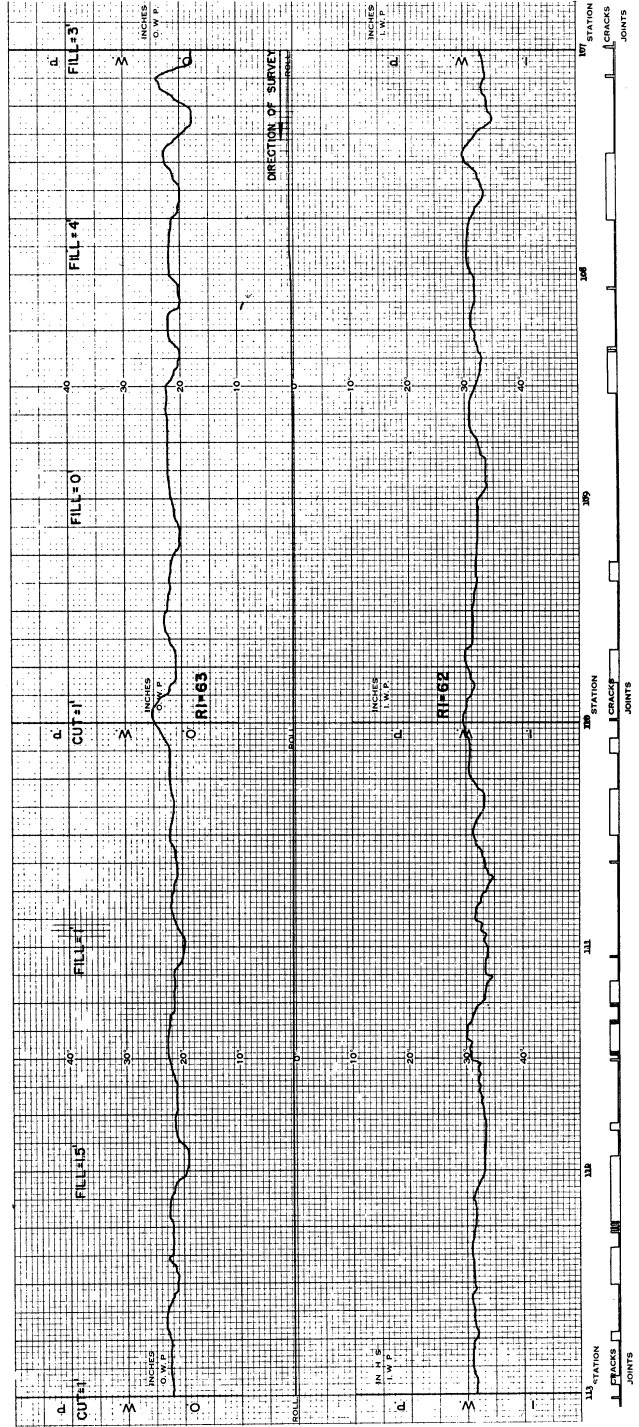
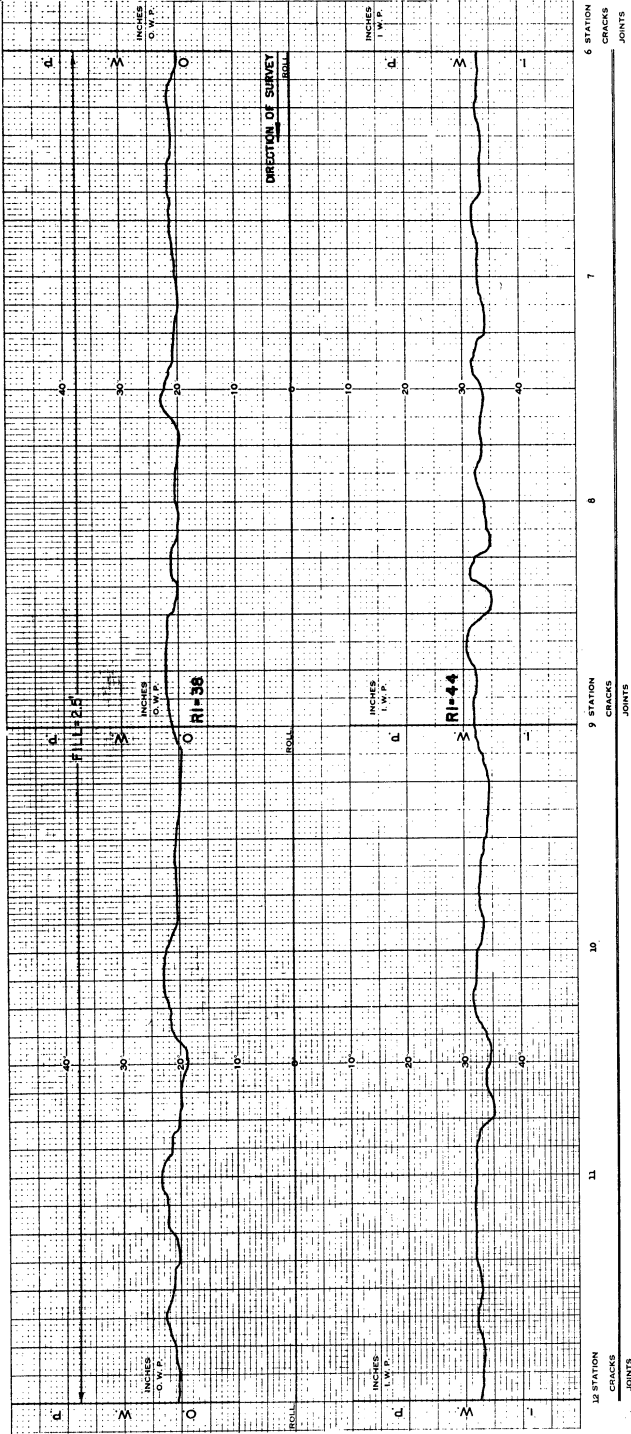
FIGURE-14 B

CLASS-II FLEXIBLE

M-50, 1949 SERVICE 9 YEARS

TRAFFIC: TOTAL 2400, COMMERCIAL 300, DHV 195

DATE OF SURVEY: DEC. 4, 1958



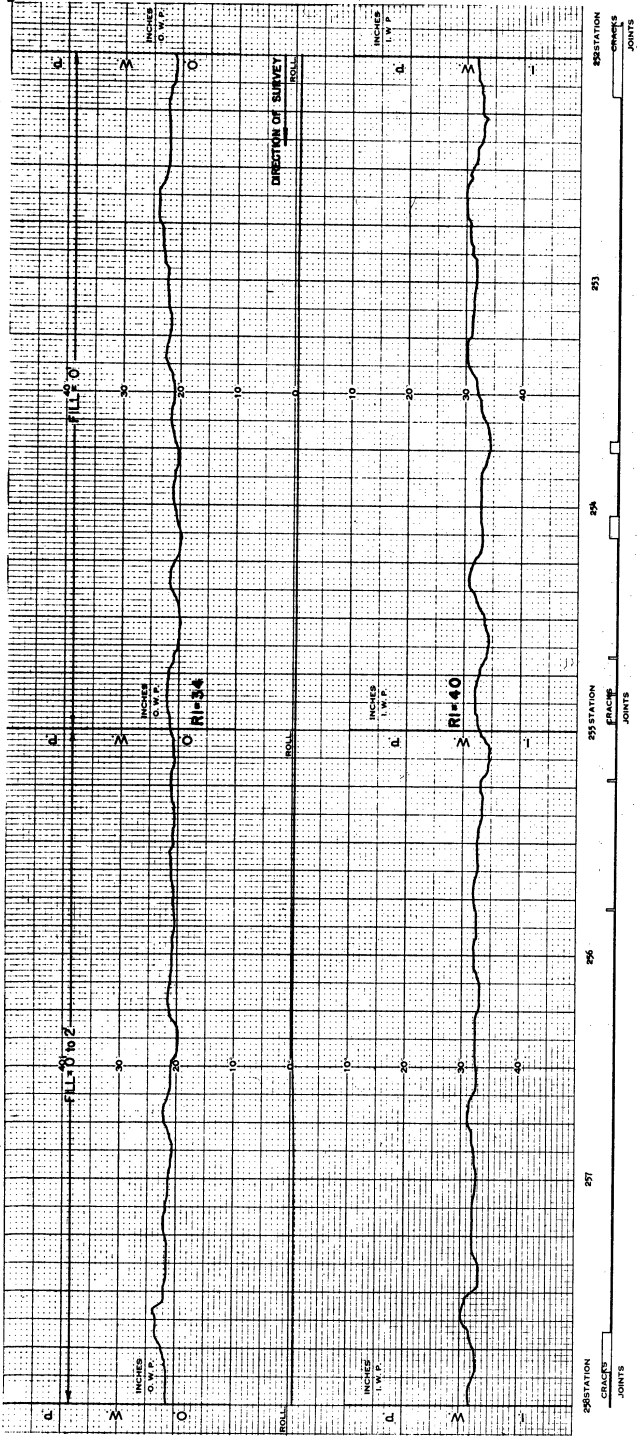


FIGURE-15 A
CLASS-III FLEXIBLE
M-100, 1953 SERVICE 5 YEARS
TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
DATE OF SURVEY: DEC. 4, 1958

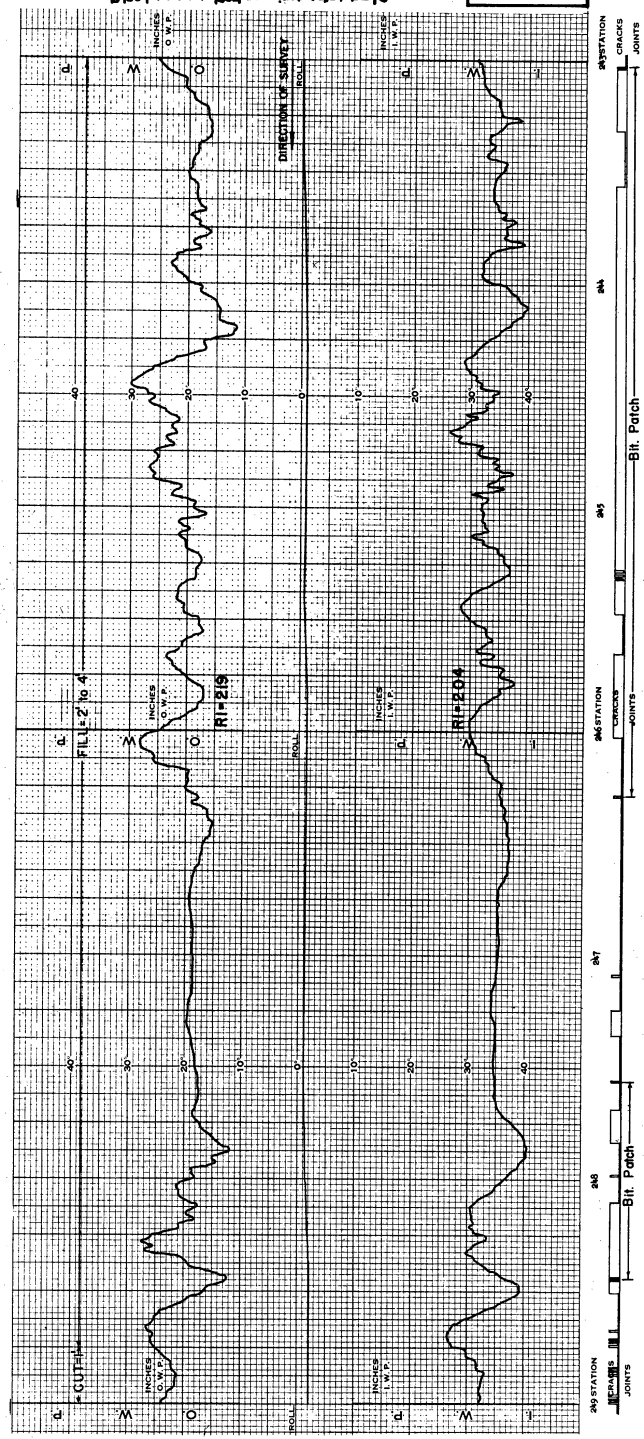
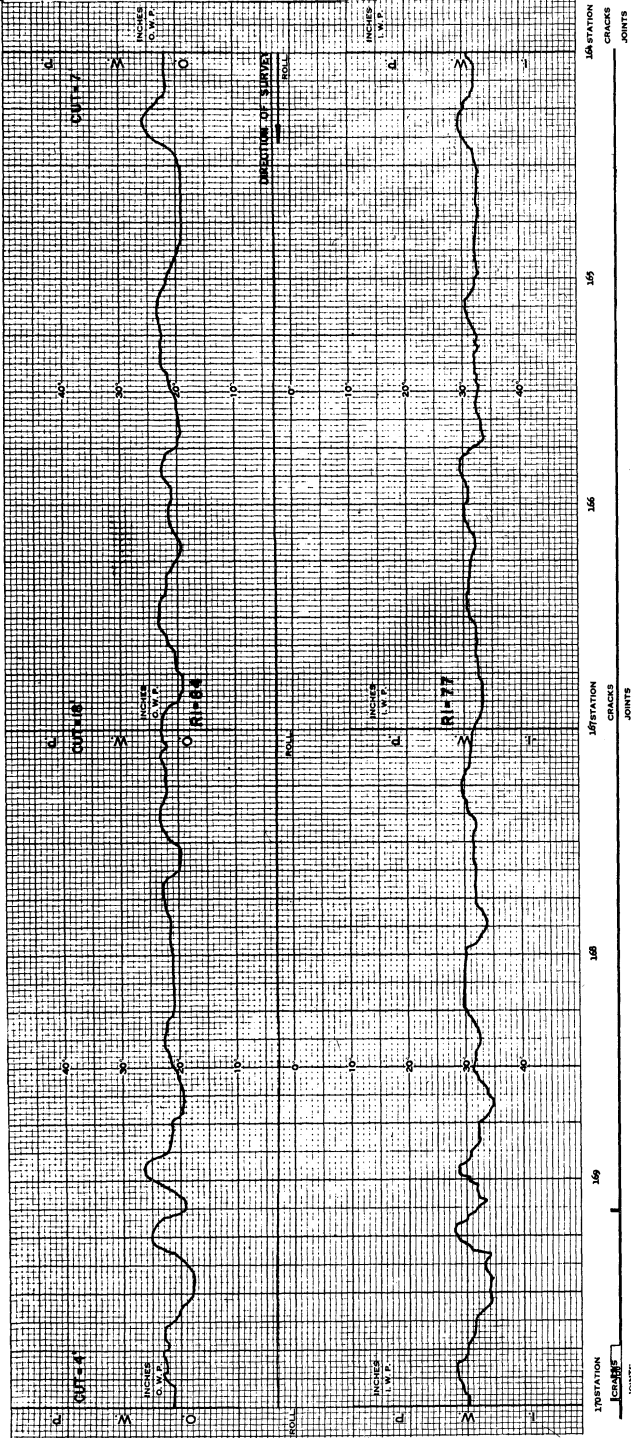


FIGURE-15 B
CLASS-III FLEXIBLE
M-100, 1953 SERVICE 5 YEARS
TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
DATE OF SURVEY: DEC. 4, 1958

1/2" SURFACE TREATMENT-1950 & 54
 2 1/2" OIL AGGREGATE-1939
 8 1/2" SUBBASE: GRAVEL
 SUBGRADE: SAND

DRAINAGE: EXCELLENT

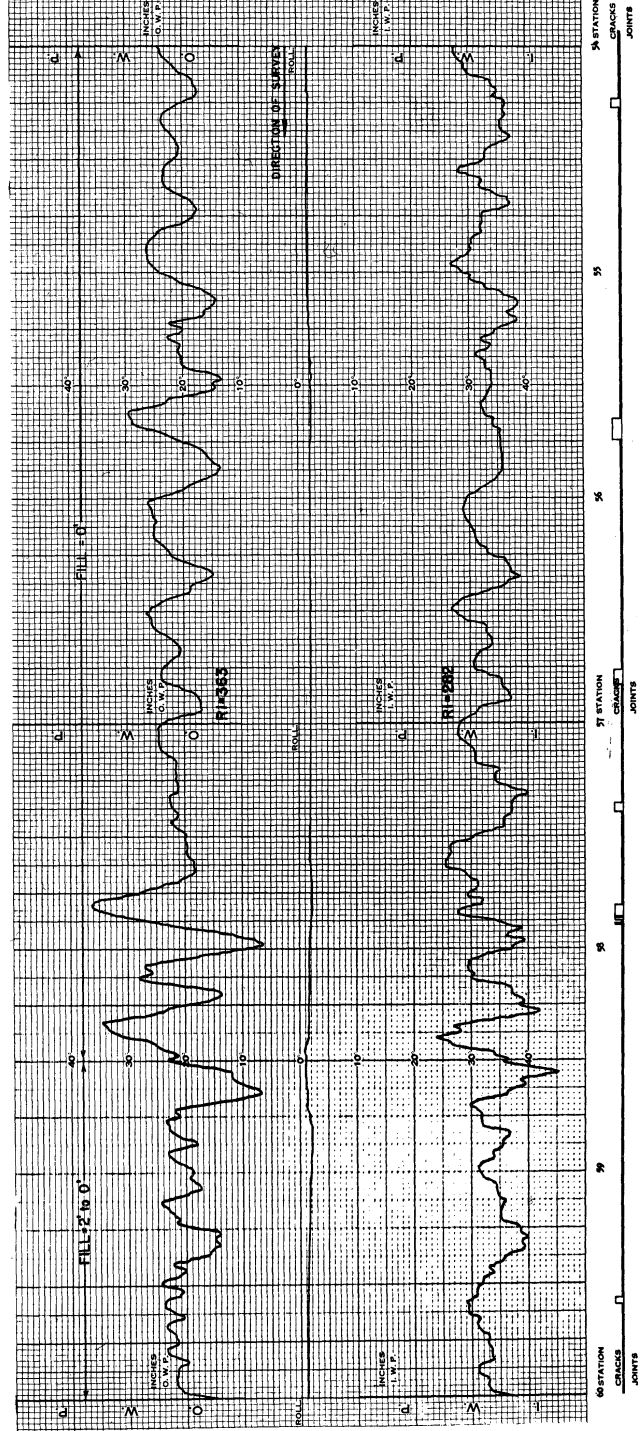
FIGURE-16 A
 CLASS-III FLEXIBLE
 M-36, 1939 SERVICE 19 YEARS
 TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
 DATE OF SURVEY: DEC. 5, 1958



1/2" SURFACE TREATMENT-1950 & 54
 2 1/2" OIL AGGREGATE-1939
 11" SUBBASE: GRAVEL
 SUBGRADE: SANDY CLAY

DRAINAGE: POOR

FIGURE-16 B
 CLASS-IV FLEXIBLE
 M-36, 1939 SERVICE 19 YEARS
 TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
 DATE OF SURVEY: DEC. 5, 1958



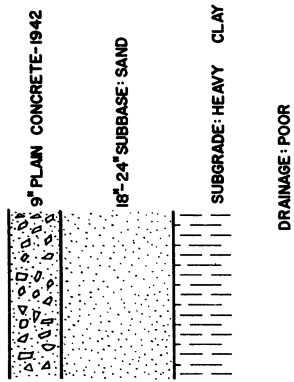
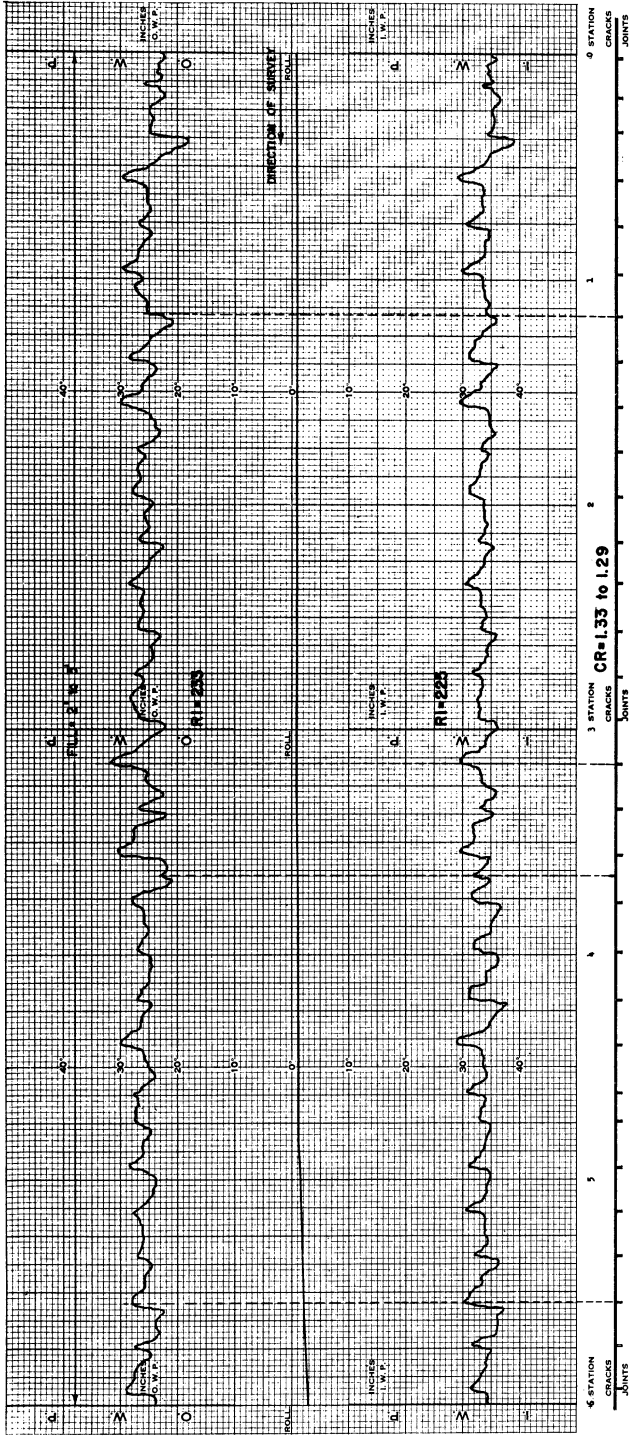


FIGURE-17
 CLASS-II RIGID
 US-24 A, 1942 SERVICE 16 YEARS
 TRAFFIC: TOTAL 10,000, COMMERCIAL 3025, DHV 575
 DATE OF SURVEY: JUNE 19, 1958

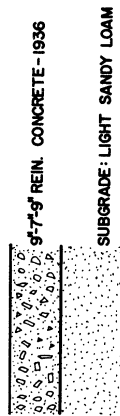
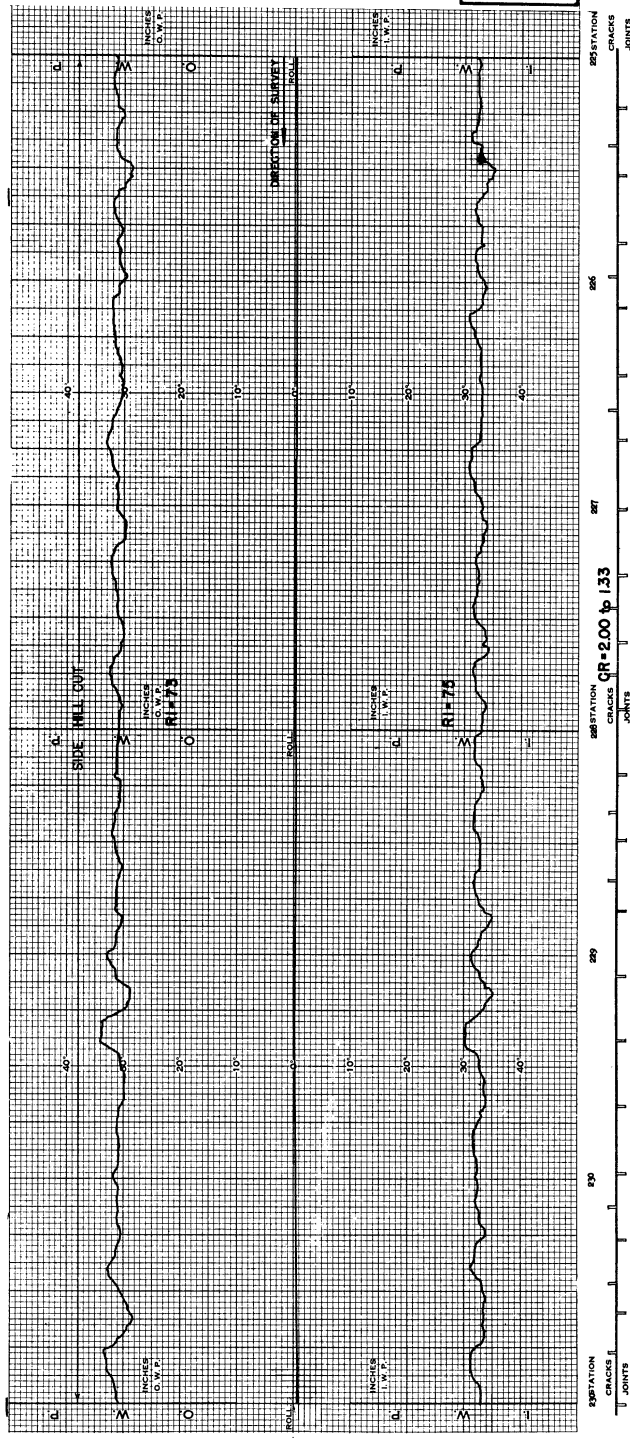
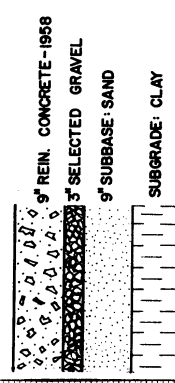
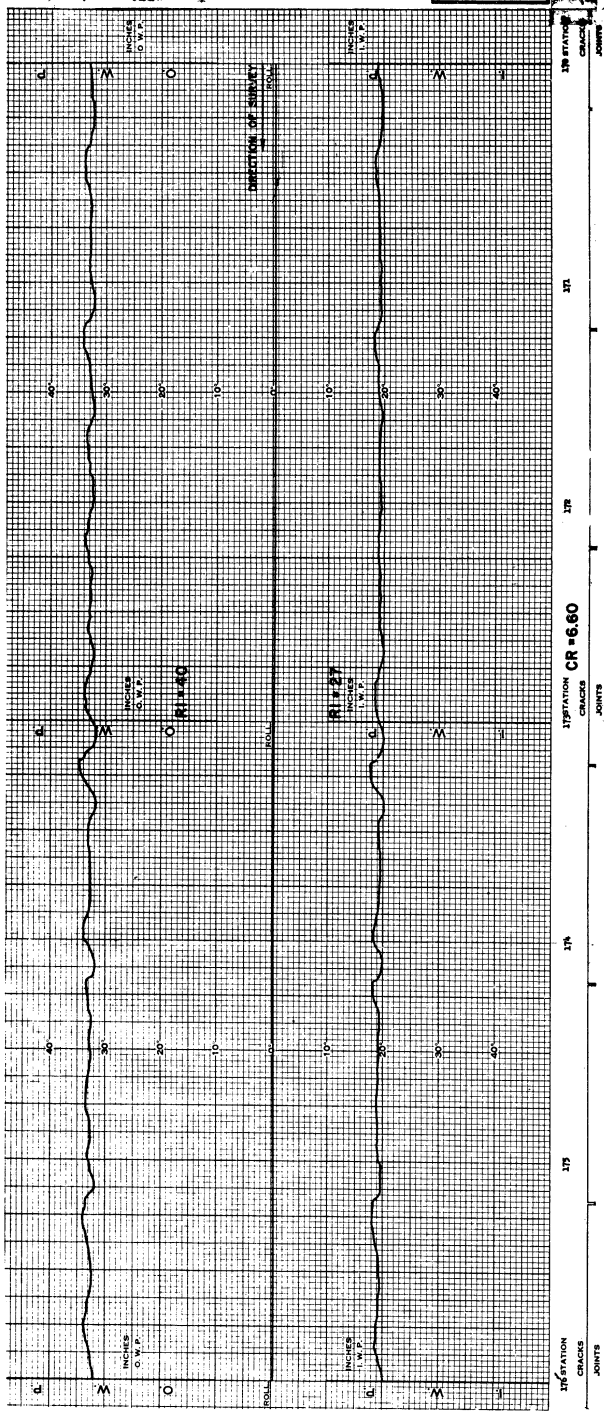
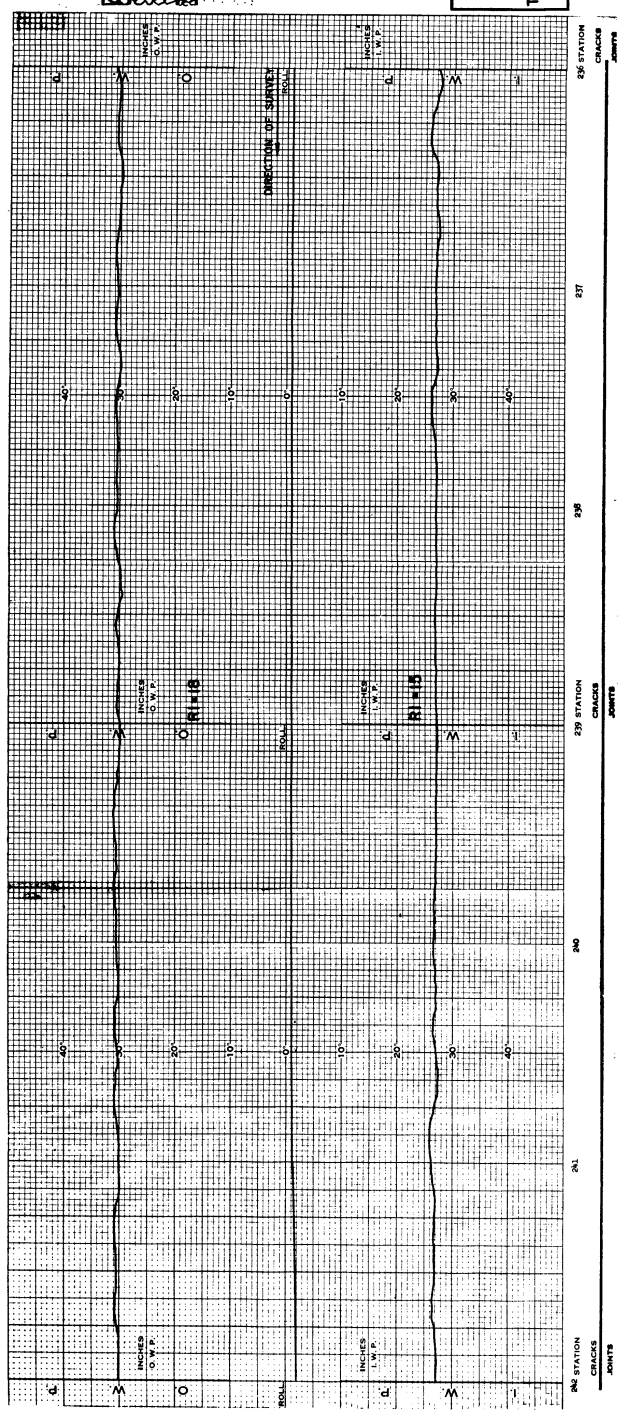


FIGURE-12 B
 CLASS-III RIGID
 US-41, 1936 SERVICE 22 YEARS
 TRAFFIC: TOTAL 1500, COMMERCIAL 280, DHV 190
 DATE OF SURVEY: NOV. 7, 1958



DRAINAGE: FAIR

FIGURE-18 A
CLASS - II RIGID
 M-20, 1958 NEW PAVEMENT
 TRAFFIC: TOTAL 5000, COMMERCIAL 875, DHV 785
 DATE OF SURVEY: NOV. 18, 1958



DRAINAGE: EXCELLENT

FIGURE-18 B
CLASS - I FLEXIBLE
 US-31, 1958 NEW PAVEMENT
 TRAFFIC: TOTAL 8200, COMMERCIAL 1625, DHV 925
 DATE OF SURVEY: SEPT. 25, 1958

DEPARTMENT OF CIVIL ENGINEERING
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LEGAL WEIGHT LIMITATIONS ON MOTOR VEHICLES

BY

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PRESENTED

AT

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LEGAL WEIGHT LIMITATIONS ON MOTOR VEHICLES

by

William S. Housel*

The subject assigned to the writer and his collaborator, Mr. Evans, for discussion at this conference was understood to have particular reference to the current policy of the Michigan State Highway Department to remove load restrictions on a selected network of state trunk lines during the Spring thaw. This discussion also includes the probable results of this practice in 1959, after the unusually severe winter which Michigan has just experienced. As the writer sees it, the subject might better be weather vs. traffic as factors in pavement performance. The object of this approach is to place weather as an environmental factor in its proper perspective with the other set of important factors in pavement design usually included under the general term of traffic. Traffic, of course, must be expanded to include at least both the volume of traffic and the axle loads for which the pavements are or should be designed. Weather, likewise, is only the most obvious of several directly related environmental factors, the other most important factors being soil conditions and drainage.

The title and original assignment fails to emphasize that weather may be the more dominant factor, and that this is the case is becoming increasingly apparent as the evidence accumulates after the most severe winter in Michigan which many of us can remember. The controlling influence of weather in concert with other phases of environment is the foundation of Michigan pavement design, as pointed out in many papers presented in the past by Michigan highway engineers at this and other conferences. To correlate the

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effect of environment as defined above is the objective of pavement performance surveys carried on in Michigan for many years, the further development of which is the aim of the Michigan Pavement Performance Surveys now being conducted by the University of Michigan in collaboration with the Michigan State Highway Department.

It is the primary purpose of this paper to develop the subject assigned by presenting some of the interesting data obtained by this survey, to review the experience with raising of Spring load restrictions in 1958, and to discuss briefly the critical problems presented by the continuation of this policy during the same period in 1959, which is just now beginning. Before proceeding with this review, a few comments seem to be in order on the general subject of weather and roads in Michigan and in other places where parallel experiences are being reported. All of this may serve to remind us of some old truths that somehow seem to be forgotten as the emphasis has shifted from the art to the so-called science of road building. A case in point is the often quoted statement of former Commissioner of the Bureau of Public Roads, Thomas M. MacDonald, that "The roads are more destroyed really by climatic and soil conditions than they are by any use that is made of them."

The excessive damage to Michigan roads during the winter months that are just passing has been almost exclusively the result of alternating rain and thawing followed by severe freezing. Recapped concrete pavements, patches, joint and crack filling, light surface treatments, and various types of stage construction are the ones that have been most affected. Every type of construction which permitted the infiltration of water, which was subsequently frozen by the abnormal periods of low temperature, has suffered severely.

This may be termed surface damage, though none the less severe in destroying riding quality and necessitating expensive maintenance. The worst of this situation is that the maintenance is no more permanent than the original patching and will be lost just as rapidly in any subsequent period of severe weather conditions.

The next phase of winter damage that could be even more serious is the aftermath of the Spring thaw, when the loss of subgrade support and more basic structural damage is a possible result which is still hanging in the balance. The fact that the Michigan State Highway Department has enlarged the so-called "frost-free network" in 1959 to include important connecting roads not freed of restrictions in 1958 should not be construed as taking lightly the severe winter and the calculated risk of exposing additional substandard roads to the heavier legal axle loads. This is a problem that has caused a great deal of concern on the part of everyone involved in the raising of load restrictions. Maintenance forces and field engineers in all parts of the state have been alerted to keep close watch on all roads in the network during this critical period, with particular attention to weak sections of lower classification. Instructions have been issued to prepare for heavier maintenance, and to relay to the Department in Lansing all evidences of unanticipated damage. Commercial carriers have likewise been asked for their complete cooperation and warned that unfavorable weather during the Spring thaw may necessitate placing restrictions back in force for at least limited periods of time.

So far the weather has been quite favorable during the transition from winter to spring, with a very gradual change to warmer weather and without excessive precipitation in the area affected to date in the southern part of

the state. If similar conditions are experienced throughout **the state during** the next few weeks, the results may not be as serious as many **have feared and** the program of raising restrictions may be carried through with **results as** favorable as they were in 1958. It is hoped that this will be the case, as the stakes are high.

From figures gathered by the Michigan State Highway Department last year, it was estimated that Spring restrictions increased the transportation cost to Michigan industry and agriculture by some \$20,000,000. The provision of an unrestricted network last year has been credited with saving a substantial part of this sum for state industries. The results were gratifying to those directly interested in this policy and its related research program. There was little if any evidence of damage due to the 25 per cent increase due to full legal axle loads, and only a few miles of abnormally weak roads had to be taken off the frost-free network. Last year the total network of unrestricted roads was approximately 4600 miles, or a little less than 50 per cent of the state trunk line mileage. This year the network was increased to approximately 5785 miles, or a little more than 60 per cent of the trunk line mileage. This potential increase in the already substantial benefits noted above as even the indirect result of the pavement performance research program places the latter in the category of productive research of more than ordinary importance.

In this connection, the writer has been a little puzzled by the lack of public interest in the benefits of the raising of load restrictions on commercial transportation. In a state where the climate for industry is purported to be unfavorable to industry, coupled with the critical financial condition of the state and local governments, one would think that a real and immediate savings of the magnitude of \$20,000,000 would be worthy of some notice. At least it would provide public officials with a strong argument in presenting the obvious need for increased taxes to support highway construction with such

a positive contribution to our industrial economy. Yet when the expanded unrestricted network for 1959 was released by the Michigan State Highway Department and the 1959 maps circulated, the writer failed to find any notice given to the policy in the metropolitan area newspapers and heard of only one short article on the release in the Lansing State Journal.

Turning to other subjects, and before presenting some of the pavement survey results, some attention may be given to other experience on the dominant influence of weather on pavement performance. A recent article in the Chicago Tribune reported some unhappy experience of this sort in England under the heading "Bit of Winter Ruins Britain's First Superroad". The article goes on to paint a rather sorry picture of the closing of this road a few weeks after it had been opened and describes it as a fiasco of road construction. One might suspect this to be an example of spectacular news reporting which the facts would reveal as greatly exaggerated.

However, a more objective account of the subsequent investigation reported in the February issue of Roads and Road Construction, an authoritative English road magazine, indicates that there were serious oversights in the design and construction of the "Preston By-Pass" which ignored the effects of moisture, climate, and soil conditions. The relatively small percentage of early failure in terms of square yards of total paving still presented evidence of fundamental weakness that over longer periods of time would probably have seriously impaired sufficiently greater portions of road to have rendered it an extensive failure. Here again the experience brings to mind another belabored quotation that is none the less a basic truth taken from the classic report by John McAdam in 1823 as follows:

"The roads can never be rendered thus perfectly secure until the following principles be fully understood, admitted and acted upon, namely, that it is the native soil which really supports the weight of traffic, that while it is preserved

in a dry state it will carry any weight without sinking and that it does, in fact, carry the road and the carriage also, that this native soil must previously be made quite dry and the covering, impenetrable to rain, must then be placed over it to preserve it in that dry state, that the thickness of a road should only be regulated by the quantity of material necessary to form such impervious covering and never by any reference to its own power of carrying weight."

"The erroneous opinion so long acted upon and so tenaciously adhered to -- that a road may be made sufficiently strong artificially to carry heavy carriages though the subsoil be in a wet state and by any such means to avert the inconveniences of the natural soil receiving water from rain or other causes has produced most of the defects of the roads of Great Britain."

TYPICAL PAVEMENT PROFILES

The primary objective of this report is to present recorded data from projects selected from more than 1900 miles of available profiles to illustrate the various conditions of service for which the Michigan design standards have been formulated. Examples of pavements have been selected in all four classifications of adequacy and their performance measured in terms of the roughness index and the structural continuity where applicable. In this way, it may first be possible to find significant evidence on the relative importance of soil type, drainage, frost action, and other environmental conditions in pavement performance as compared to the effect of vehicle size and weight and traffic volume.

In the second place, the examples should provide evidence of the reliability and selectivity of pavement profiles as a means of differentiating between adequate and inadequate roads on the basis of service behavior. The data should also provide some idea of the validity of pavement roughness and structural continuity as measures of pavement adequacy, and further show their value as a means of identifying the factors responsible for variations in behavior.

In presenting these pavement profiles as examples of the direct correlation between pavement performance and adequacy to carry legal axle loads, some reference must be made to the Michigan method of pavement evaluation. This and other background data have been presented in previous publications ^{1,2,3}, but some of the essential information must be reviewed briefly in this discussion. The truck-mounted profilometer, its operation, and the type data that it provides, may be obtained from a previous report ¹. The method of rating the adequacy of Michigan pavements is so essential to the present discussion that it will be repeated as presented in the "First Pavement Evaluation of January 1, 1958".

This first state-wide evaluation provides the foundation for future surveys, and will be the basis for reclassification of highways and for future improvements that will be made from time to time. In this road classification for legal axle loads, four levels of adequacy have been selected which will subsequently be referred to as Class I, II, III, or IV, and are defined as follows, with colors referring to the appended Pavement Evaluation Map.

Class I (Solid black)

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

Class II (Dot-dash black)

No seasonal restrictions; pavement designs which compensate for seasonal loss of strength as represented by subgrades of

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- ¹ Pavement Profile Surveys to Correlate Michigan Design Practice with Service Behavior, by William S. Housel and Olaf L. Stokstad, January, 1959.
 - ² Highway Engineering -- Pavement Performance as Related to Design, by William S. Housel, Proceedings of the Fortieth Annual Michigan Highway Conference, 1955.
 - ³ Highway Loads and How They Are Carried, by William S. Housel, Proceedings of the Thirty-Fifth Annual Highway Conference, 1950.

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 III (Dot-dash red)

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 IV (Solid red)

Spring load restrictions required; pavement design 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 first pavement evaluation brought together a great volume of basic design data available in the Department's records, as well as other pertinent information on soil types and drainage, maintenance experience, and pavement performance data going back to the previously mentioned condition surveys of many years ago. Through the collaboration of the District Soil Engineers and other Department personnel, much of this classification reflects the unrecorded knowledge of the men who have lived with these roads for many years. The pavement evaluation map brings together this great store of working knowledge and, for the first time, presents it in a completely integrated form as a matter of definite record. It will thus provide a foundation and starting point for future reclassification in terms of pavement adequacy and provide a basis for future improvements in the highway system.

Pavement performance surveys which are now in progress have as their objective more precise measurement of pavement adequacy leading to improvement in the accuracy of this first classification. It is expected that weak sections will be found in roads now classified as adequate, and strong sections in those classified as inadequate. In the case of weak sections, more extensive field investigations will be made to pinpoint the sources of weakness and

determine the most practicable corrective measures. This leads directly to the reconstruction or betterment of such roads, and already preliminary study of the problems involved has led to including several strategic sections in early construction programs. Guided by experience from previous surveys, the methods of measuring pavement condition are predicated on the idea that the ability of a pavement to carry the loads of present day traffic is shown by the two types of pavement behavior which, in inadequate pavements, results in progressive changes in pavement cracking and pavement roughness.

With these introductory remarks, attention may now be given to the examples selected for illustration. In reviewing these examples, there are several things to be kept in mind as a means of correlating the data on some comparative basis. In Fig. 1 is shown a tentative roughness rating to associate with the measured roughness indexes as representative of varying degrees of riding quality from exceptionally smooth to extremely rough. Roughness indexes in excess of 200 inches per mile are considered unacceptable, though there are many examples of such roads in service. For those familiar with the more widely used single-wheel roughometer, comparative roughness indexes have been established on the basis of a relatively small number of projects where both types of equipment have been used.

Pavement profiles are presented in a series of charts that follow a definite format, which will be explained at the outset with no further reference thereafter. Each chart presents 600 feet of profile in the outer and inner wheel paths of the traffic lane. The outer wheel path is generally at the top of the sheet on the right hand side of the lane, with the direction followed by the profile truck generally being to the left. Cut and fill in feet is indicated at the top of the sheet. 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. It should be kept in mind that vertical displacements are recorded full scale (1" = 1"), while the horizontal distances are to a scale of 1" = 25'. This results in a 1 to 300 magnification of vertical displacements, thus exaggerating the roughness in terms of pavement grade or slope.

Pavement joints and cracks are shown along a 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 this connection, it should be noted that the continuity ratio is defined in terms of a standard slab length of 15 feet representing a continuity ratio of 1. Thus an uncracked slab between hundred 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 of 15 feet.

To the right of the pavement profiles are given pertinent data on the pavement section, including pavement type and thickness of various components, subgrade classification, drainage, road classification from the standpoint of adequacy, project identification, traffic based on 1957 surveys, and date of survey.

Rigid Pavements

The first series of charts to be presented are for rigid pavements consisting of Portland cement concrete, both plain and reinforced, and, in the case of some of the older pavements covered by a bituminous surface.

Fig. 2, Example 7, Class I and Class IV, Rigid. The first set of two charts shown in Fig. 2 are taken from one of the older, heavily traveled roads in Michigan, US-112. The pavement profiles in Example 7-A with roughness

indexes of 72 in the outer wheel path and 75 in the inner, would still be rated very good from the standpoint of riding quality. This performance reflects some thirty-two years of service for the original concrete pavement and ten years of service for the bituminous resurfacing laid in 1948. The traffic is 4800 vehicles per day, of which 1525 are commercial type. There are relatively few cracks between the hundred foot joints with a reduction in continuity ratio of something less than 50 per cent since construction in 1926.

The pavement section in Example 7-B, which is only a short distance from the first section with which it is compared, has been in almost continuous trouble since it was built, requiring continuous maintenance with an unusual amount of concrete patches and continued cracking of the old pavement. After having been resurfaced with bituminous concrete in 1943, it developed the rough profile shown by a heavy line with a roughness index of 291 in the outer wheel path and 395 in the inner wheel path. It was resurfaced in 1958 with the pavement profile indicated by the finer line superimposed on the profile before resurfacing. In connection with this resurfacing, it is interesting to note the degree to which slab displacements are ironed out and the riding quality recovered. There is a much greater reduction in continuity ratio in Example 7-B, but it should be noted that these data are not completely satisfactory. In a pavement that has been resurfaced as much as this, it is difficult to locate all of the cracks in the structural slab as many of them do not show through the resurfacing. The extensive amount of concrete patching also has been indicated as joints, producing the irregular spacing of joints that were originally 100 feet apart.

The significant difference in these two sections of pavement lies in the subgrade type and drainage. The pavement giving very good service in Example

7-A falls within a relatively small area of glacial outwash which is predominantly sandy with fair internal drainage. This short stretch of pavement was originally included in a much larger area of Class IV road rated inadequate at all times of the year. Its performance, as indicated by the pavement profile and roughness index, suggested reclassification and this was confirmed by field investigation. The example of Class IV pavement in Example 7-B is also a reclassification on the basis of the pavement profile, this section having been included in an area of Class I pavement due to an error in transposing soil survey data on the pavement evaluation map. While the classification of subgrade soil accompanied by poor drainage as Class IV is obvious, the error was discovered by a study of the pavement profiles taken over this portion of US-112.

Fig. 3, Examples 8-A and 8-B, Class I and Class IV, Rigid. The comparisons shown in Examples 8-A and 8-B again show the dominant influence of subgrade, with good performance over the sand subgrade with excellent drainage and poor performance over the lake bed clay with poor drainage. Other contributing factors may have entered into this difference in behavior in that the good performance is on a reinforced concrete pavement with lighter traffic and a service period of twenty-eight years, as compared to the plain concrete pavement over a heavy lake bed clay with poor drainage and thirty-six years of service under substantially heavier traffic. The poor performance in the latter case is also associated with a marked reduction in continuity ratio, even though the older pavement has the advantage of being laid over an old road of four to five inches of water-bound macadam. It is felt that the extra support of this old road is a factor which has enabled this pavement to last over this long period of years under otherwise rather unfavorable conditions. One thing that may be noted in connection with most of the profile charts is

the fact that the roughness index of the outer wheel path is generally greater than that of the inner wheel path, a condition which may be either built in at the time of construction or the result of weakness along the free edge of the pavement associated with unpaved shoulders.

Fig. 4, Examples 10-A and 10-B, Class II Rigid, Plain and Reinforced Concrete. The two pavement sections shown in Examples 10-A and 10-B provide a direct comparison between plain and reinforced concrete pavements, with other design factors substantially the same. This pavement represents one designed up to Michigan's present standards with 12" sand subbase over a clay subgrade, good drainage, and subjected to fairly heavy traffic. Both sections are in excellent condition and would be rated very good or exceptionally smooth, with the slightly better performance in favor of the reinforced concrete section. Eleven years of service is rather short, but long enough to reveal any serious weakness in the pavement. Structural continuity is still high with the absence of cracking in the plain concrete a function of the 50 foot slab length, as compared to the 100 foot slab length in the reinforced section. In both cases, the high capacity foundation with the 12" well compacted sand subbase is the major factor in the excellent performance of both plain and reinforced sections. The differential in roughness index is not large but is probably a measurable difference, being somewhat more than the probable error in recording roughness although not enough to indicate any marked difference in behavior.

Fig. 5, Examples 12-A and 12-B, Class III, Rigid. The two pavement sections shown in Examples 12-A and 12-B make one of the most interesting comparisons provided by the pavement profiles presented in this report. Both sections are taken from US-41, running north out of Menominee into the Upper Peninsula, and are only a few miles apart. This road, incidentally, is known as one of the

roughest riding pavements in the state, and the two sections selected represent the roughest and the smoothest sections. There is a startling differential in roughness index in these two sections with the rough section entirely outside the rating scale, while the smooth section would be rated good or very good. This contrast is directly the result of a difference in subgrade soil and drainage. The rough section is a swamp fill following peat excavation, with the backfill material being described as a heavy sandy loam. The water table is close to the surface, and in this area of deep frost penetration the combination of circumstances promotes severe frost action. The smooth section, on the other hand, is on a side-hill cut and fill, with the subgrade material being described as a light sandy loam sufficiently granular to supply more adequate internal drainage. This in combination with the side-hill surface drainage has produced very good performance over a service period of 22 years. Similar differentials are shown in the change in structural continuity, although to some extent the small amount of cracking in the good section is the result of the 30 foot joint spacing.

Flexible Pavements

As part of the pavement performance surveys conducted during the past year, it was arranged to obtain typical profiles of the four different classifications of flexible pavement for comparison with the rigid type pavement and to test the applicability of pavement profile surveys and pavement roughness as measures of performance. The next series of examples will, therefore, be devoted to flexible pavements and their performance.

Fig. 6, Examples 13-A and 13-B, Class I, Flexible Pavement. The two pavement sections shown in Examples 13-A and 13-B are representative of high type bituminous construction which, on the basis of performance, is being given increased consideration in the state. Example 13-A has had only a service period

of three years, but the subgrade and structural components of the pavement are so closely similar to the pavement section in Example 13-B that they should give comparable performance over longer periods of time. Both are on superior type subgrades 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 would be rated as exceptionally smooth, indicating excellent construction practice as well as completely adequate 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 foundation are important factors in the superior riding quality that has been produced.

One other special circumstance should be noted; the pavement in Example 13-B with a service period of twenty-two years was resurfaced in 1956 because, over a period of twenty years, surface abrasion and hardening of 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 but, insofar as the structural capacity of the pavement is concerned, there was no indication before resurfacing of loss in load carrying capacity reflected in increased pavement roughness.

Fig. 7, Examples 15-A and 15-B, Class III, Flexible. The two sections of pavement in Examples 15-A and 15-B are taken from the same road, M-100, in a section which has presented something of a problem over the years of increasing

traffic. Because of weak sections in the road it has been rated as Class III, inadequate during the Spring breakup period, but the pavement profile surveys generally indicate that some reclassification would be justified. The addition in 1953 of a 12" subbase and a flexible pavement with a bituminous wearing course over the old road would appear to justify at least a Class II rating, and the performance over a service period of five years with a roughness index of 34 to 40 would, in terms of riding quality, be rated as exceptionally smooth. One of the weak sections that had been the source of difficulty was selected as Example 15-B and represents the poor performance in a comparatively short stretch. The reason for this is obvious as this section was built over a peat swamp under the old road which has never been removed and has resulted in surface subsidence, followed by extensive bituminous patching, a combination which falls far short of producing the satisfactory riding quality. The basic difficulty is permanent and, as long as the unstable peat is not removed, no amount of strengthening of the pavement will correct the defect. However, with the greater portion of this road giving quite satisfactory service, it would appear that reclassification is justified, accepting that the peat swamp will always have to be provided with whatever maintenance is necessary to maintain a usable pavement.

Fig. 8, Examples 16-A and 16-B, Class IV, Flexible. The pavement sections shown in Examples 16-A and 16-B supply another study in contrast of two sections taken from the same road, just a few miles apart. Through this area this road has been rated Class IV, inadequate at all times of the year, on the basis of visual inspection and general performance. It is an old road, having been built over a period of years by County forces and later by the State Highway Department when the road was taken over as a trunk line. The comparison is of interest mainly as a demonstration of the dominant influence of a good sand

subgrade over a comparatively limited stretch as compared to a poorly drained sandy clay subgrade. In the good section, after nineteen years of service, the riding quality is still rated as good; whereas on the weak section, some new words would have to be added to the rating table for adequate description. These two examples would also seem to be another illustration of the value of the pavement profilometer in differentiating between those sections which would eventually have to be rebuilt and those which would not require rebuilding, thus adding a quantitative tool to supplement good engineering judgment.

Special Sections

Four extra examples have been selected to present some special pavement sections for which the profile data has particular interest.

Fig. 9, Examples 17 and 12-B. The pavement in Example 17 is of particular interest because it represents the effect of short 20 foot slabs, without load transfer at the joints, in what is presumably an effort to control pavement cracking. The subgrade was a heavy, lake bed clay with a fill of several feet produced by a 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" sand subbase was added on top of the grade to protect the 9" plain concrete from quite certain pumping action. The small reduction in continuity ratio indicates that the crack control was excellent, but the riding quality produced was still outside the tentative roughness scale. However, the most interesting feature of this pavement is the characteristic saw-tooth pattern that has been produced by the tilting and faulting of the short slabs, illustrating another type of valuable information from pavement profiles. From the standpoint of design, the absence of load transfer at the joints is probably the critical weakness. For purposes of comparison, the results shown in Example 17 can be compared to Example 12-B, the smooth section in US-41, where there are 30 foot

joints but with load transfer provided. Expansion joints at 60 feet are supplied with a trans-lode expansion joint base, and the reinforcing is carried through the 30 foot dummy joints.

Fig. 10, Examples 18-A and 18-B. The two pavement sections in Examples 18-A and 18-B are sections of new pavements constructed in 1958, with profiles that give some measure of the high quality of the pavement construction being produced. Example 18-A is on a divided lane highway, M-20, between Bay City and Midland, with a 12" granular subbase consisting of 9" of sand and 3" of selected gravel used as subgrade stabilization. This pavement would be rated as exceptionally smooth, with roughness indexes of 40 and 27 in the two wheel tracks. Example 18-B is representative of heavy duty flexible pavement constructed on the Muskegon - Grand Haven expressway, and is an example of the type of heavy duty flexible pavement construction proposed for use on the interstate highway system on certain selected projects. The riding quality of the pavement is exceptional, with roughness indexes of 18 and 15 in the two wheel paths. Having obtained the basic profile immediately after the pavements were constructed, these projects will be watched with considerable interest over the next few years and will provide the type of data needed to record the complete history of pavements during their useful period of service.

Frost Heave Profiles

The last figures were selected as being of particular interest in the present discussion and are pavement profiles that have been re-run during the past several weeks to measure the frost heaving that has taken place during the past winter. In each case, the heavier line, generally below the second profile, is the pavement profile run during the late summer or fall when the pavement was presumed to be in its most stable condition. The lighter line, or top profile, is the recent re-run and shows the frost heaving in quantitative terms. The

roughness index below the profiles in each case is for the stable condition and is marked as the "Fall" profile. The roughness index above the profiles is from the recent "Late Winter" profile showing the change presumed to be close to the maximum frost heaving liable to occur.

It should be noted further that increased roughness due to frost heaving is presumed to be largely temporary, at least in the well designed pavement. When the frozen subgrade is thawed and the pavement returns to its normal position some time next summer, any permanent displacement which remains will be the real measure of pavement performance and the adequacy of the construction designed to compensate for the unfavorable conditions of inferior subgrade, poor drainage, and unfavorable climate.

Fig. 11, Examples 10-D and 11-C. Example 10-D at the top of the figure is of a Class II rigid pavement designed for full legal axle loads, compensated for seasonal loss of strength. The increase in roughness in the outer wheel path from 51 to 84 and in the inner wheel path from 39 to 65 inches per mile are sizeable changes in roughness percentage-wise. However, riding quality would still be rated very good or good, and there is little if any evidence of sharp increase in pavement displacement or frost bumps suggesting permanent damage to the pavement.

On the lower part of the figure are the comparative profiles of a Class III rigid pavement, Example 11-C on M-21, in which the design does not compensate for seasonal loss of strength. The changes in roughness from 186 to 249 in the outer wheel path and from 147 to 181 in the inner wheel path are considerable, although less percentage-wise than in the previous example, but there are a number of sharp displacements or frost bumps suggesting damage in the form of permanent displacements that may not disappear completely next summer when the frost is gone and the pavement settles back to its most stable condition. This may tie in with the permanent rating of the pavement in terms

of riding quality as poor or very poor, with a temporary rating under frost action of poor and outside the rating scale in the outer wheel path.

Fig. 12, Examples 14-C and 16-D. In Fig. 12 are shown two comparative profiles of flexible bituminous pavements in the fall and late winter. Example 14-C on the top of the sheet is a well designed Class II pavement with a permanent rating of exceptionally smooth and a temporary rating that would still be fair in the outer wheel path and good in the inner wheel path under maximum frost action. The changes in roughness from 33 to 108 and 44 to 95, respectively, are considerable in percentage and there are several frost bumps which may leave a permanent effect. The data indicate the difficulty of eliminating frost heaving even in well designed roads, but the final answer again will be found in next summer's stable profiles.

It is more difficult to describe the results shown by Example 16-D, a Class IV pavement which is the rough section of M-36. The roughness index on both the fall and temporary late winter profiles are far outside the rating scale. The change in roughness due to frost heaving from 363 to 446 in the outer wheel path is considerable, while there is no significant change in the inner wheel path. The close correspondence between the fall and late winter profiles in both cases suggests that vertical displacements are closely related to frost action. Subsequent profiles after the Spring thaw and then late next summer may throw further light on the cyclic changes in the permanent riding quality.

Fig. 13, Examples 15-C and 15-D. The next two examples in Fig. 13 are also flexible pavements on M-100 rated as Class III and subject to restrictions due to seasonal loss of strength. This rating is in spite of the fact that their reconstruction in 1953, including a 12" sand subbase, gravel base and bituminous surface over the old road, brought them well up to present day standards of all-season roads. The restricted rating is due to a

few rough sections that have been trouble spots which required unanticipated maintenance.

At the top of the figure, Example 15-C is the smooth section representative of most of M-100 in the area surveyed which has been giving excellent service. The fall profile, with roughness indexes of 34 in the outer wheel path and 40 in the inner, would be rated exceptionally smooth. The frost heaving roughness indexes of 130 in the outer wheel path and 98 in the inner is an increase in roughness from 2 to 4 times that recorded in the stable summer profile. The greater heaving in the outer wheel path, which is typical of the winter profiles, may be the result of infiltration and freezing of water from the shoulder or ditch. There are several points where the heaving approaches a frost bump, but otherwise it may be reasonable to expect this road to go back to its stable summer profile without permanent loss of riding quality.

The rough section of M-100 in Example 15-D which is over buried peat shows close correspondence between fall and winter profiles, suggesting that the net increase in permanent displacement may be very small. The greater increase in roughness in the outer wheel path is again typical, and there is almost no change in the inner wheel path.

Fig. 14, Examples 7-C and 7-D. The last illustration, Fig. 14 with Examples 7-C and 7-D shows the two selected sections on US-112, with the smoother section at the top of the figure and the rougher section at the bottom. Increase in roughness due to frost action is considerable, but there is only one frost bump of any significant magnitude, at the extreme right of the profile, which may produce permanent vertical displacement. Example 7-D at the bottom is a section of pavement rated as Class IV that has had heavy maintenance since construction in 1926. After resurfacing in 1943, it again

became rough and was resurfaced late in 1958. The smoother profile in Fig. 14 with roughness indexes of 66 and 65 in the outer and inner wheel paths respectively changes to roughness indexes of 112 and 94, which are not as large as on several of the other projects. There are several sharp displacements approaching frost bumps which result in permanent changes. From the past history of this road and lack of evidence of severe frost action, the poor performance of this pavement may be due more to inadequate subgrade support and heavy traffic than to frost action. If so, significant changes will depend on a longer period of service under traffic.

Conclusion

As a result of these first frost heave profiles, it is considered inadvisable to attempt to draw conclusions until more data are available and profiles are taken after the Spring thaw and late summer. However, it is felt that the quantitative measure of frost action is not only interesting, but shows some evidence of significant changes. The amount of heaving on roads of present day standards, designed to compensate for seasonal changes, is more than was anticipated. From the results, it may be suggested that the designs do not prevent frost heaving or temporary displacements. It would then appear that the adequacy of these pavements will depend more on their behavior during the period of decreased subgrade support as reflected in permanent changes after the pavements reach their most stable condition in late summer.

TENTATIVE

PAVEMENT ROUGHNESS RATING

VERTICAL DISPLACEMENT - INCHES PER MILE

U.S. BUREAU OF PUBLIC ROADS ⁽¹⁾ <u>SINGLE WHEEL ROUGHOMETER</u>	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 4-5 MILES PER HOUR.

FIG. 1

2" BITUMINOUS CONCRETE-1948
 9" PLAIN CONCRETE-1926

SUBGRADE: SANDY CLAY LOAM

DRAINAGE: FAIR

FIGURE - 7 A

CLASS-I RIGID

US-112, 1926 SERVICE 32 YEARS

TRAFFIC: TOTAL 4800, COMMERCIAL 1525, DHV 675

DATE OF SURVEY: JULY 28, 1958

2" BITUMINOUS CONCRETE-1958
 9" PLAIN CONCRETE-1926

SUBGRADE: SANDY CLAY LOAM

DRAINAGE: POOR

FIGURE - 7 B

CLASS-IV RIGID

US-112, 1926 SERVICE 32 YEARS

TRAFFIC: TOTAL 4300, COMMERCIAL 1400, DHV 675

DATE OF SURVEY: AUGUST 4, 1958

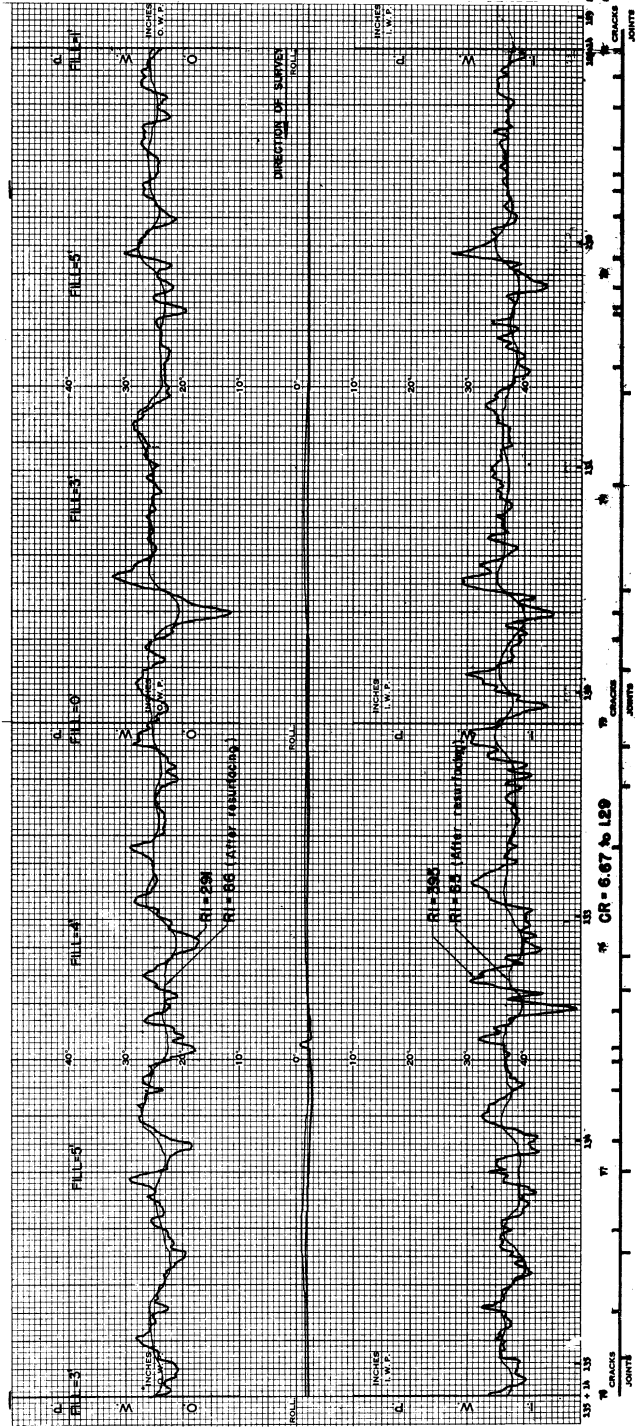
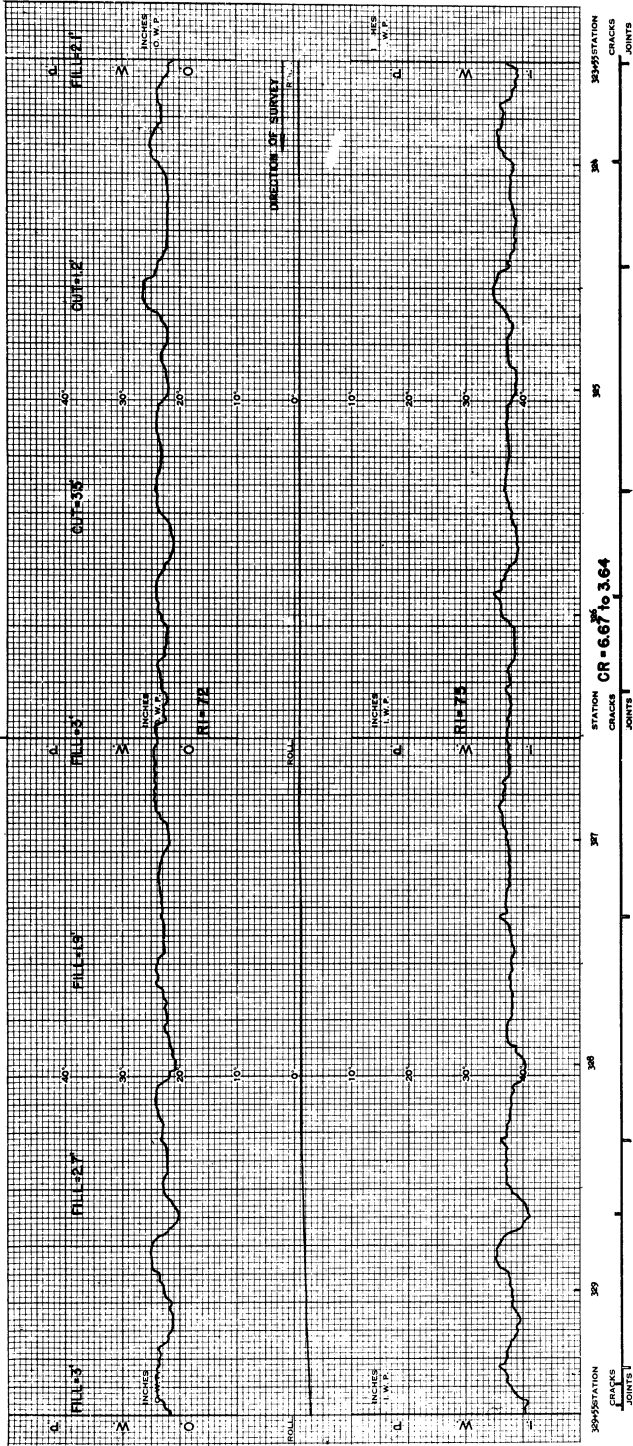
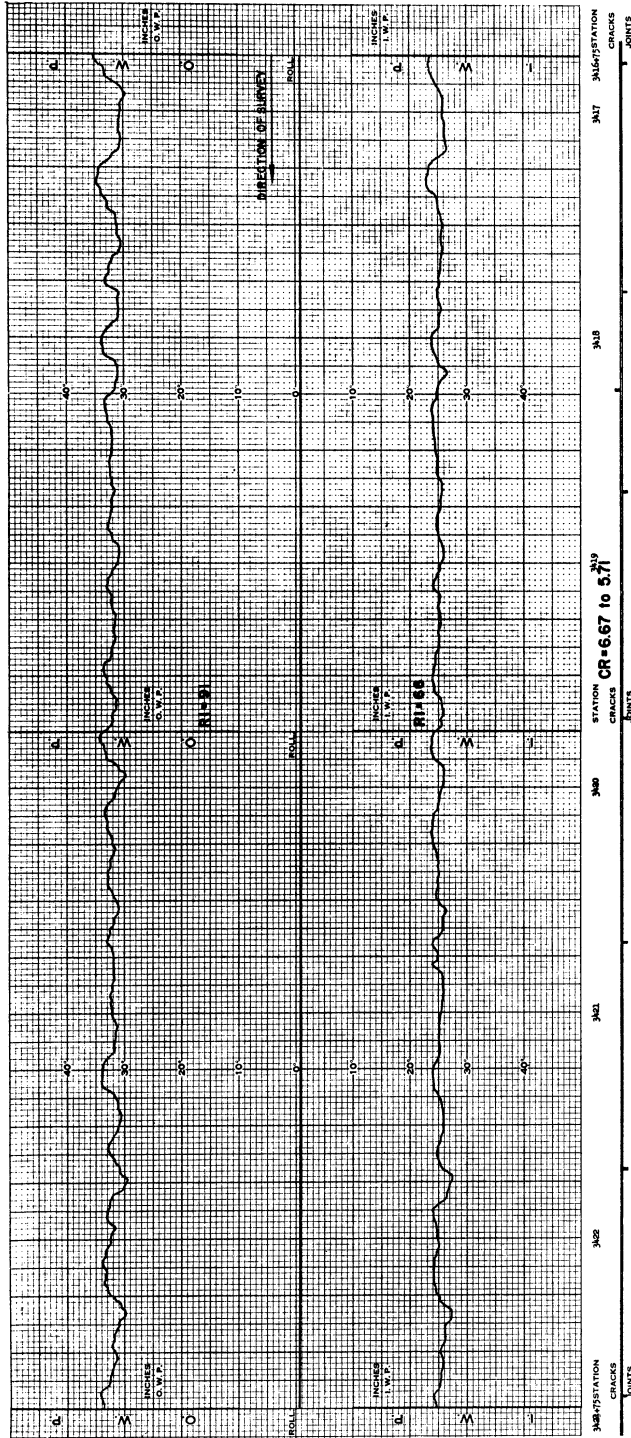
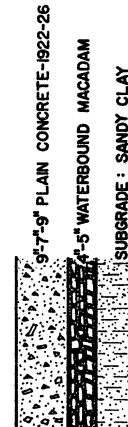
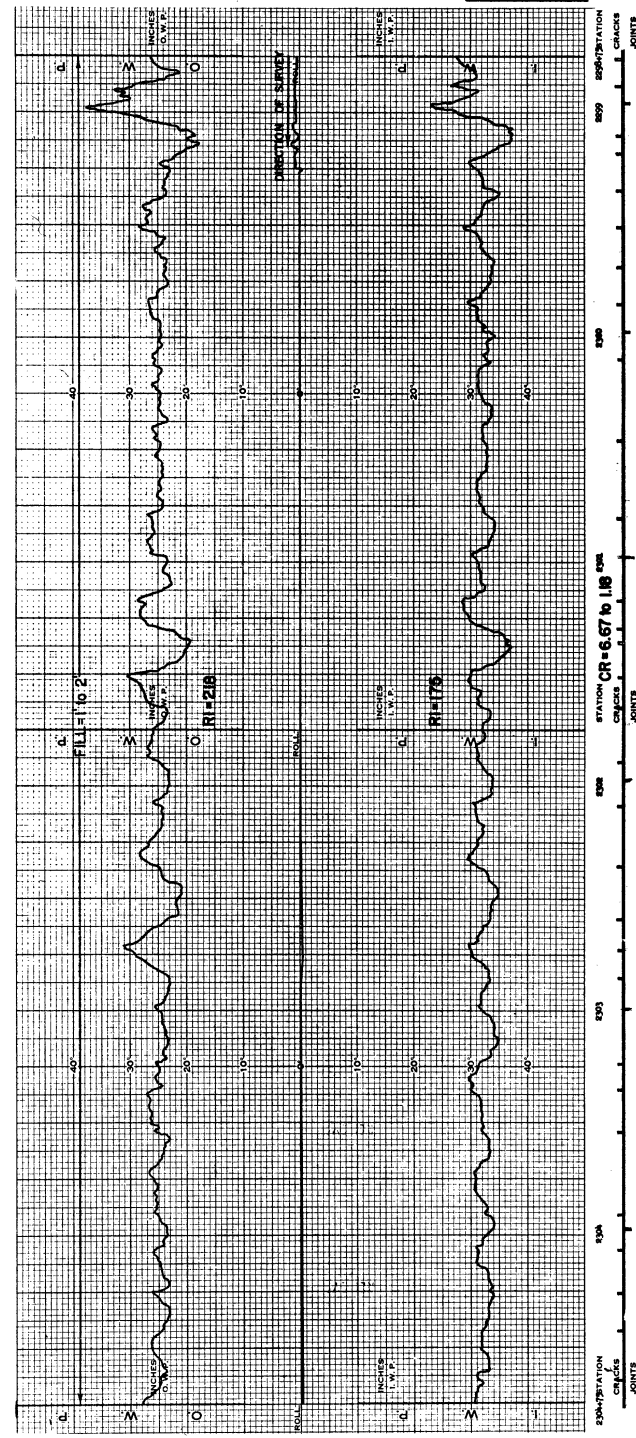


FIG. 2



DRAINAGE: EXCELLENT

FIGURE - 8 A
CLASS-1 RIGID
M - 25, 1930-31 SERVICE 28 YEARS
TRAFFIC: TOTAL 700, COMMERCIAL 60, DHV 140-150
DATE OF SURVEY: JUNE 5, 1959



DRAINAGE: POOR

FIGURE - 8 B
CLASS-IV RIGID
M - 25, 1922-26 SERVICE 36 YEARS
TRAFFIC: TOTAL 1800, COMMERCIAL 310, DHV 350
DATE OF SURVEY: JUNE 6, 1958

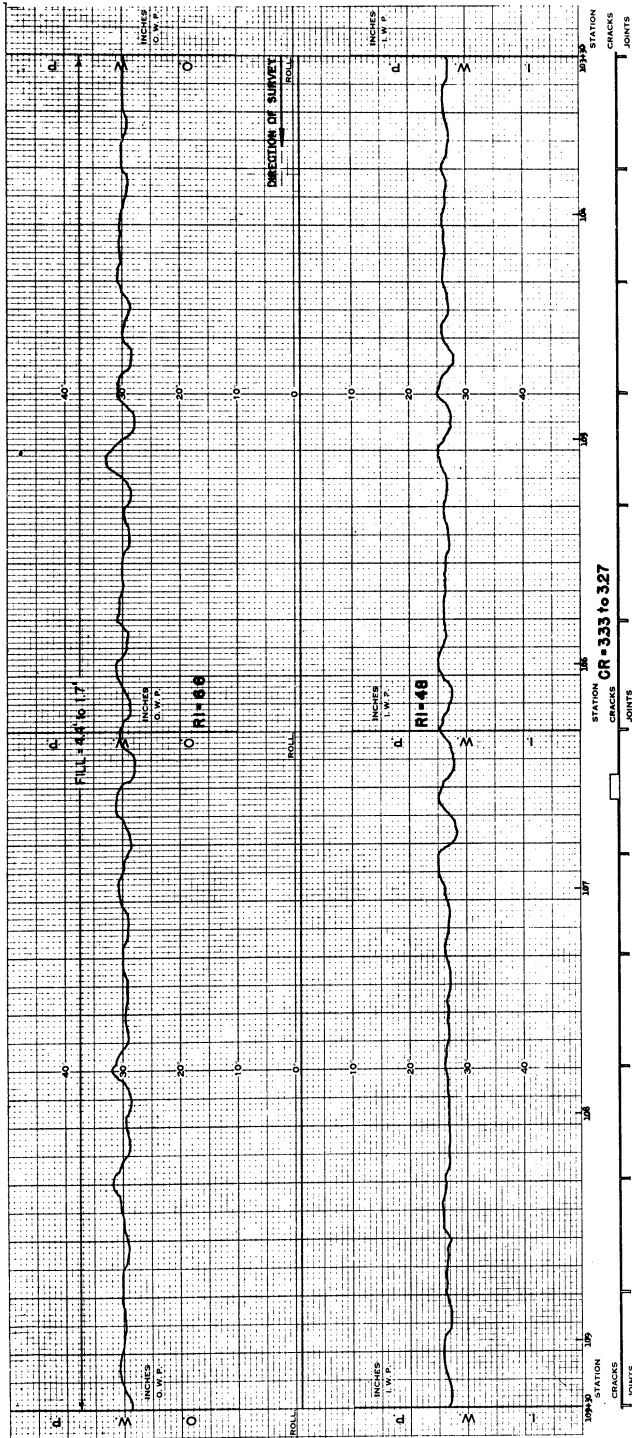


FIGURE - 10 A
 CLASS - II RIGID
 M - 43, 1947 SERVICE 11 YEARS
 TRAFFIC: TOTAL 1500-3300, COMMERCIAL 280, DHV 400
 DATE OF SURVEY: OCT. 7, 1958

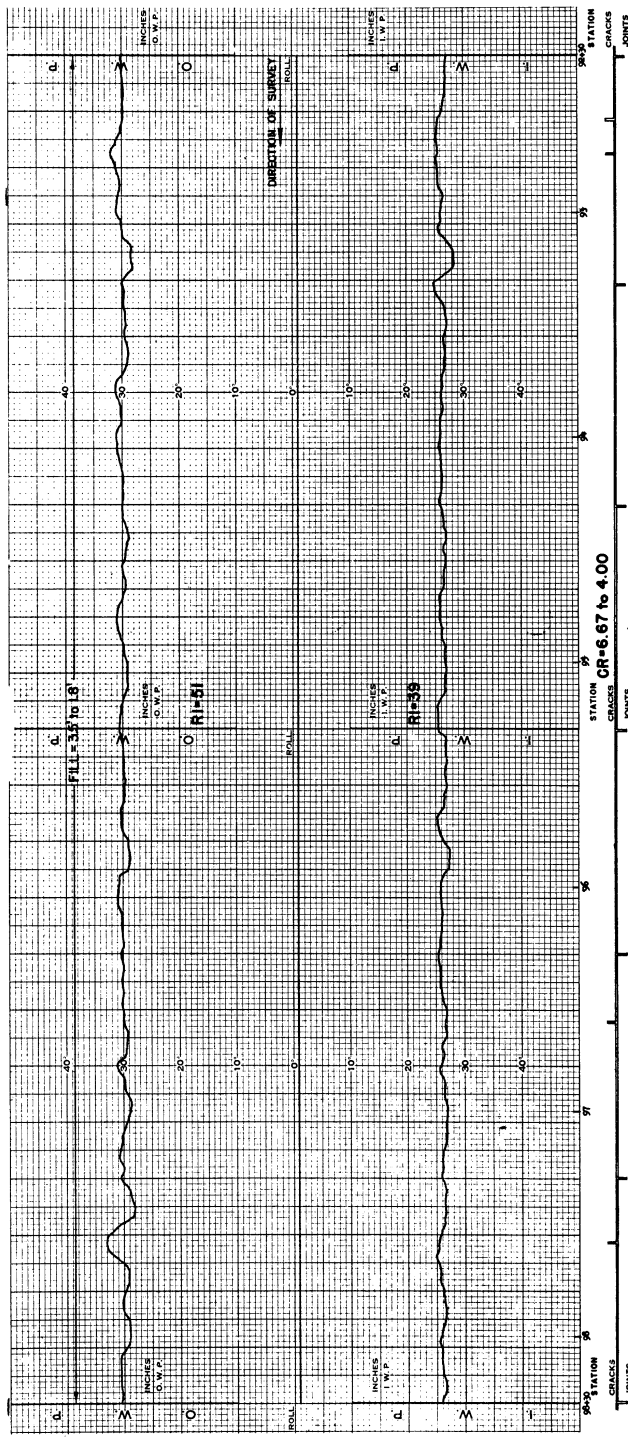
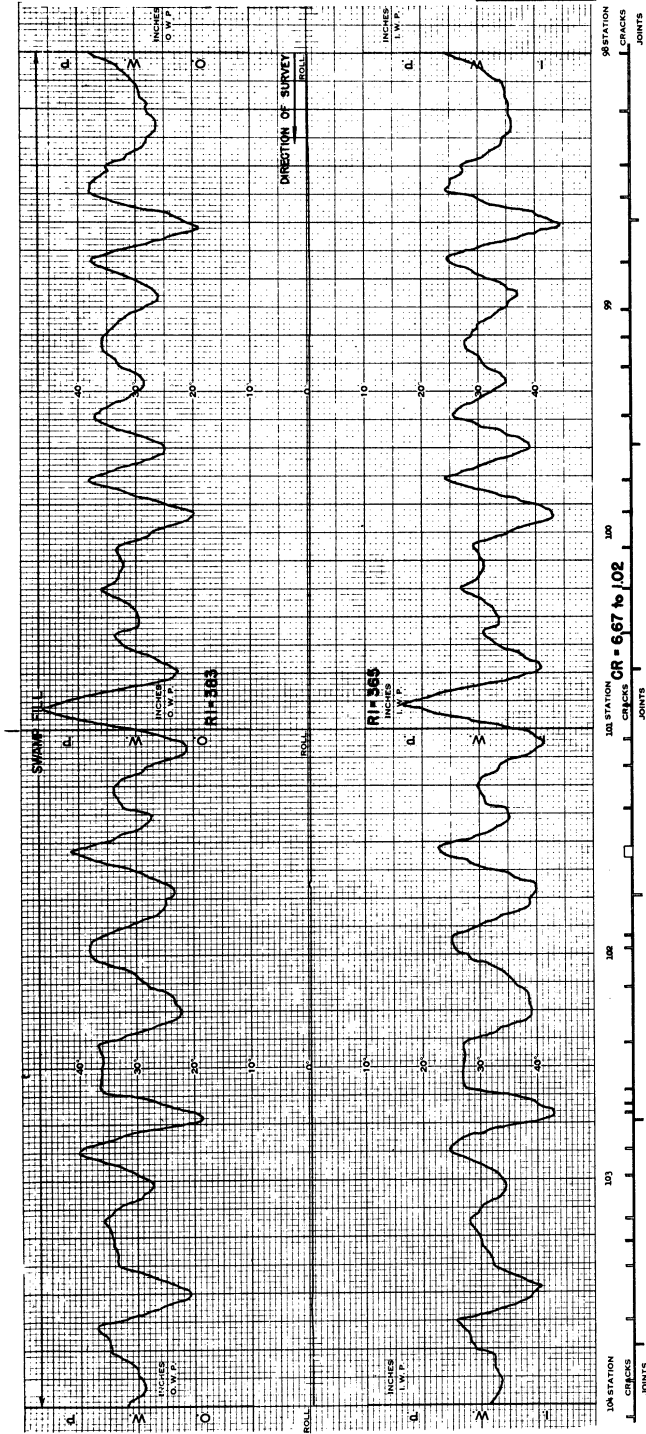


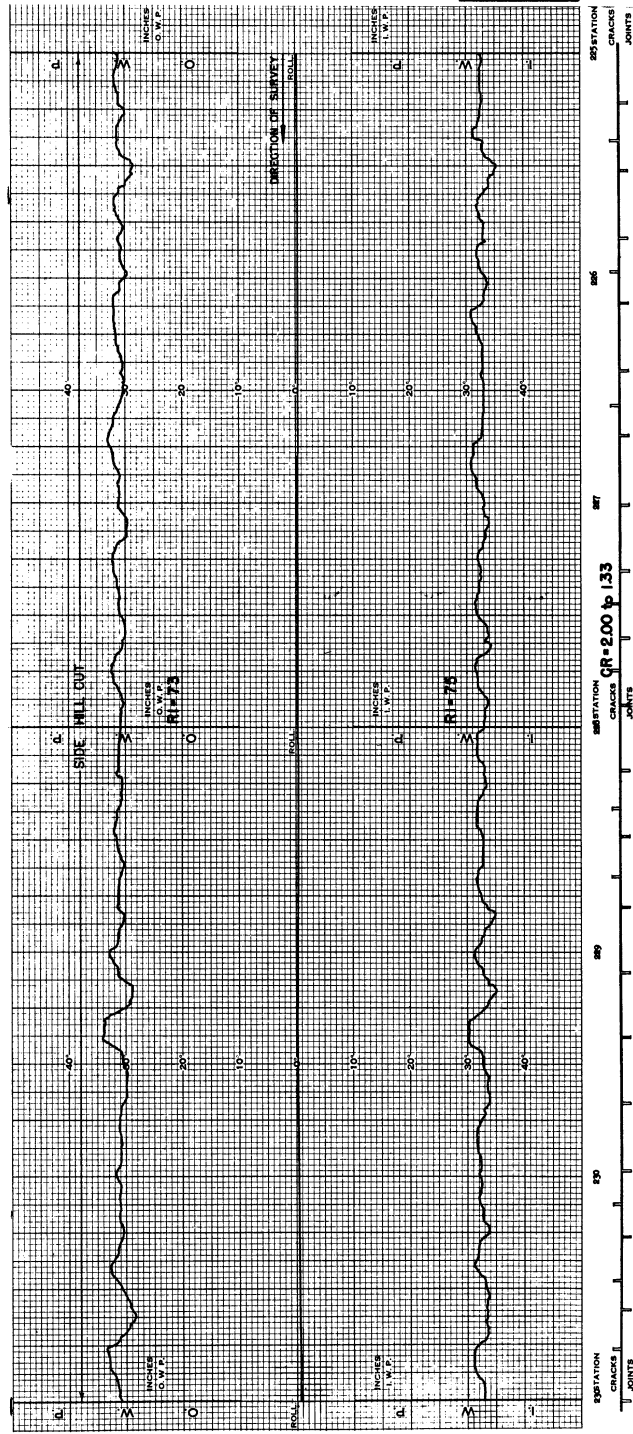
FIGURE - 10 B
 CLASS - II RIGID
 M - 43, 1947 SERVICE 11 YEARS
 TRAFFIC: TOTAL 1500-3300, COMMERCIAL 280, DHV 400
 DATE OF SURVEY: OCT. 7, 1958



9 4 0 8 7 6 5 4 3 2 1 0 0 1 2 3 4 5 6 7 8 9
 5'-6" SUBGRADE HEAVY SANDY LOAM

DRAINAGE: POOR

FIGURE - 12 A
 CLASS-III RIGID
 US-41, 1936 SERVICE 22 YEARS
 TRAFFIC: TOTAL 2000, COMMERCIAL 280-550, DHV 310
 DATE OF SURVEY: NOV. 8, 1958



9 4 0 8 7 6 5 4 3 2 1 0 0 1 2 3 4 5 6 7 8 9
 SUBGRADE: LIGHT SANDY LOAM

DRAINAGE: GOOD

FIGURE - 12 B
 CLASS-III RIGID
 US-41, 1936 SERVICE 22 YEARS
 TRAFFIC: TOTAL 1500, COMMERCIAL 280, DHV 190
 DATE OF SURVEY: NOV. 7, 1958

FIG. 5

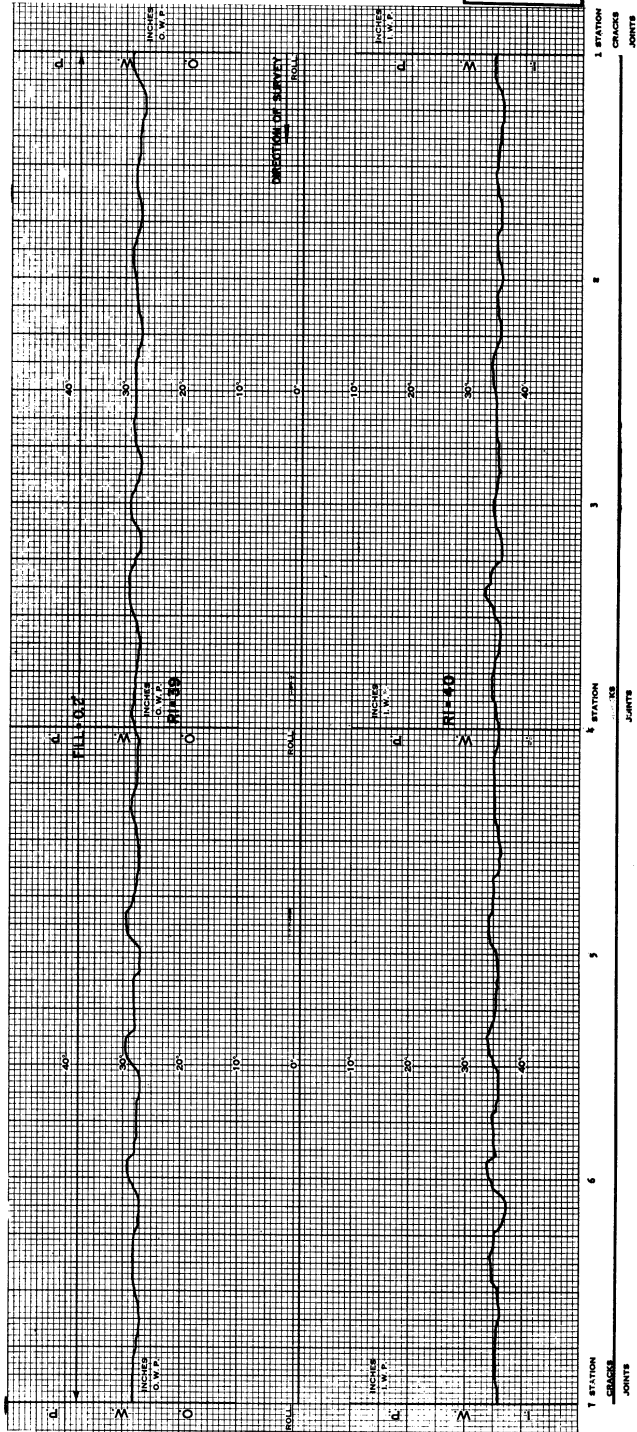
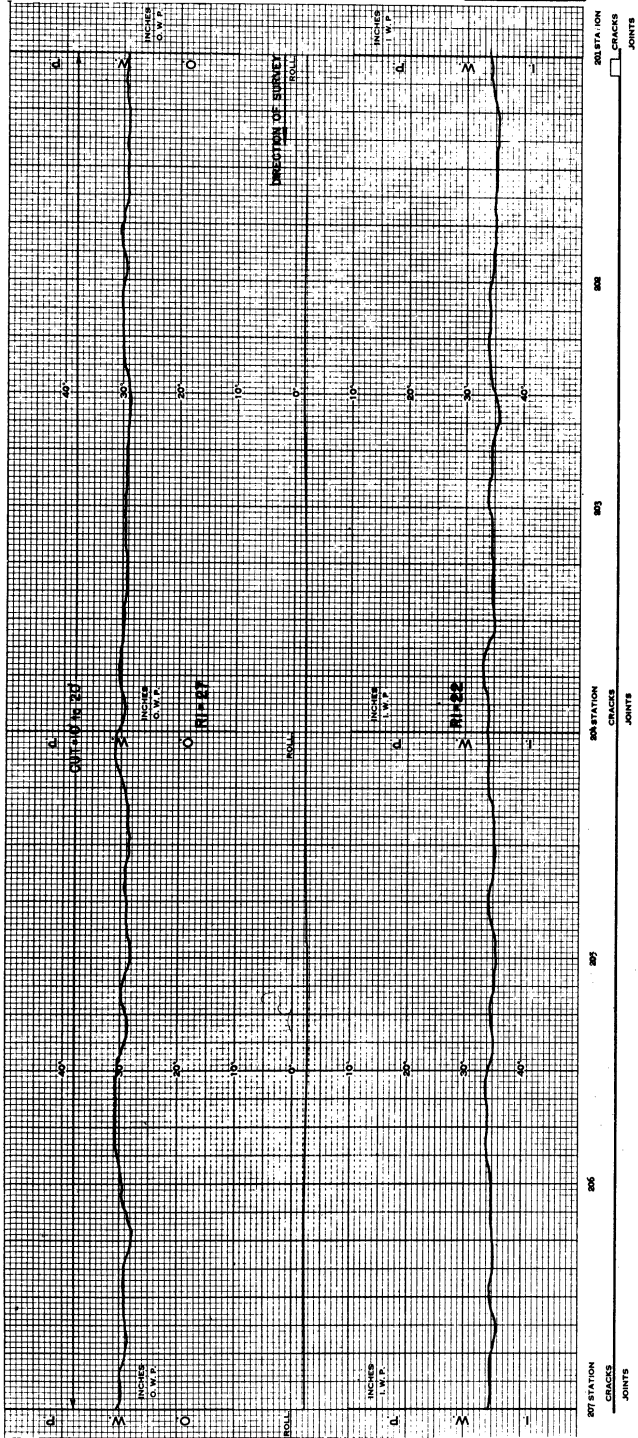


FIG. 6

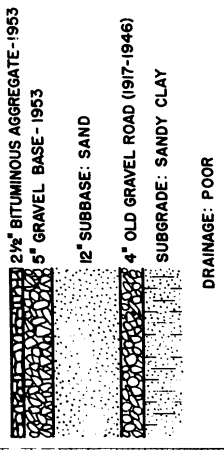


FIGURE-15 A
CLASS-III FLEXIBLE
M-100, 1953 SERVICE 5 YEARS
TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
DATE OF SURVEY: DEC. 4, 1958

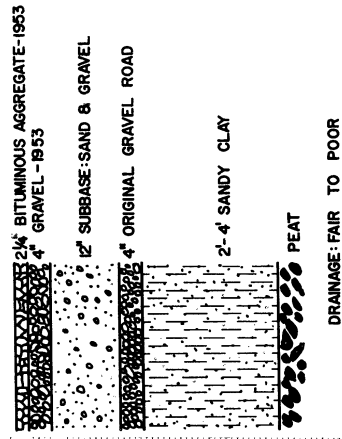
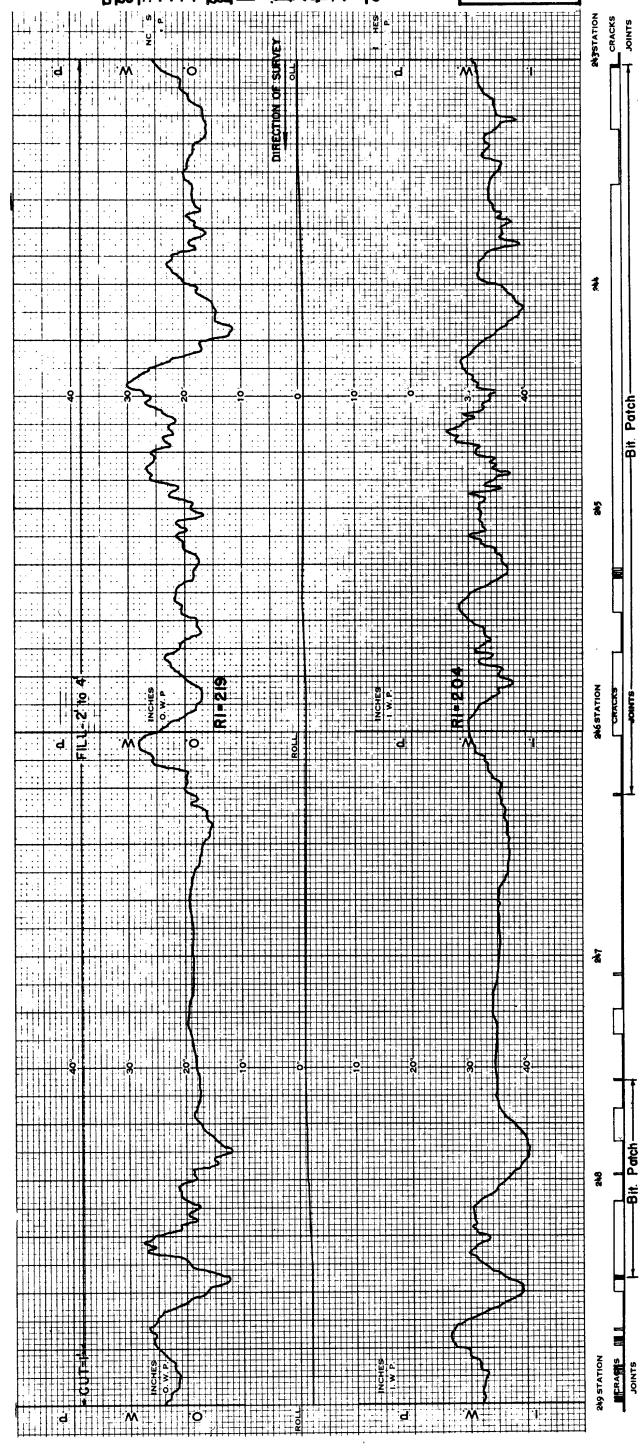
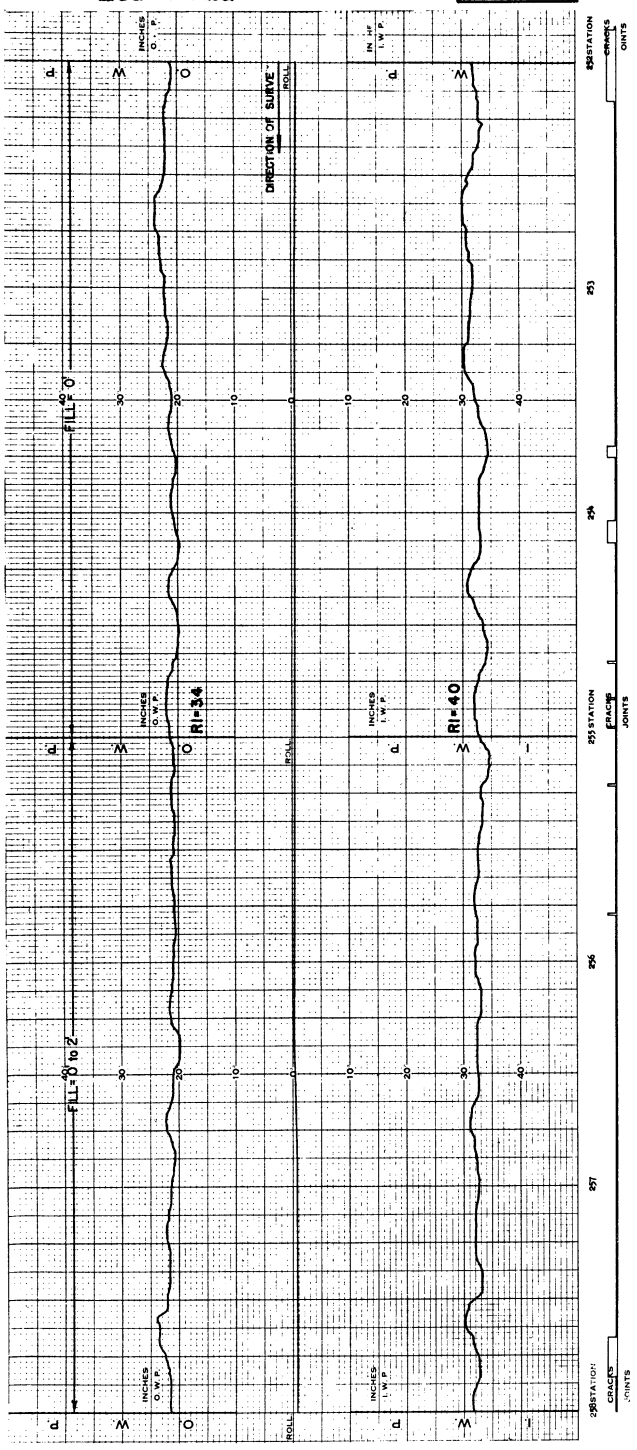
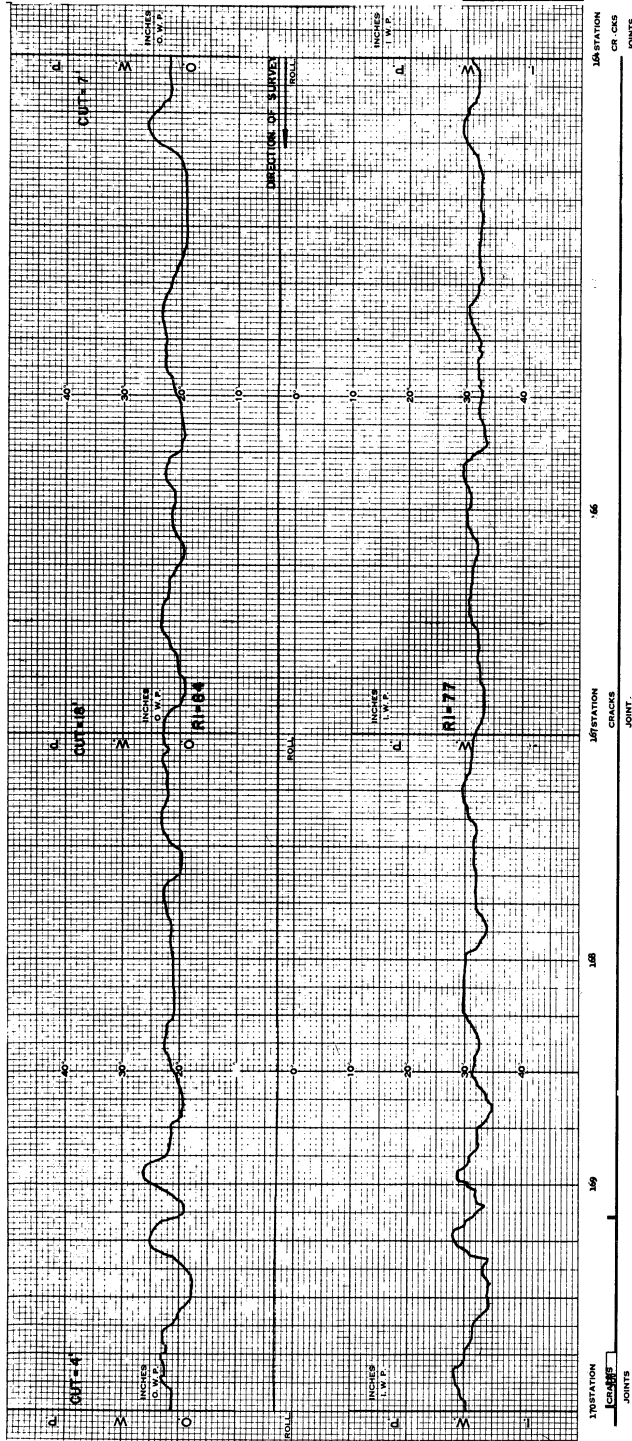


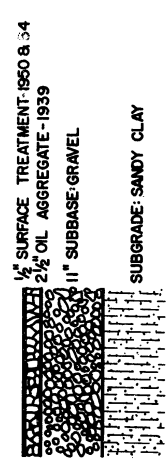
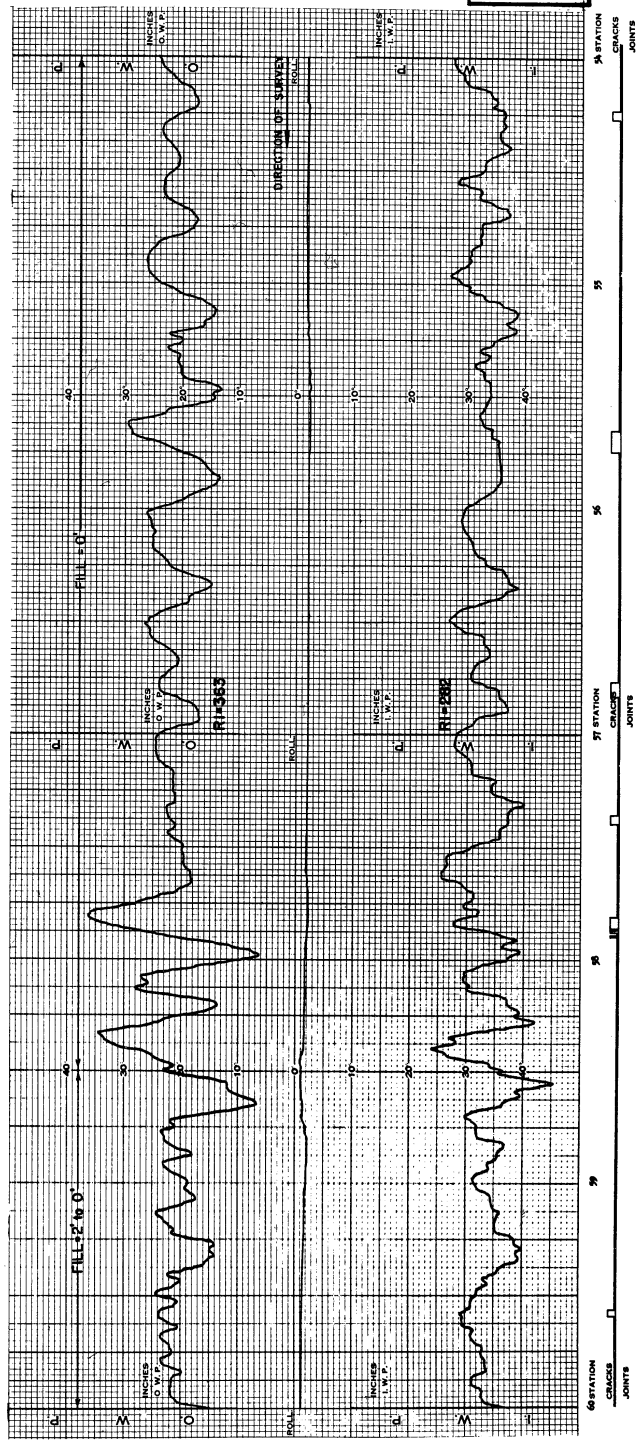
FIGURE-15 B
CLASS-III FLEXIBLE
M-100, 1953 SERVICE 5 YEARS
TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
DATE OF SURVEY: DEC. 4, 1958





DRAINAGE: EXCELLENT

FIGURE-16 A
 CLASS-III FLEXIBLE
 M-36, 1939 SERVICE 19 YEARS
 TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
 DATE OF SURVEY: DEC. 5, 1958



DRAINAGE: POOR

FIGURE-16 B
 CLASS-IV FLEXIBLE
 M-36, 1939 SERVICE 19 YEARS
 TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
 DATE OF SURVEY: DEC. 5, 1958

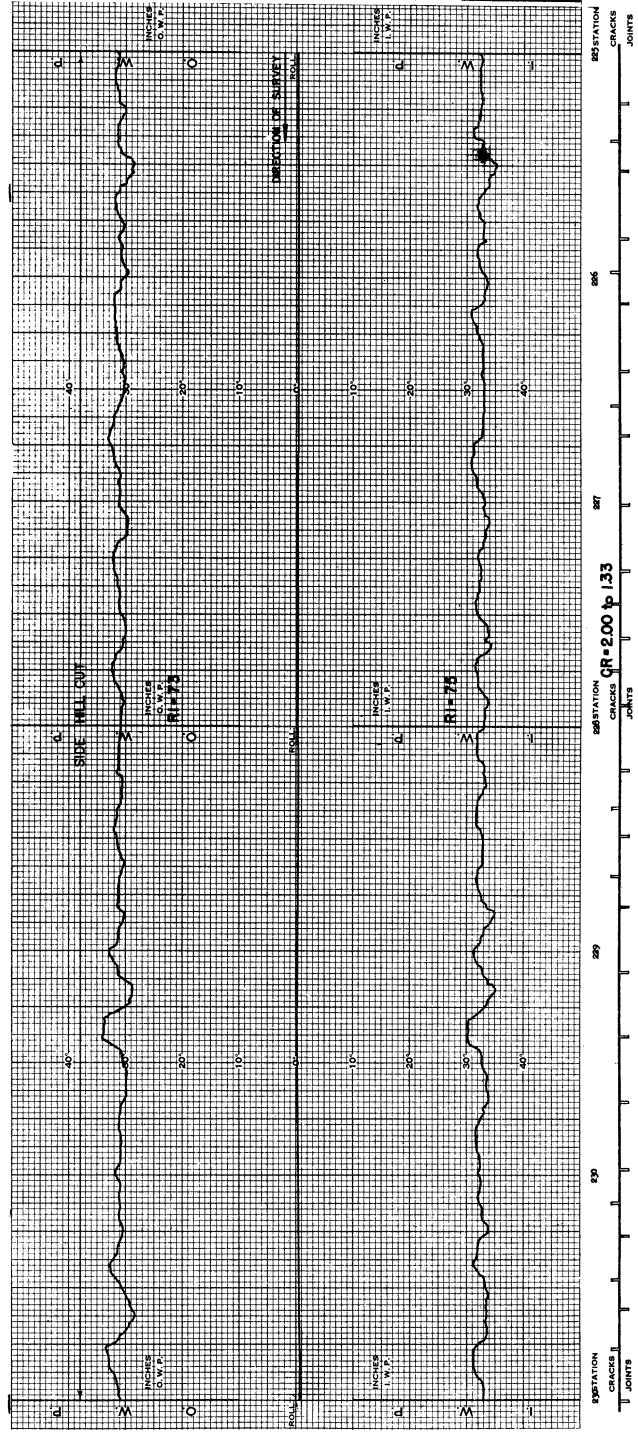
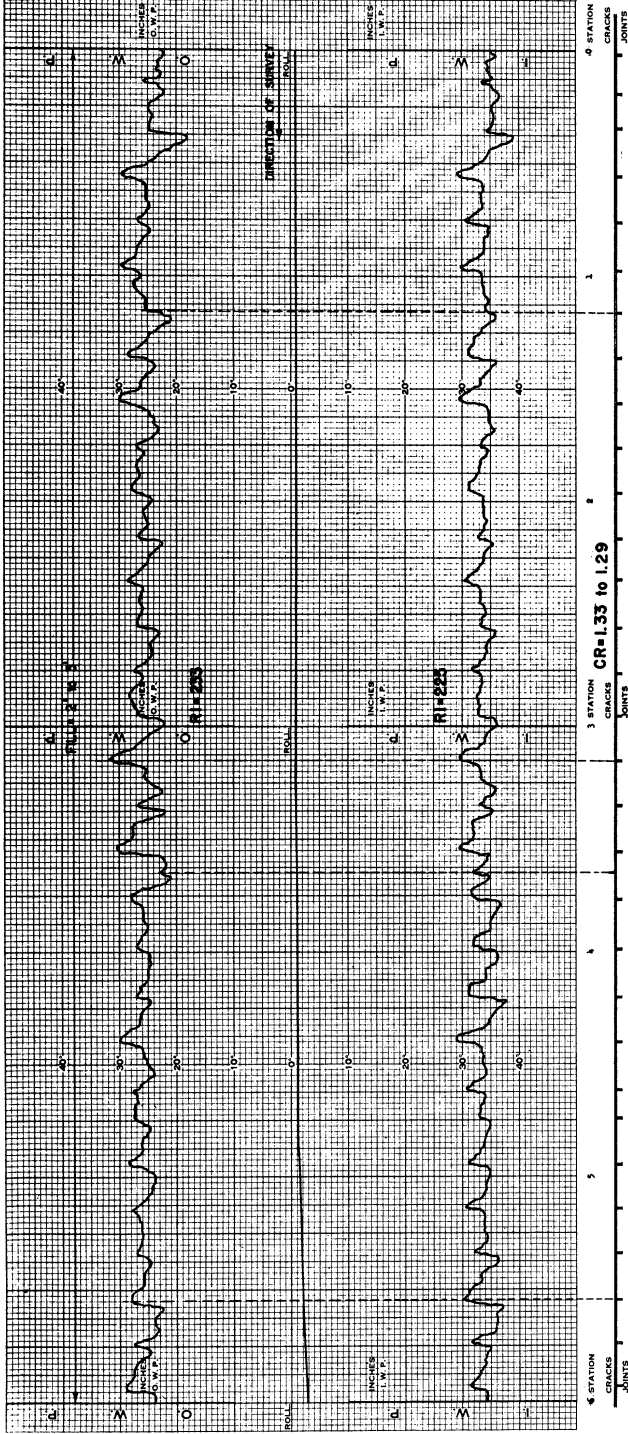


FIG. 9

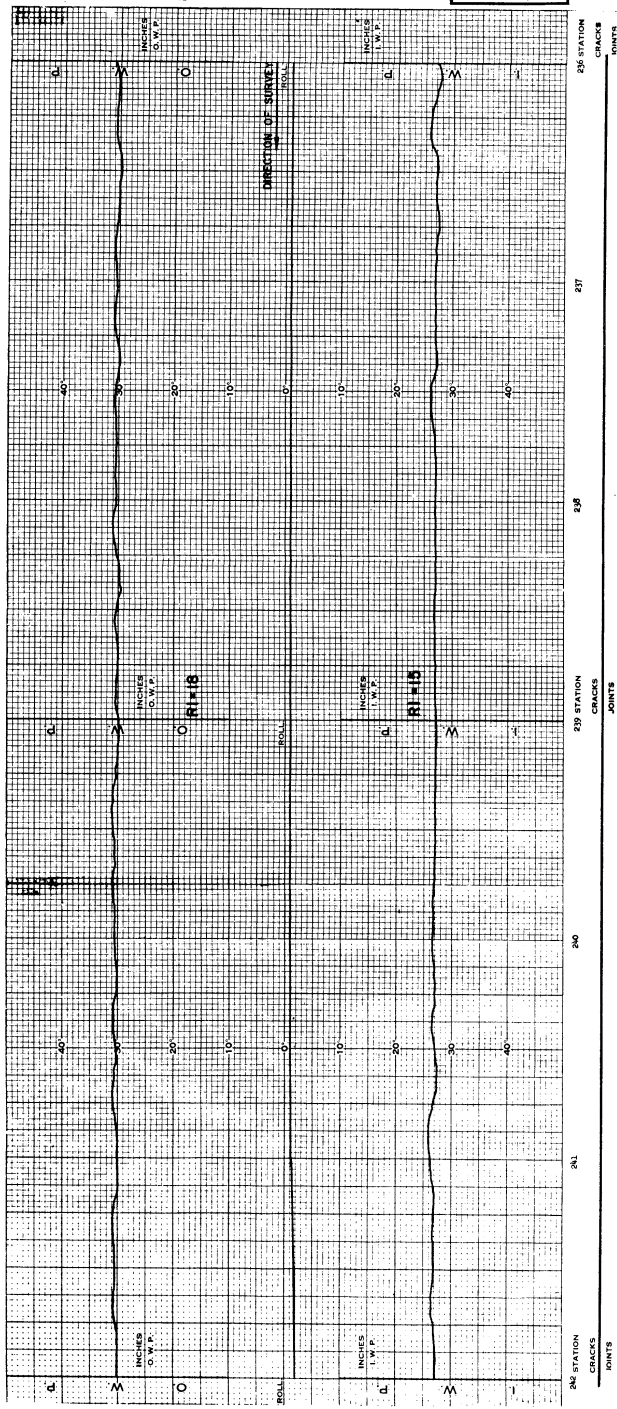
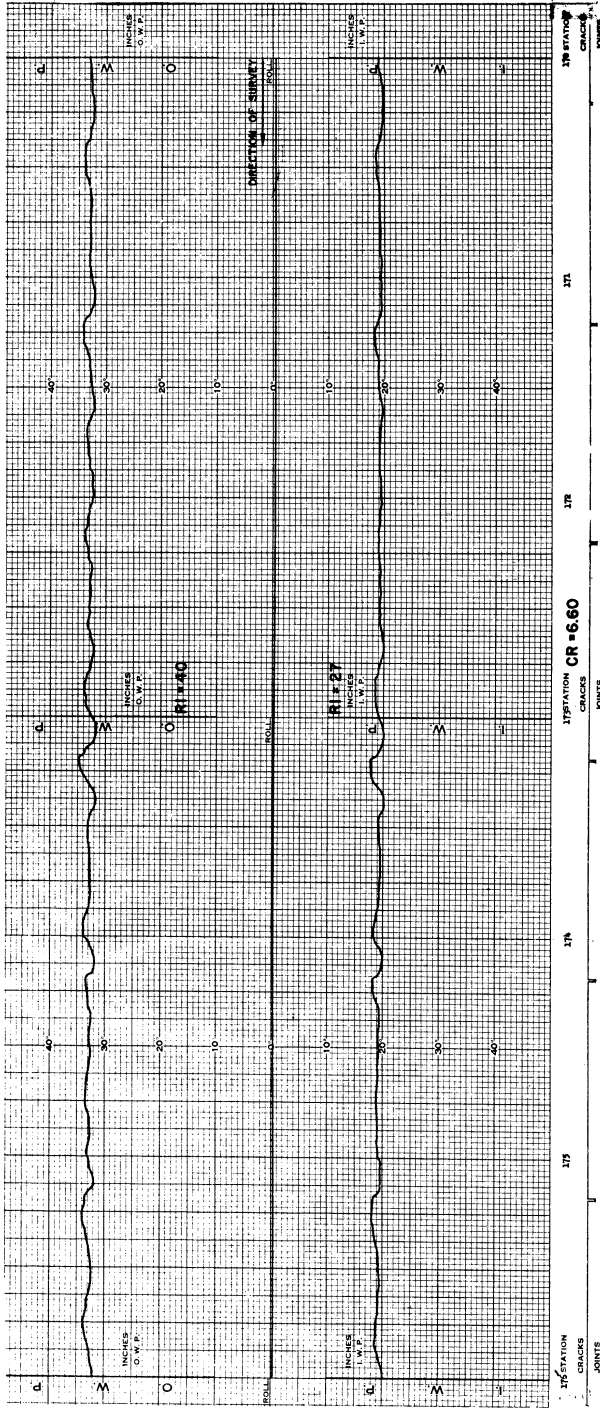


FIGURE -10

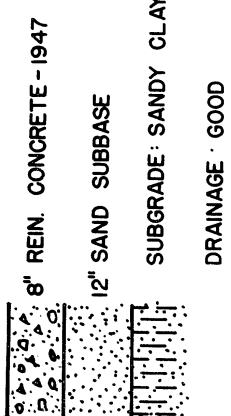
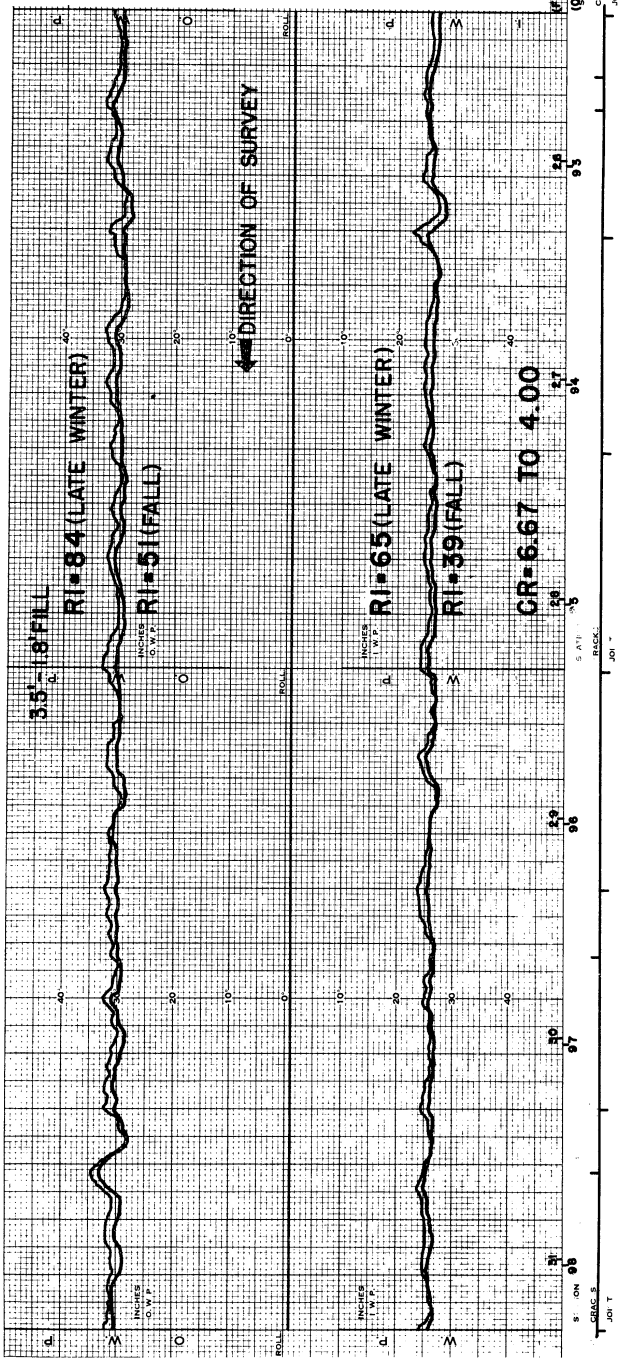


FIGURE - 10 D
 CLASS-II RIGID
 M-43, 1947 SERVICE II YEARS
 TRAFFIC: TOTAL 2400, COMMERCIAL 280, DHV 400
 DATE OF SURVEY: OCT 7, 1958 & FEB 20, 1959

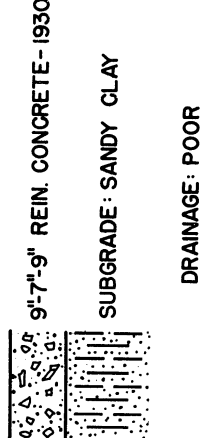
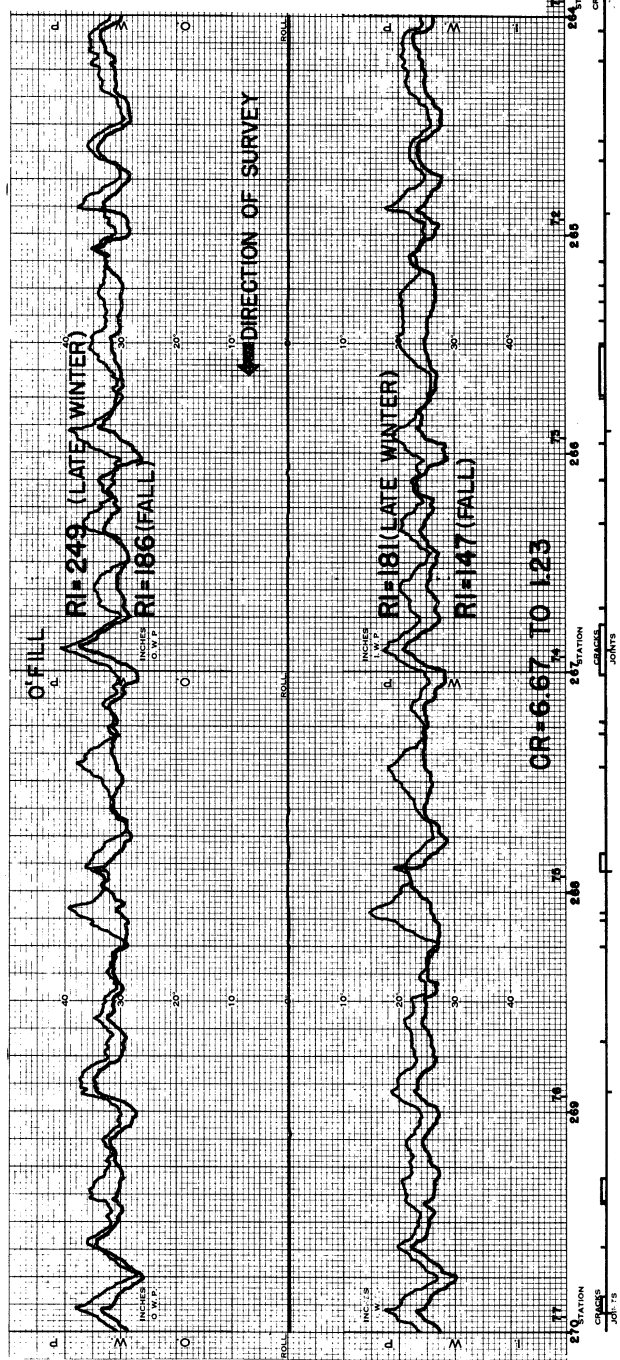


FIGURE - 11 C
 CLASS-III RIGID
 M-21, 1930 SERVICE 28 YEARS
 TRAFFIC: TOTAL 3800, COMMERCIAL 500, DHV 480
 DATE OF SURVEY: OCT. 10, 1958 & FEB 20, 1959

2 1/2" BITUMINOUS AGGREGATE - 1952
 GRAVEL BASE - 1952
 12" SUBBASE: SAND
 ORIGINAL GRAVEL SURFACE
 SUBGRADE: SANDY CLAY
 DRAINAGE: GOOD

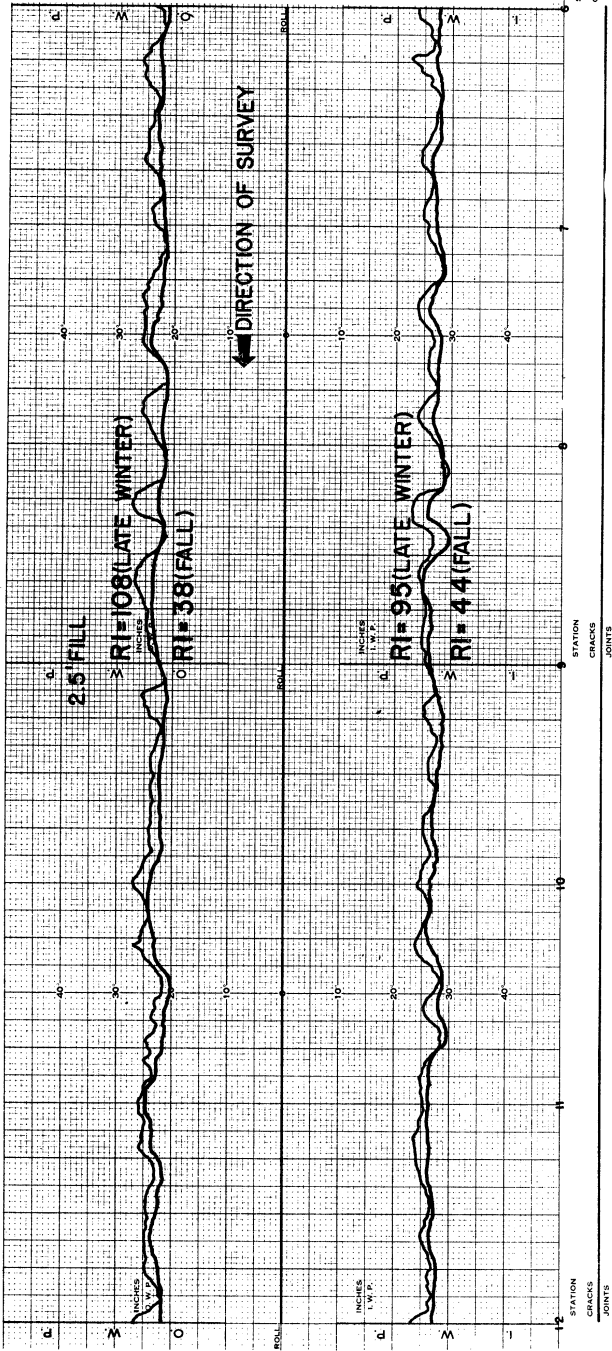
FIGURE - 14 C

CLASS-II FLEXIBLE

M-57, 1952 SERVICE 6 YEARS

TRAFFIC: TOTAL 1000, COMMERCIAL 200, DHV 130

DATE OF SURVEY: DEC. 5, 1958 & FEB. 20, 1959



1/2" SURFACE TREATMENT - 1950 & 54
 2 1/2" OIL AGGREGATE - 1939
 11" GRAVEL SUBBASE
 SUBGRADE: SANDY CLAY
 DRAINAGE: POOR

FIGURE - 16 D

CLASS-IV FLEXIBLE

M-36, 1939 SERVICE 19 YEARS

TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250

DATE OF SURVEY DEC. 5, 1958 & FEB. 20, 1959

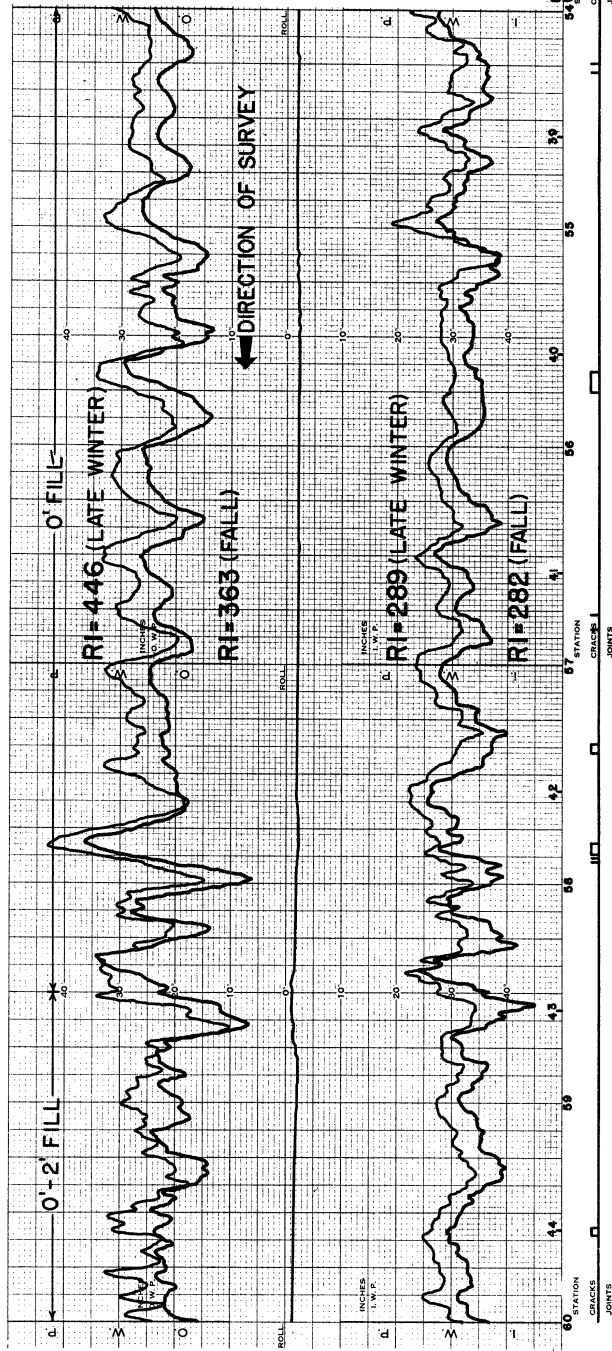


FIG. 12

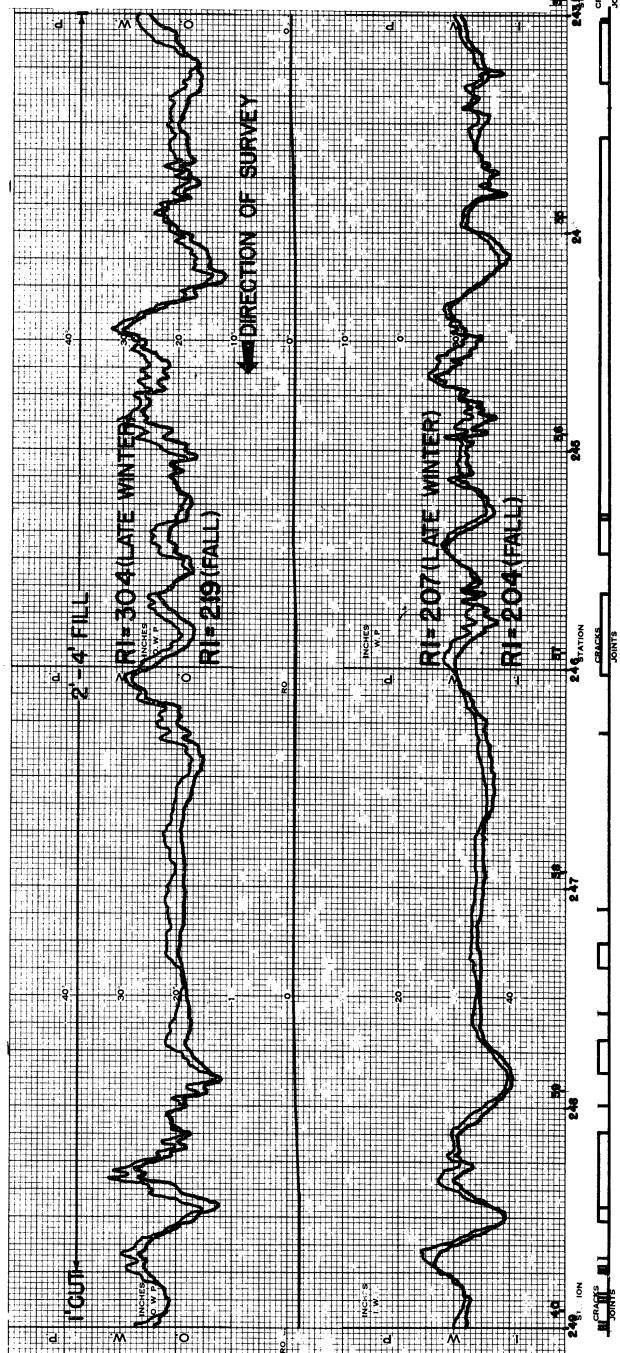
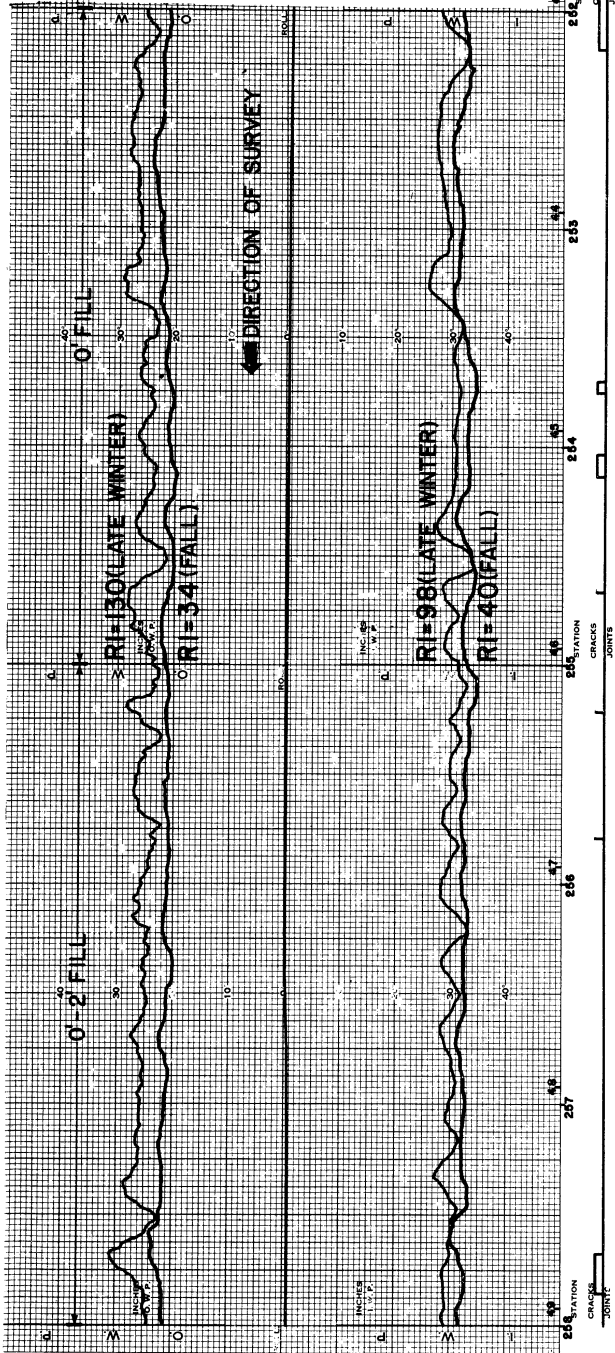
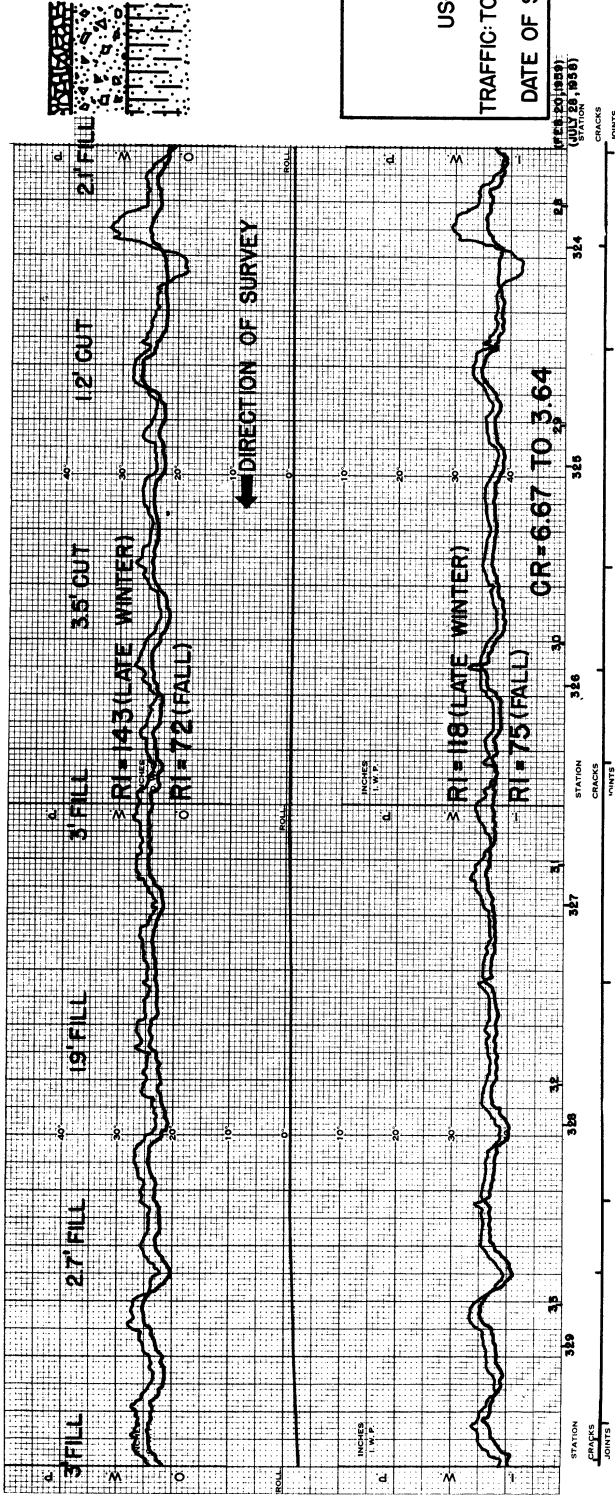


FIG. 13



2.4" BITUMINOUS CONCRETE - 1948
 9" 7"-9" PLAIN CONCRETE - 1926
 SUBGRADE: SANDY CLAY LOAM

DRAINAGE: FAIR

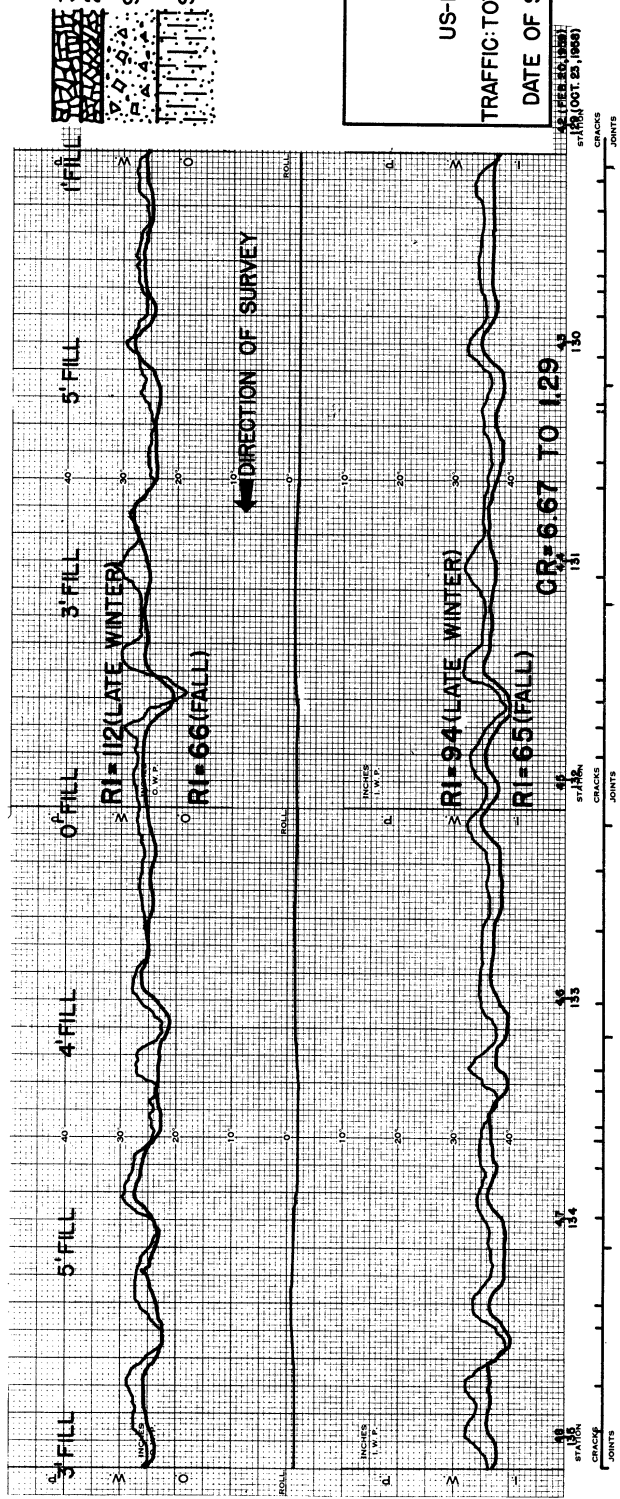
FIGURE - 7C

CLASS - I RIGID

US-112, 1926 SERVICE 32 YEARS

TRAFFIC: TOTAL 4800 COMMERCIAL 1525, DHV 675

DATE OF SURVEY: JULY 28, 1958 & FEB 20, 1959



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

DRAINAGE: POOR

FIGURE - 7D

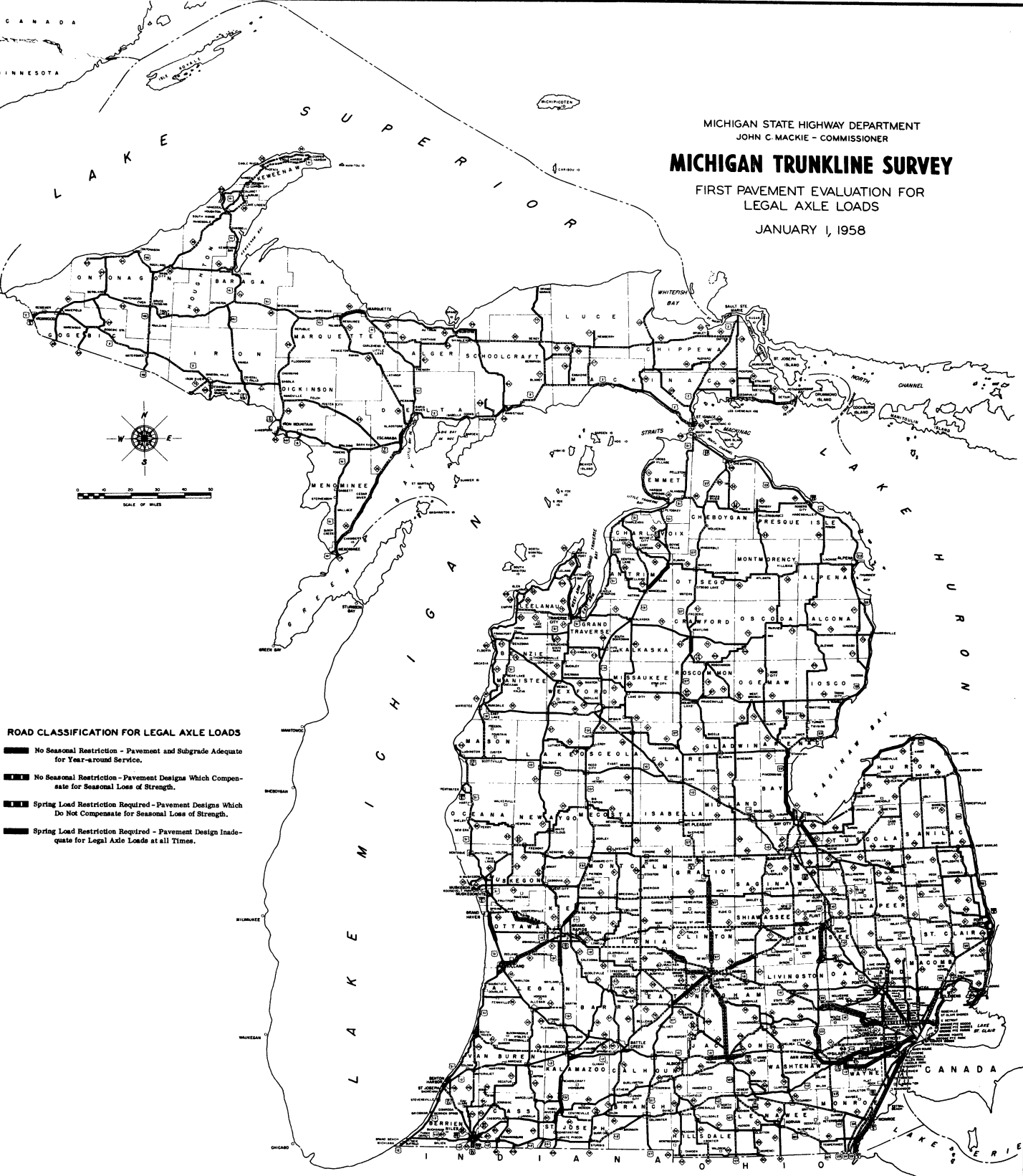
CLASS - IV RIGID

US-112, 1926 SERVICE 32 YEARS

TRAFFIC: TOTAL 4300 COMMERCIAL 1400, DHV 675

DATE OF SURVEY: AUG. 4, 1958 & FEB 20, 1959

FIG. 14



MICHIGAN STATE HIGHWAY DEPARTMENT
 JOHN C. MACKIE - COMMISSIONER

MICHIGAN TRUNKLINE SURVEY

FIRST PAVEMENT EVALUATION FOR
 LEGAL AXLE LOADS

JANUARY 1, 1958

- ROAD CLASSIFICATION FOR LEGAL AXLE LOADS**
- No Seasonal Restriction - Pavement and Subgrade Adequate for Year-around Service.
 - No Seasonal Restriction - Pavement Designs Which Compensate for Seasonal Loss of Strength.
 - Spring Load Restriction Required - Pavement Designs Which Do Not Compensate for Seasonal Loss of Strength.
 - Spring Load Restriction Required - Pavement Design Inadequate for Legal Axle Loads at all Times.

DEPARTMENT OF CIVIL ENGINEERING
THE UNIVERSITY OF MICHIGAN
ANN ARBOR, MICHIGAN

SERVICE BEHAVIOR AS A CRITERION
FOR PAVEMENT DESIGN

BY

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FORTY-EIGHTH ANNUAL MEETING
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SAN ANTONIO, TEXAS
MARCH, 1960

SERVICE BEHAVIOR AS A CRITERION FOR PAVEMENT DESIGN

by

William S. Housel¹

Most highway users, as laymen not familiar with the problems of highway design and construction, probably assume that highway design is a relatively accurate science regulated by well established physical laws which provide competent engineers with precise methods of pavement design. An uninitiated public would probably find it hard to understand if they were told that pavement design and construction at this late date in the history of highway transportation is still an unsolved problem with sharply contrasting viewpoints which, for some strange reason, are still unresolved after hundreds of years of highway building. Many have asked the question, the answer to which is still a subject of vigorous debate. How do you know when a pavement is structurally adequate?

To answer this question requires first the answers to a number of corollary questions. 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 the subgrade support put 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 put into a formula for pavement design? What about volume and character of traffic?

Michigan believes that the answers to these questions can be found in the behavior of roads that have been subjected to these conditions over many years. Pavement design in Michigan has been developed over a period of more than 35 years, using pavement performance studies and pavement condition surveys to correlate soil type and climatic environment with service behavior. With the coming of increased traffic and heavier loads, there has been some modification of the pavement structure including nominal increases in thickness; but, Michigan continues to emphasize the dominant role of climatic environment and soil types in pavement design. Briefly stated, actual field experience has indicated to Michigan highway designers that the most effective and the most economical method of providing for increased axle loads and increased load repetition of modern traffic is to supply the required load carrying capacity in the foundation of the pavement. This has been done by more accurate evaluation of soil conditions, drainage, and climatic influence; the utilization of local soil materials of favorable characteristics; and, the selection of pavement structures which more fully utilize available subgrade support.

It is recognized that many of these factors are difficult to measure and are not subject to control. Nevertheless, it is this same group of uncontrolled variables for which successful pavement design must provide. Pavement performance studies in Michigan over the past few years have been formulated

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Research Consultant - Michigan State Highway Department

on the proposition that the integrated result of these uncontrolled variables on pavement performance can 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, which are the two most important attributes of a highway pavement.

This may appear to be only stating the obvious, as highway builders from the very first have focused their attention on pavement roughness and cracking as direct evidence of its adequacy or, rather, its inadequacy. What has been lacking, however, is a practical yet sufficiently accurate method of measuring these indicators 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 the various physical conditions which cause these changes. 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.

Pavement performance studies which have been in progress in Michigan since 1952 have been predicated on the belief that an accurate field survey of existing pavements followed by intelligent analysis of the results would provide the most effective method of answering some of the unsolved problems of pavement design. This viewpoint has recently received some encouraging recognition. In a statement on urgently needed research in connection with rigid pavements, the Committee on Rigid Pavement Design of the Highway Research Board has proposed a priority project on the "Investigation of Existing Pavements". In outlining the problem, this statement lists a number of significant changes in design of rigid pavements in recent years, and concludes with the following statement:

"It is believed to be highly important to determine the effects of these changes in order to avoid the possibility of constructing many miles of pavement which might otherwise fail prematurely. 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."

The first attempt to measure pavement performance quantitatively in terms of pavement roughness and structural continuity by the roughness index (RI) and a continuity ratio (CR), as defined in this paper, was in 1952. During the period from 1952 to 1956, the University of Michigan, the Michigan State Highway Department, and the Wire Reinforcement Institute collaborated in a study to measure the effect of steel reinforcement on concrete pavements under actual service conditions. Pavement performance surveys were conducted on selected projects in southern Michigan which had been in service for periods varying from 10 to 25 years. Each project included comparable reinforced and unreinforced sections which were observed for approximately 5 years. The data that were obtained from the pavement condition survey of these selected projects showed that pavements with steel reinforcement were measurably smoother and the cracking was measurably less than in the unreinforced pavements.

These studies gave positive evidence of the value of steel reinforcement in pavement design as a means of maintaining structural continuity and

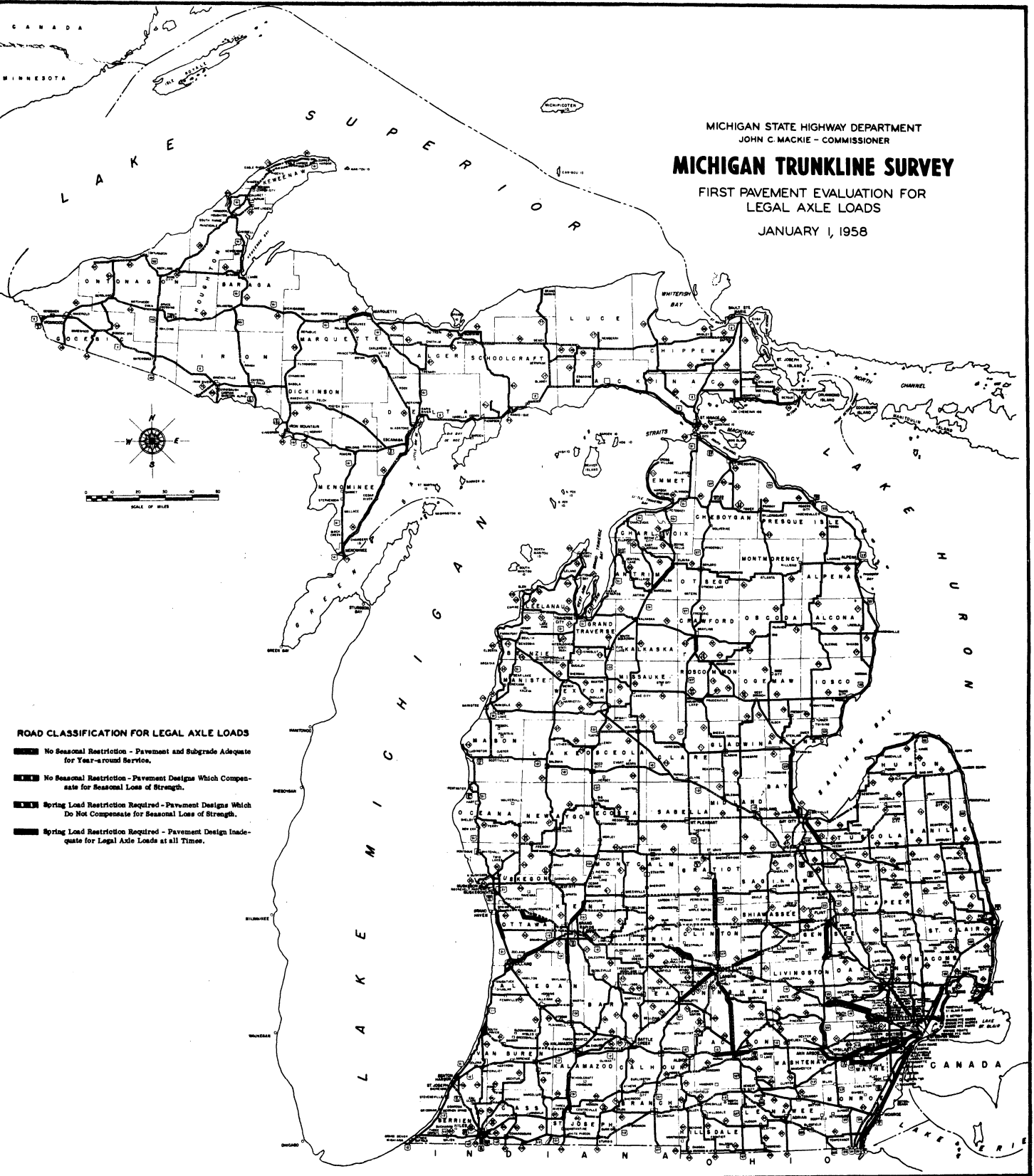
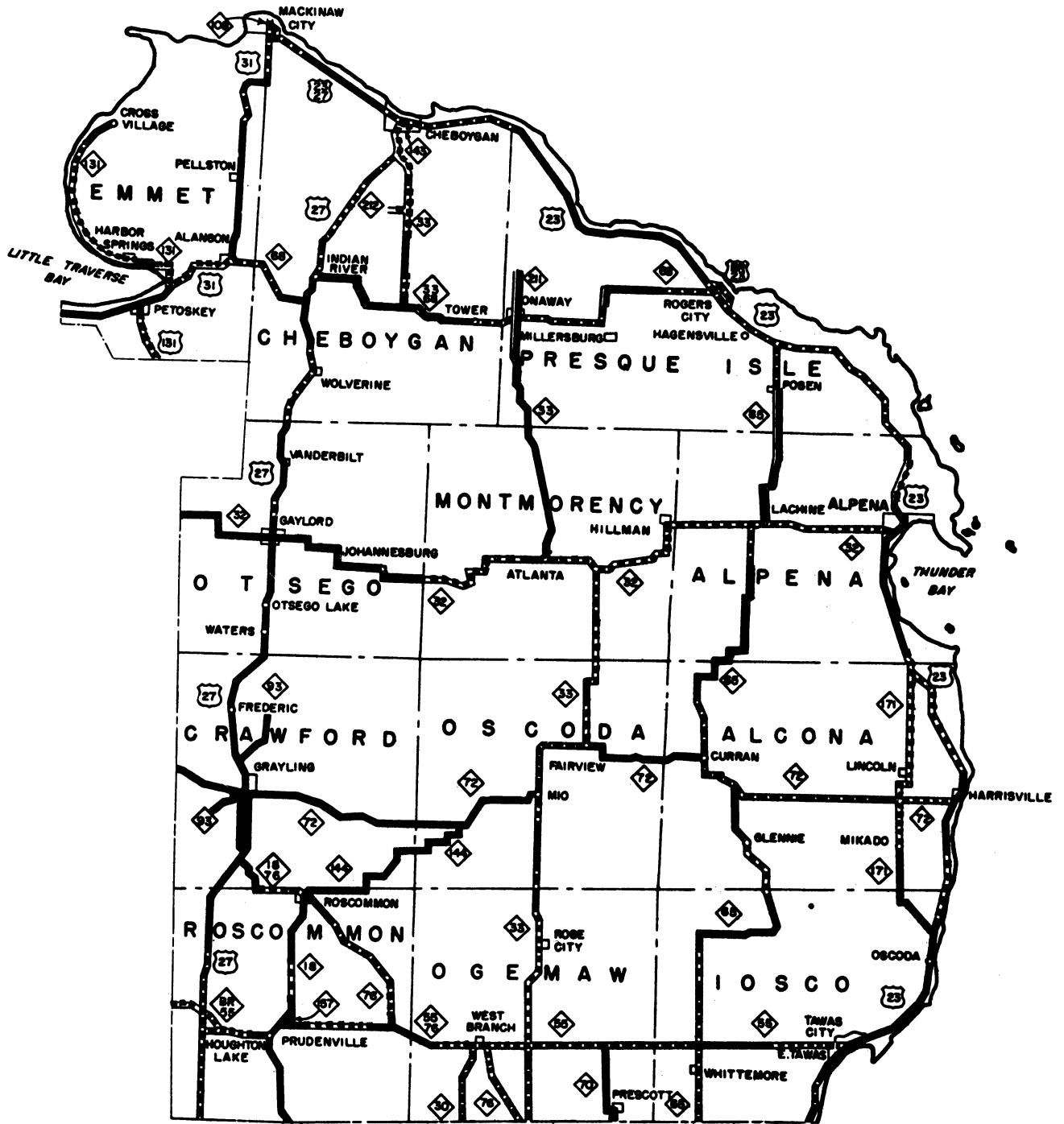


FIGURE- 1



DISTRICT NO. 4

ROAD CLASSIFICATION FOR LEGAL AXLE LOADS





-  No Seasonal Restriction - Pavement and Subgrade Adequate for Year-around Service.
-  No Seasonal Restriction - Pavement Designs Which Compensate for Seasonal Loss of Strength.
-  Spring Load Restriction Required - Pavement Designs Which Do Not Compensate for Seasonal Loss of Strength.
-  Spring Load Restriction Required - Pavement Design Inadequate for Legal Axle Loads at all Times.

FIGURE-2

preserving riding quality. However, of more importance in the present discussion, was the demonstration that the behavior of existing pavements under actual service conditions could be measured quantitatively in terms of pavement roughness and structural continuity. The results obtained on this first cooperative research program encouraged the later studies in which the problems of pavement design were attacked on a wider front. This program included several phases. In the first place, the Michigan State Highway Department established in 1958, for the first time, an "All Season" network of selected state trunklines on which the Spring load restrictions were to be lifted. In the second place, the Department joined with the University of Michigan in a cooperative research program involving a continuing pavement performance survey in which quantitative measures of pavement roughness and structural continuity on a large scale were to be accumulated.

The results of these surveys were then used to confirm pavement classifications and inclusion of selected projects in the "Frost Free" network, to provide a basis for reclassification of other roads, and for assessing the deficiencies in those roads not capable of carrying legal axle loads at all times. This project was sponsored by three trucking association groups which included the Michigan Trucking Association; the American Trucking Associations, Inc.; and the Motor Truck Division of the Automobile Manufacturers Association. Equipment was designed and built and procedures were developed for pavement performance surveys and this project ran for approximately two years, until July, 1959. Approximately 2000 miles of pavement profile were obtained and the results have been analyzed and reported in part in published form.^{1,2} It is from these data that examples have been selected for the present discussion to illustrate the value of a study of existing pavements as a guide to pavement design.

To fully appreciate the examples that will be given, it is necessary to review briefly the development of pavement design in Michigan over the past 35 years to serve as a background for present practice and procedures. During this period, the Department has accumulated a tremendous volume of information on pavement performance, maintenance, and correlation of behavior with soil conditions, drainage, and other environmental factors. The first step in the program of statewide pavement performance studies was to assemble all of the presently available information into a single statewide inventory of pavement adequacy. Four classifications were made with respect to the ability to support legal axle loads. These classifications are subsequently referred to as Class 1, 2, 3, and 4 and are shown on both Figs. 1 and 2 in the terms by which they are defined. Fig. 1 is a state map showing the first statewide pavement classification of January 1, 1958. Fig. 2 is a typical district map, indicating the classification of state trunkline roads within the several counties and made to a little larger scale, making it easier to identify the four road classifications which are defined as follows:

-
- 1 "Pavement Profile Surveys to Correlate Michigan Design Practice with Service Behavior", Proceedings, Highway Research Board, Vol. XXXVIII, pp. 149-177, 1959.
 - 2 "Legal Weight Limitations on Motor Vehicles", Presented at the 44th Annual Michigan Highway Conference, Grand Rapids, Michigan, March 17-19, 1959.

Class 1 (Solid black)

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 (Dot-dash black)

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 (Dot-dash red)

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 (Solid red)

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.

In this first pavement evaluation, covering some 9300 miles of the state trunkline system, approximately 55% of the pavements were rated as adequate for year-round service, leaving 45% as inadequate. The first practical result of significance from this program was the raising of Spring load restrictions on the selected network of state trunklines that were opened for full legal axle loads in the Spring of 1958. Some 4600 miles of highway, or approximately 50% of the state trunkline system, were included in the first year's "Frost Free" network. Due to new construction, betterments, and reclassification of pavements, the "All Season" network was increased in 1959 to 5784 miles, or a little more than 60% of the state system.

Fig. 3 is the map of the "All Season Trunkline Highways for 1960" showing a network of unrestricted roads amounting to 6240 miles which is two thirds of the total mileage in the state trunkline system. When this policy was inaugurated three years ago, the Michigan State Highway Department made a survey of state industries from which it was estimated that the Spring load restrictions cost industry in the state approximately \$20,000,000 each year; and, as recently pointed out by the State Highway Commissioner, the expanded "All Season" system has saved Michigan's industries, agriculture, and consumers many millions of dollars annually.

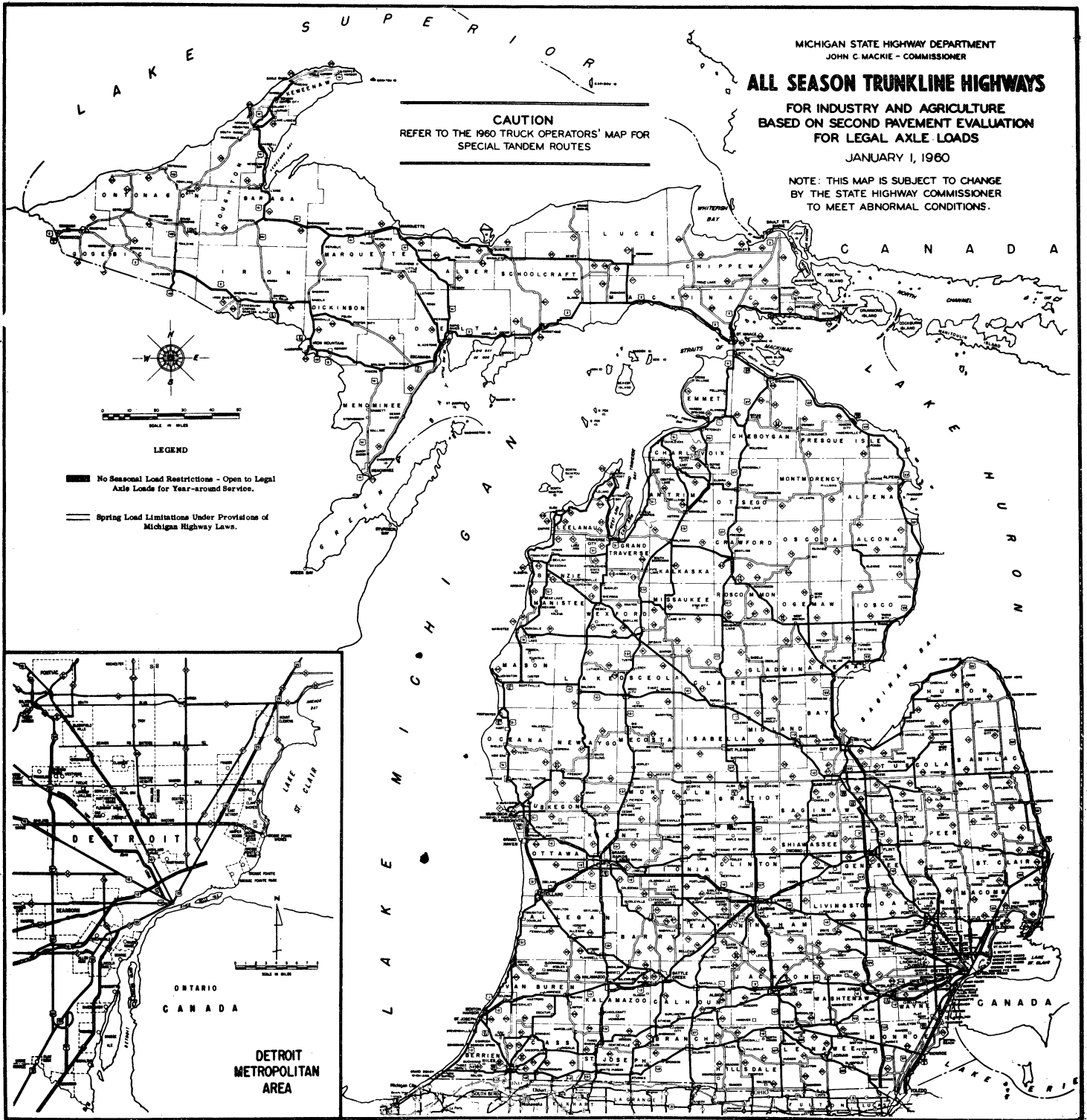


FIGURE - 3

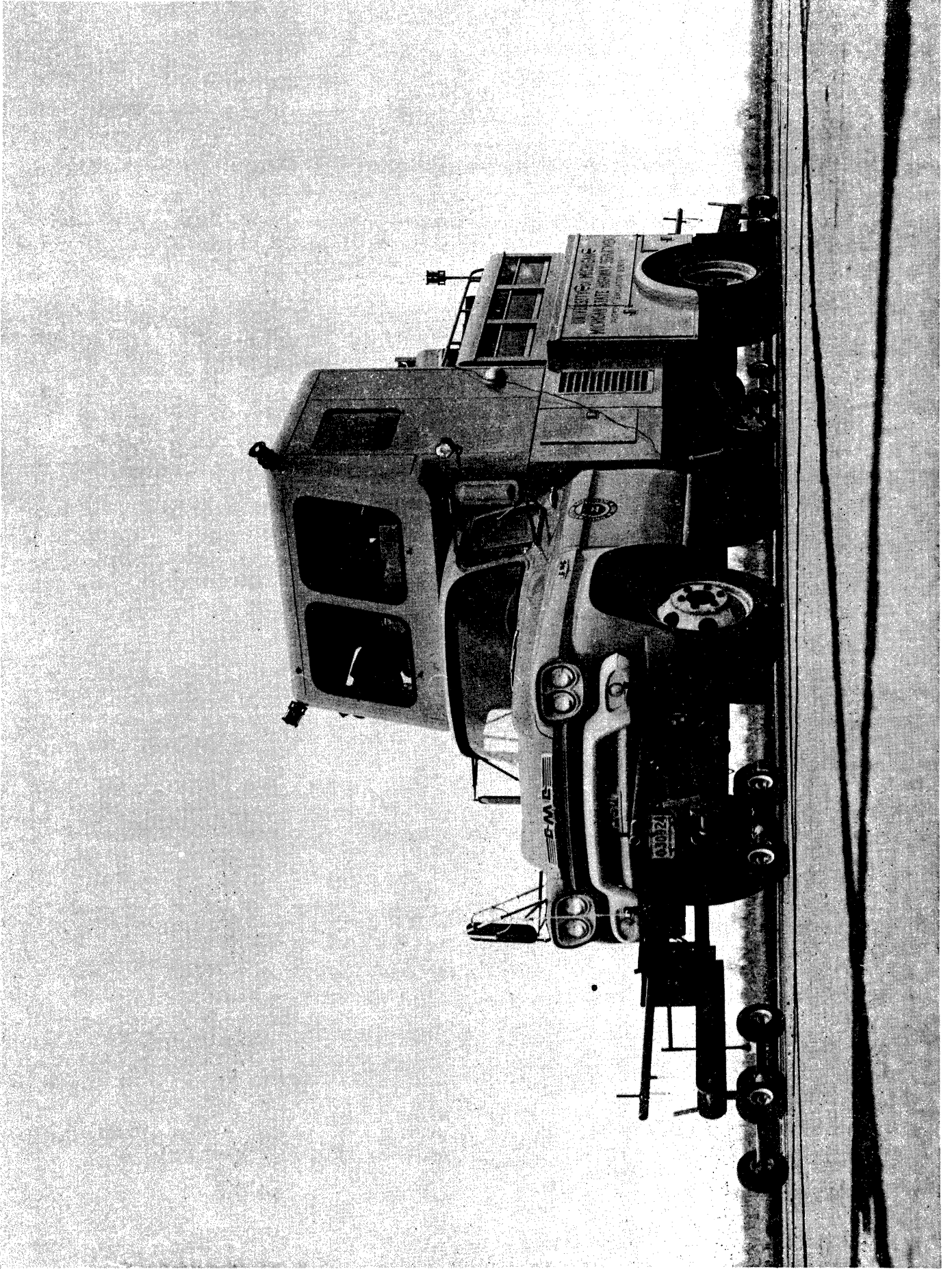


FIGURE - 4 MICHIGAN TRUCK - MOUNTED PROFILOMETER

PAVEMENT PROFILE SURVEYS

The essential piece of equipment for measuring and recording pavement profiles is the truck mounted profilometer or profilograph shown in Fig. 4. 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 in 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 these reference points. Pavement profiles are recorded on a continuous chart and may be retraced after a designated period of service to measure any progressive 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 per mile for any selected length of pavement. This cumulative vertical displacement in inches per mile is used as a roughness index in subsequent pavement classification. The roughness 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 for this program by F. N. Hveem, Materials and Research Engineer for California. In addition to recording the cumulative vertical displacement, equipment is provided for measuring and recording the location of cracks and joints to provide a measure of the structural continuity of the pavement surface. A device has also been provided to measure and record the relative elevation in both wheel tracks by what is called a "roll indicator" with a recording pen in the center of the profile chart. If progressive changes in the pavement profile are to be used to measure pavement performance, it is obvious that the profile trace at any one time and the recorded roughness index must be very accurately measured and recorded. One of the first questions which arises in this connection is, "Just what is it that the profile truck is measuring, and how accurately will it produce an actual pavement profile?" The answer to this question depends primarily on the selection of a datum or reference to which the measured vertical displacements may be referred. For the purpose of explanation, several pavement profiles taken on a test course at the Willow Run Airport have been plotted for comparison in Fig. 5.

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 from elevations taken from levels at intervals of 2.5 feet along the test course. The latter profile represents the vertical difference in elevation at the recording station at the center of the truck with respect to the average elevation of the bogie wheels at the front and the back of the truck. In other words, the reference plane for vertical displacement is represented by a straight line drawn from the center of gravity of the two sets of bogie wheels. This computed trace is considered to be most representative of the vertical displacements recorded by the profile truck, which is actually measuring deflection at the center of a 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, however, several other problems in measurement and instrumentation which are too involved to take up in the present discussion. Suffice it to say that the problem

is one of reproducing pavement profiles with sufficient accuracy to reflect significant differentials in the pavement surface and still keep the equipment itself within practicable limits. Data from successive pavement profiles which have been analyzed to date indicate that the profilometer equipment as now designed is capable of achieving this objective.

Typical Pavement Profiles

In order to illustrate the use of pavement profiles as a means of differentiating between adequate and inadequate roads on the basis of service behavior, a number of typical pavement profiles will now be presented. In reviewing these examples, there are several things to be kept in mind as a means of correlating the data on some comparative basis. In Fig. 6 is shown a tentative roughness rating to associate with the measured roughness indexes, as representative of varying degrees of riding quality from exceptionally smooth to extremely rough. Roughness indexes in excess of 200 inches per mile are considered unacceptable, although there are many examples of such roads in service. For those familiar with the more widely used single wheel roughometer, comparative roughness indexes have been established on the basis of a relatively small number of projects where both types of equipment have been used.

Pavement profiles are presented in a series of charts of which Fig. 7 is the first. These charts follow a definite format which will be explained at the outset, with no further reference thereafter. Each chart presents 600 feet of profile in the outer and inner wheel paths of the traffic lane. The outer wheel path is generally at the top of the sheet on the right hand side of the lane, with the direction followed by the profile truck generally being to the left. Cut and fill in feet is indicated at the top of the sheet. 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. It should be kept in mind that vertical displacements are recorded full scale (1 inch = 1 inch) while the 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 pavement grade or slope.

Pavement joints and cracks are shown along a horizontal line at the bottom of the sheet, above which the continuity ratio is given. The first figure is for the pavement as originally constructed, and the second figure, on the right, being computed for the combination of joints and cracks which develop after some period of service. In this connection, it should be noted that the continuity ratio is defined in terms of a standard slab length of 15 feet representing a continuity ratio of 1. Thus, an uncracked slab between hundred 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.

To the right of the pavement profiles are given pertinent data on the pavement section including pavement type and thickness of the various components, subgrade classification, drainage, road classification from the standpoint of adequacy, project identification, traffic count based on the 1957 survey, and the date of the profile survey.

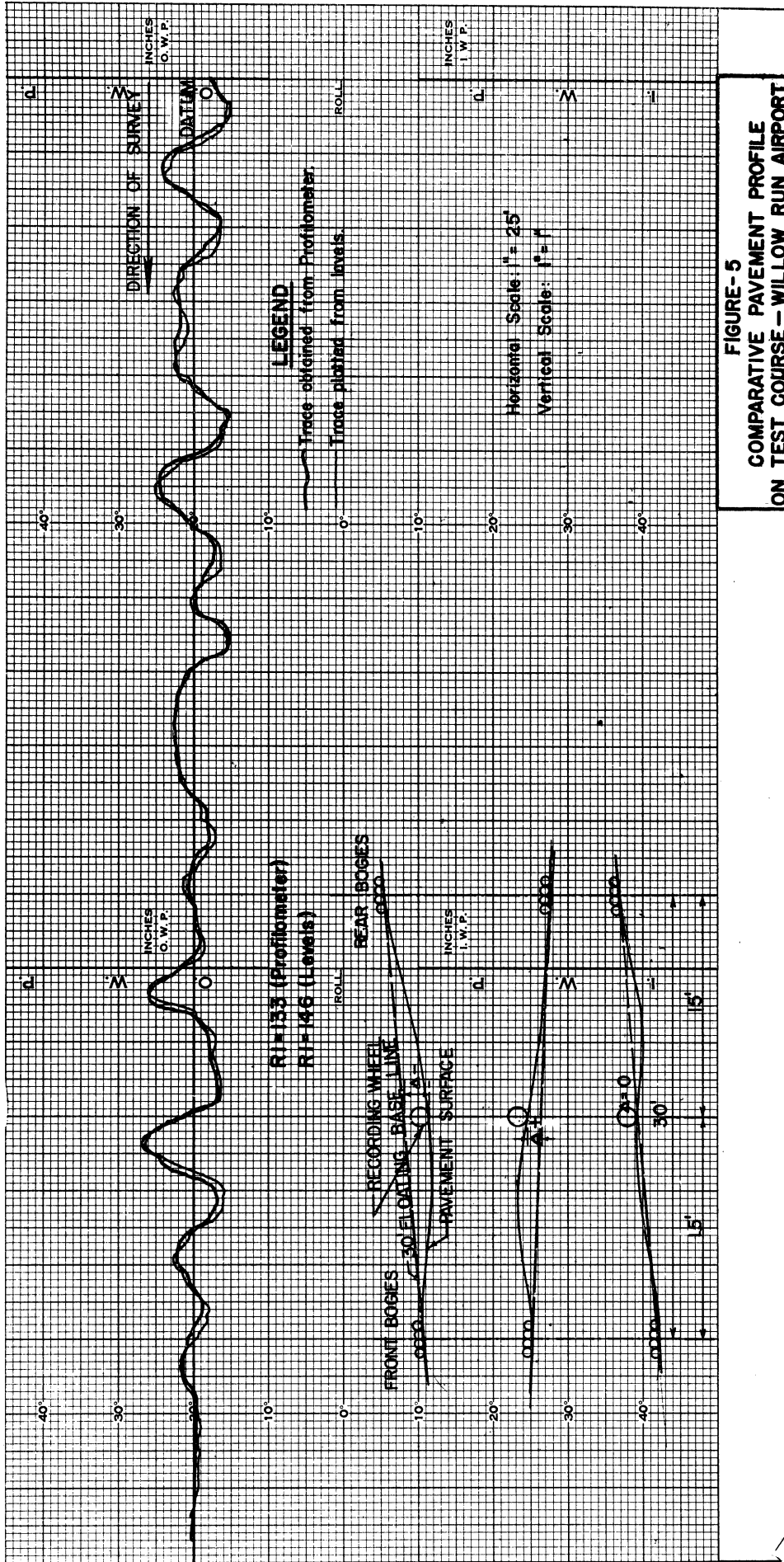


FIGURE-5
COMPARATIVE PAVEMENT PROFILE
ON TEST COURSE - WILLOW RUN AIRPORT

**TENTATIVE
PAVEMENT ROUGHNESS RATING
VERTICAL DISPLACEMENT - INCHES PER MILE**

U.S. BUREAU OF PUBLIC ROADS⁽¹⁾ <u>SINGLE WHEEL ROUGHOMETER</u>	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 4-5 MILES PER HOUR.

FIGURE - 6

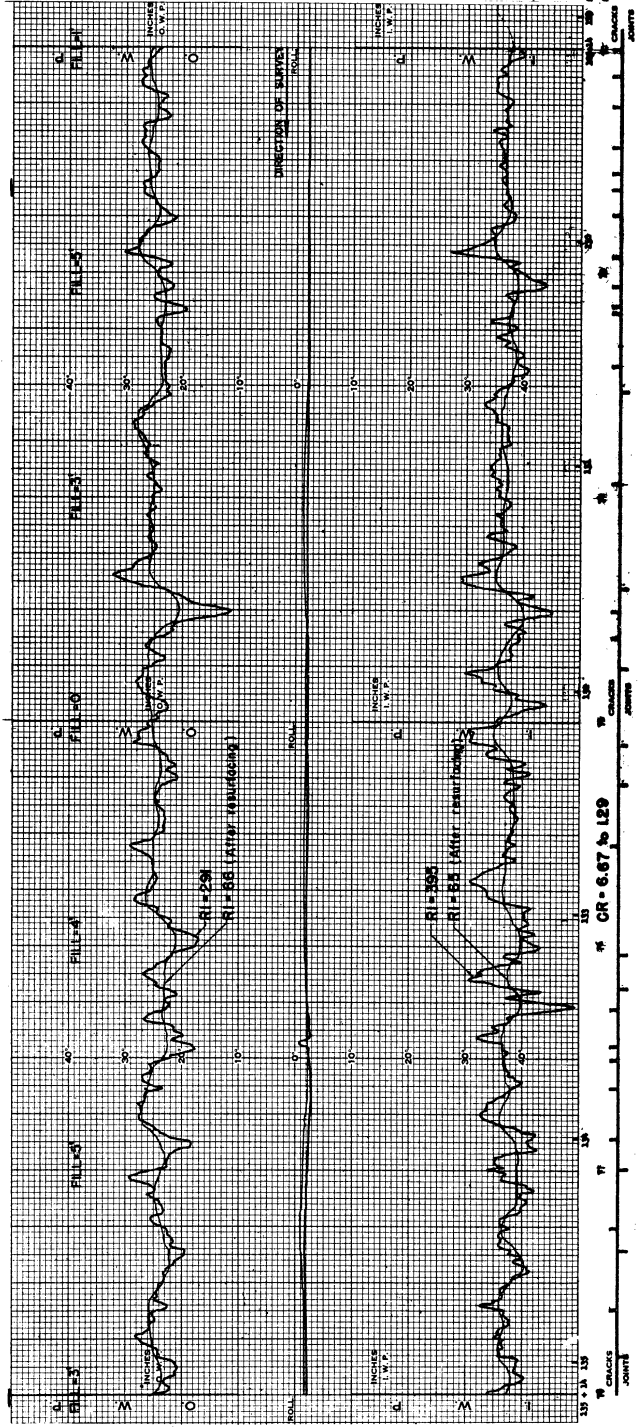
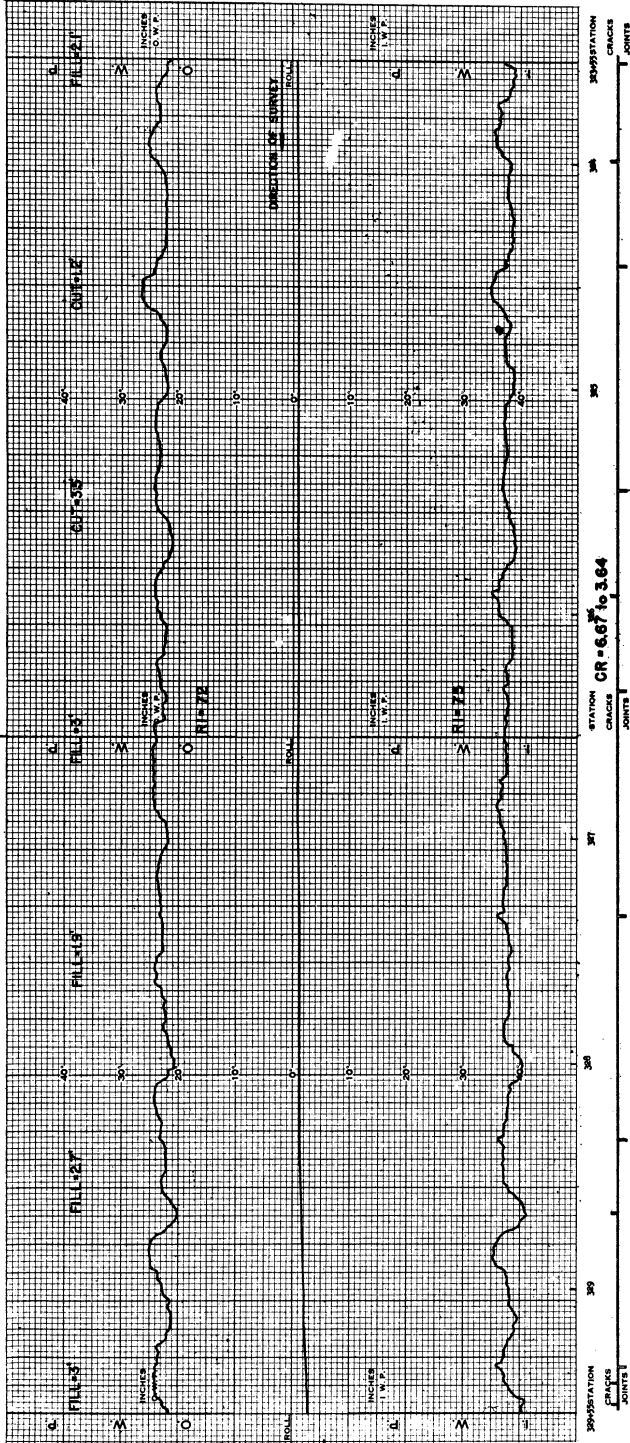
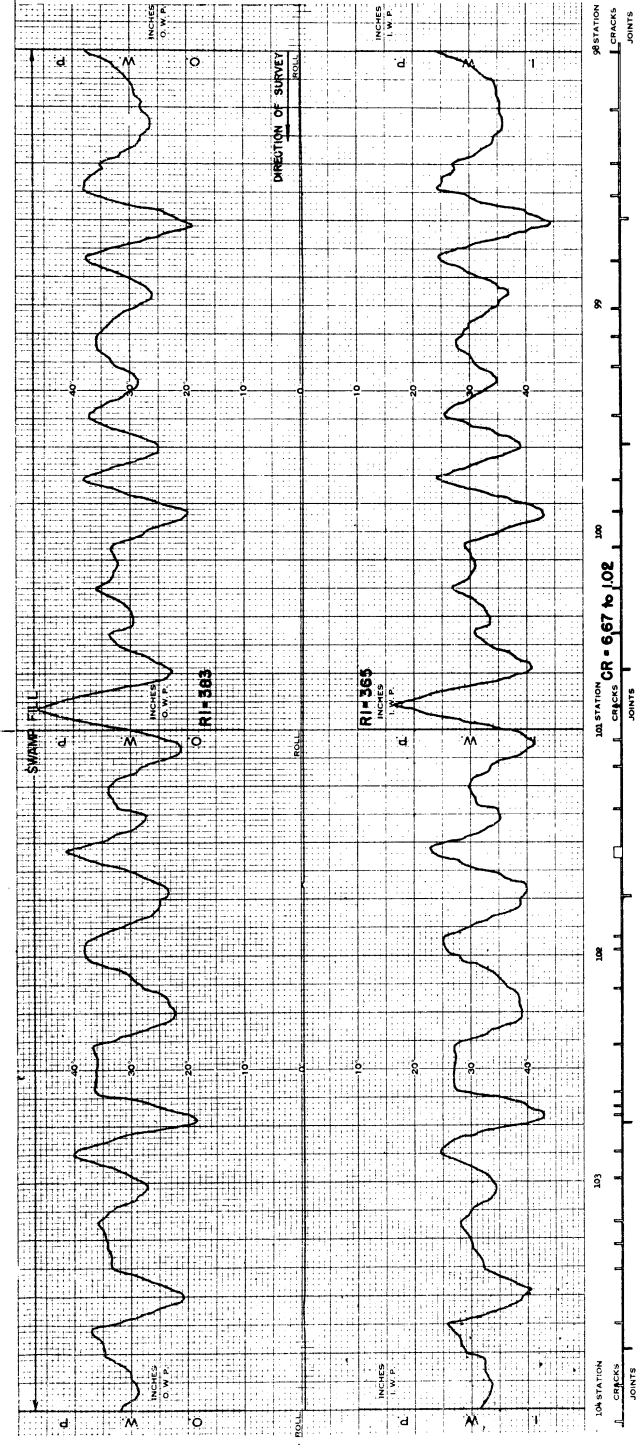


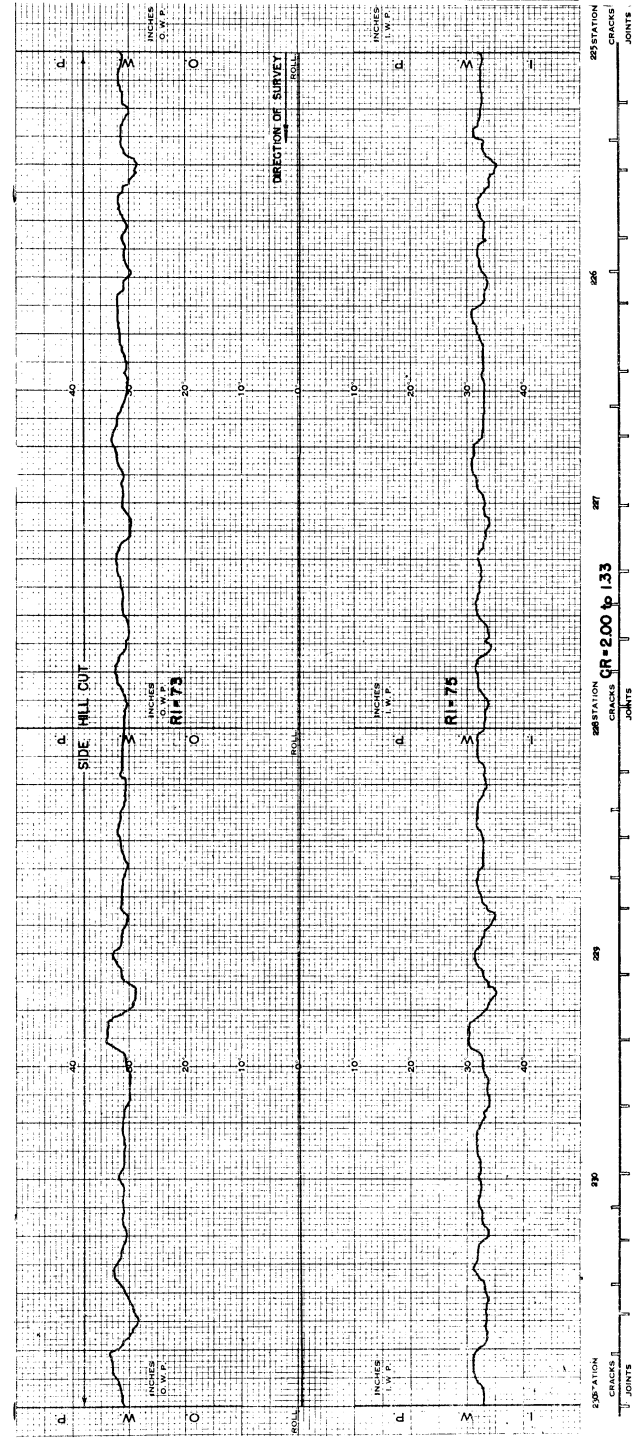
FIGURE - 7



9'-7\"/>

DRAINAGE: POOR

FIGURE-12 A
 CLASS-III RIGID
 US-41, 1936 SERVICE 22 YEARS
 TRAFFIC-TOTAL 2000, COMMERCIAL 280-550, DHV 310
 DATE OF SURVEY: NOV. 8, 1958



9'-7\"/>

DRAINAGE: GOOD

FIGURE-12 B
 CLASS-III RIGID
 US-41, 1936 SERVICE 22 YEARS
 TRAFFIC-TOTAL 1600, COMMERCIAL 280, DHV 190
 DATE OF SURVEY: NOV. 7, 1958

FIGURE - 8

Class 1 and Class 4 Rigid Pavements. The first two profile charts shown in Fig. 7 are taken from one of the older, heavily traveled roads in Michigan, US-112. The pavement profiles in Fig. 7-A, with roughness indexes of 72 in the outer wheel path and 75 in the inner, would still be rated very good from the standpoint of riding quality after some 32 years of service insofar as the original concrete pavement is concerned or 10 years of service on the bituminous resurfacing laid in 1948. The traffic is 4800 vehicles per day, of which 1525 are commercial. There are relatively few cracks between the 100 foot joints, with a reduction in the continuity ratio of something less than 50% since construction in 1926.

The pavement section in Fig. 7-B, which is only a short distance from the first section with which it is compared, has been in almost continuous trouble since it was built, requiring continuous maintenance with an unusual amount of concrete patching and continual cracking of the old pavement. After having been resurfaced with bituminous concrete in 1943, it developed the rough profile shown by the heavy line, with a roughness index of 291 in the outer wheel path and 395 in the inner wheel path. It was resurfaced in 1958, with the pavement profile indicated by the finer line superimposed on the profile before resurfacing. In connection with this resurfacing, it is interesting to note the degree to which vertical displacements are ironed out and the riding quality recovered. There is a much greater reduction in the continuity ratio in Fig. 7-B, but it should be noted that these data are not completely satisfactory. In a pavement that has been resurfaced as much as this, it is difficult to locate all the cracks in the original slab as many of them do not show through the resurfacing. The extensive amount of concrete patching also has been indicated as joints, producing the irregular spacing of joints that were originally 100 feet apart.

The significant difference in these two sections of pavement lies in the subgrade type and drainage. The pavement giving very good service, in Fig. 7-A, falls within a relatively small area of glacial outwash which is predominantly sandy with fair internal drainage. The example of Class 4 pavement, in Fig. 7-B, is characterized by a subgrade soil of sandy clay loam and poor drainage, a combination which provides inadequate subgrade support for this heavily loaded pavement.

Class 3 Rigid Pavement. The two pavement sections shown in Fig. 8 provide one of the most interesting comparisons found in the pavement profiles obtained during the first year. Both sections are taken from US-41 running north out of Menominee in the Upper Peninsula, and are only a few miles apart. This road, incidentally, is known as one of the roughest riding pavements in the state and the two sections selected represent the roughest and the smoothest sections in this stretch of highway. There is a startling differential in roughness index in these two sections, with the rough section being entirely outside the rating scale while the smooth section would be rated good or very good. This contrast is the direct result of a difference in subgrade soil and drainage. The rough section is over a swamp fill following peat excavation, with backfill material being described as a heavy sandy loam. The water table is close to the surface and in this area of deep frost penetration the combination of conditions promotes severe frost action. The smooth section, on the other hand, is on a side hill cut and fill with the subgrade material being described as a light sandy loam, sufficiently granular to supply adequate internal drainage. This in combination with the side hill surface drainage has produced very good performance over a service period of 22 years. Similar differentials are shown in the changes in structural continuity, although, to some extent, the small amount of cracking in the good section is the result of the 30 foot joint spacing.

Flexible Pavements

During the first year's survey, the attempt was made to obtain profile data on representative pavements of the various types subjected to the range of variation in soil type and traffic service. The first two examples were of rigid concrete pavements and the next two will be typical flexible or bituminous type pavements.

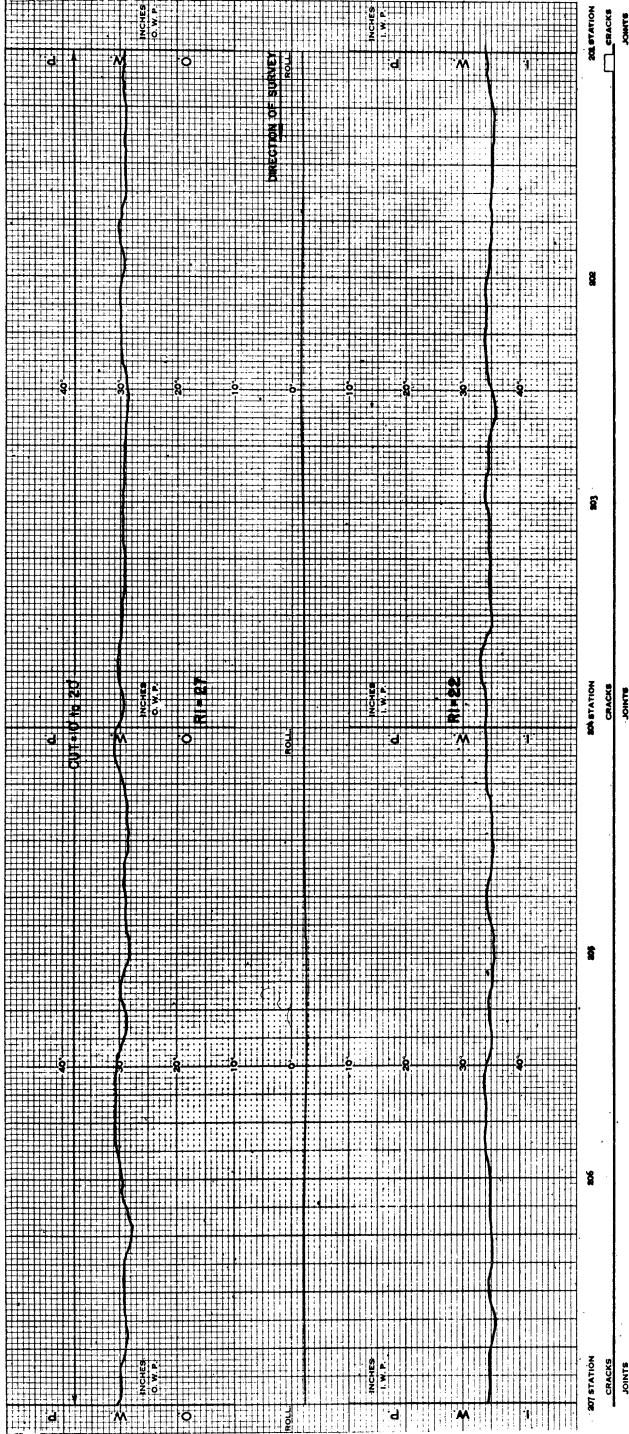
Class 1 and Class 4 Flexible Pavements. The two pavement sections shown in Fig. 9 are representative of high type bituminous construction which, on the basis of performance, is being given increased consideration in the state. Both are on superior type subgrades of a sand outwash with excellent drainage characteristics. In both cases, gravel bases were constructed first and subjected to traffic for a sufficient period of time to be thoroughly compacted before laying the bituminous wearing course. In terms of riding quality, both would be rated exceptionally smooth -- indicating excellent construction practice as well as completely adequate 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 the 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 foundation are important factors in the superior riding quality that has been produced.

One other special circumstance should be noted and that is that the pavement in the lower part of Fig. 9 was resurfaced in 1956 because, after a period of some 20 years, surface abrasion and hardening of bituminous material had produced a fragile surface which either had to be sealed or resurfaced. This, combined with the necessity for widening the road from 20 to 22 feet, led to the use of a bituminous concrete retread; but, insofar as the structural capacity of the pavement is concerned, there was no indication before resurfacing of loss in load carrying capacity resulting in increased pavement roughness.

In Fig. 10 is shown a Class 4 flexible pavement that supplies another study in contrast between two sections taken from the same road just a few miles apart. This is an old road, having been built up over a period of years by county forces and later by the State Highway Department when the road was taken over as a trunkline. The comparison is mainly of interest as a demonstration of the dominant influence of a good sand subgrade over a comparatively limited stretch as compared to a poorly drained, sandy clay subgrade. In the good section, after 19 years of service, the riding quality is still rated as good; whereas, on the weak section, there are no terms included in the rating table which would provide an adequate description. These two examples would also seem to be another illustration of the value of the pavement profilometer in differentiating between those sections which would eventually have to be rebuilt and those which would not require rebuilding, thus adding a quantitative tool to supplement good engineering judgment.

Special Sections

Several special pavement sections have been selected to illustrate examples of pavement design and construction which are of particular interest. The top profiles in Fig. 11 show the effect of using short, 20 foot slabs of plain



2 1/2" BITUMINOUS AGGREGATE - 1955
 1/4" BITUMINOUS SEAL COAT - 1954
 7" GRAVEL BASE (22A) - 1954
 3"-5" GRAVEL
 SUBGRADE: SAND

DRAINAGE: EXCELLENT

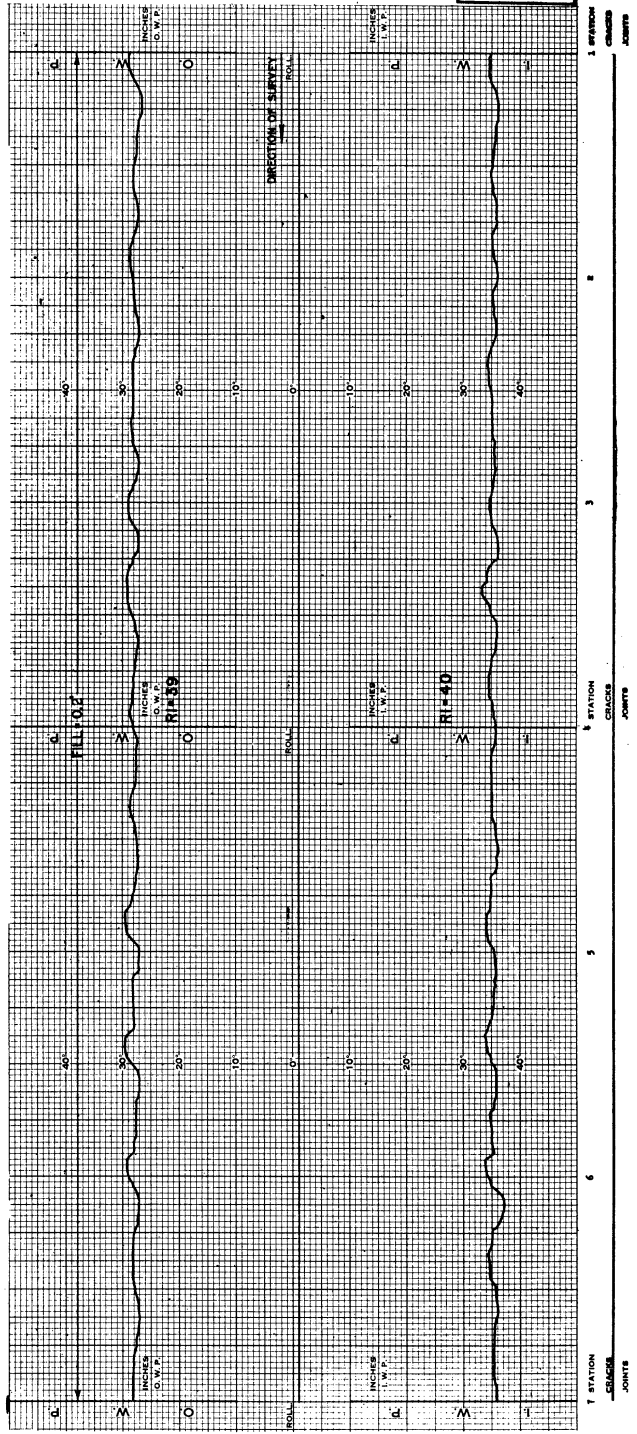
FIGURE-13A

CLASS-I FLEXIBLE

M-37, 1955 SERVICE 3 YEARS

TRAFFIC: TOTAL 500, COMMERCIAL 120, DHV 95

DATE OF SURVEY: SEPT. 11, 1958



2 1/2" BITUMINOUS CONCRETE - 1956
 2 1/2" BITUMINOUS CONCRETE - 1936
 8" GRAVEL - 1932
 2" EARTH FILL
 SUBGRADE: SAND

DRAINAGE: EXCELLENT

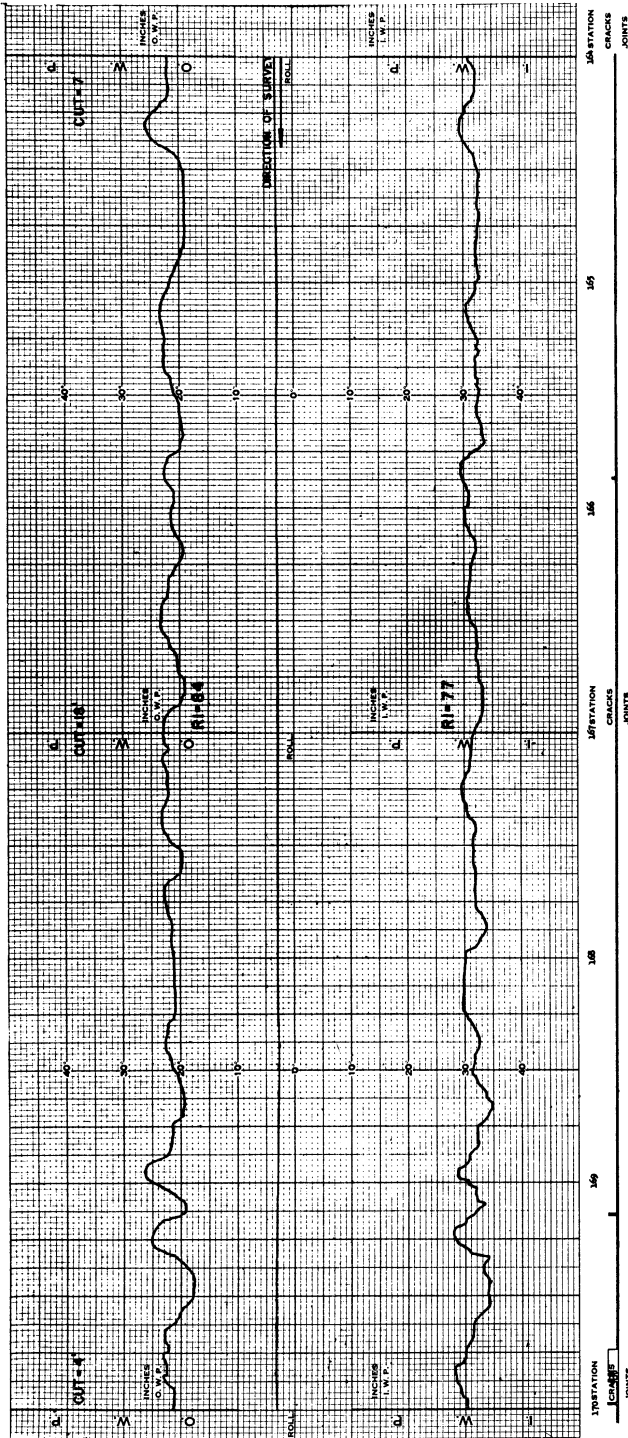
FIGURE-13B

CLASS-I FLEXIBLE

M-55, 1936 SERVICE 22 YEARS

TRAFFIC: TOTAL 900, COMMERCIAL 200, DHV 125

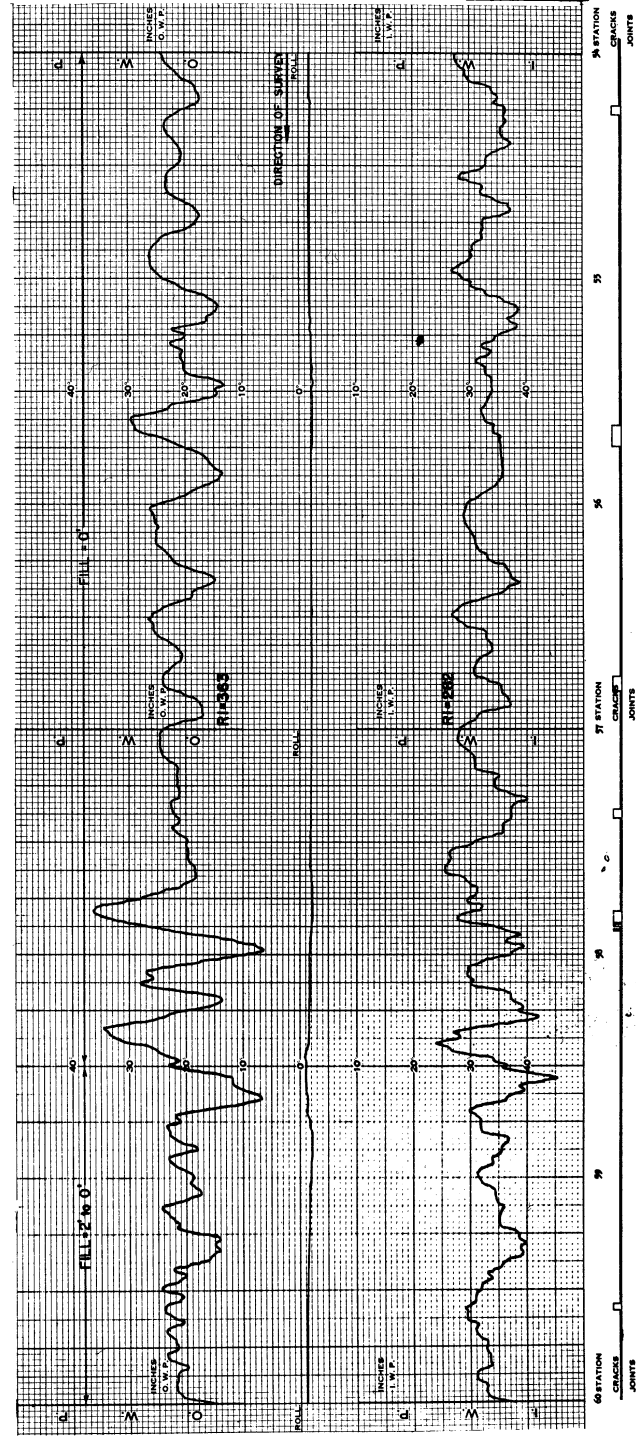
DATE OF SURVEY: SEPT. 12, 1958



1/4" SURFACE TREATMENT-1950 & 54
 2 1/2" OIL AGGREGATE-1939
 8 1/2" SUBBASE: GRAVEL
 SUBGRADE: SAND

DRAINAGE: EXCELLENT

FIGURE-16 A
 CLASS-IV FLEXIBLE
 M-36, 1939 SERVICE 19 YEARS
 TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
 DATE OF SURVEY: DEC. 5, 1958



1/4" SURFACE TREATMENT-1950 & 54
 2 1/2" OIL AGGREGATE-1939
 11" SUBBASE: GRAVEL
 SUBGRADE: SANDY CLAY

DRAINAGE: POOR

FIGURE-16 B
 CLASS-IV FLEXIBLE
 M-36, 1939 SERVICE 19 YEARS
 TRAFFIC: TOTAL 2000, COMMERCIAL 385, DHV 250
 DATE OF SURVEY: DEC. 5, 1958

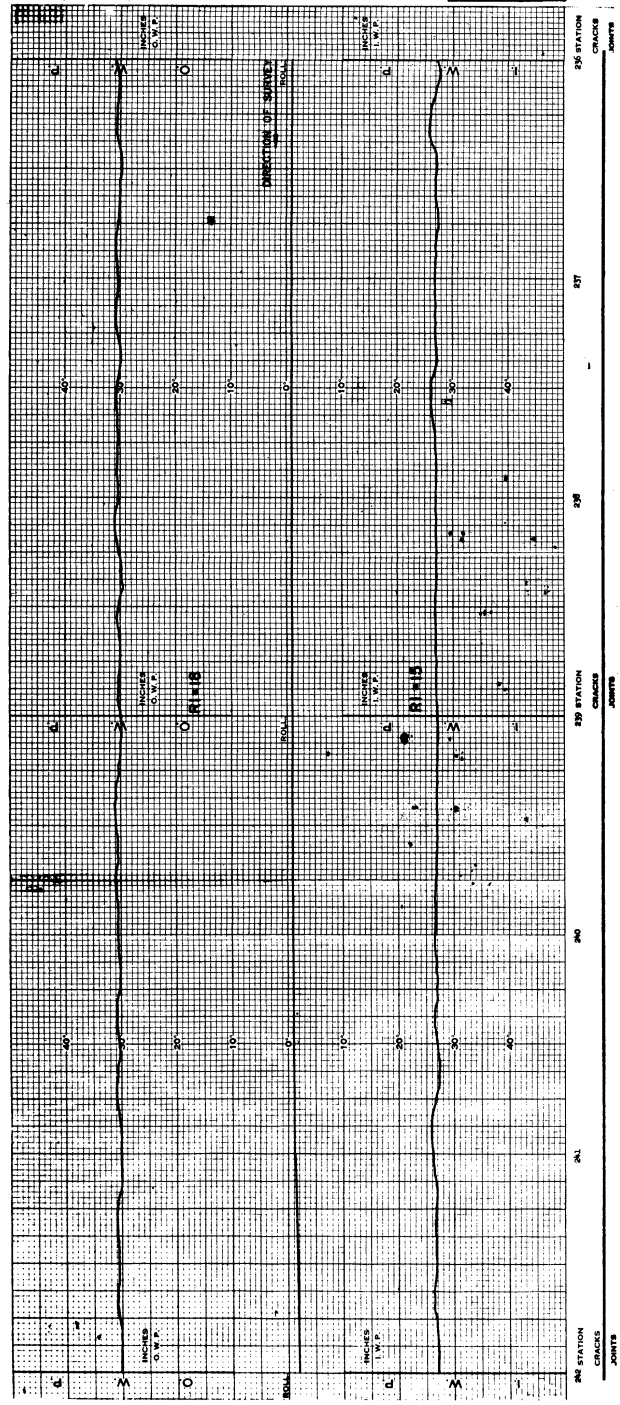
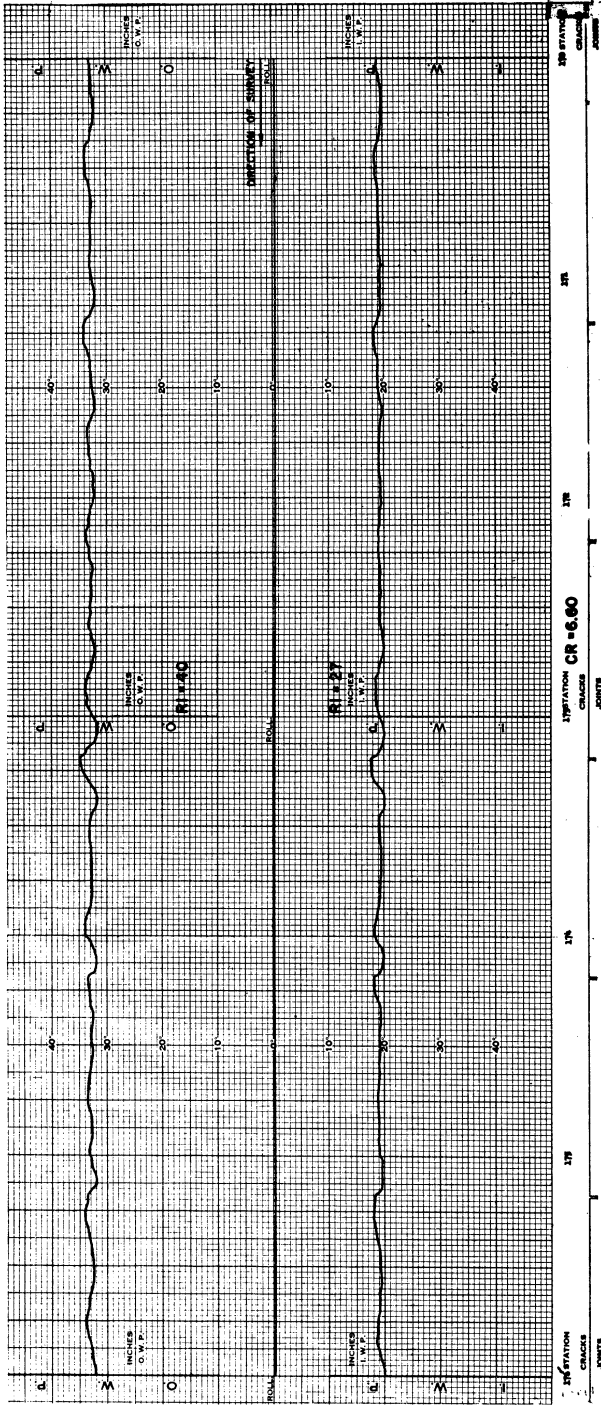


FIGURE-12

concrete without load transfer at the joints in what is presumably an effort to control pavement 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 subbase was added on top of the grade to protect the 9 inch plain concrete from quite certain pumping action. The small reduction in the continuity ratio indicates that the crack control was excellent, but this design produced one of the roughest riding pavements in southern Michigan. Perhaps 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 the value of pavement profiles in identifying different types of pavement distress. From the standpoint of design, the absence of load transfer at the joints is probably the critical weakness. For purposes of comparison, the profile shown in the bottom half of Fig. 11 is from the smooth section of US-41 where there are 30 foot joints, but with load transfer provided. Expansion joints at 60 feet are supplied with trans-load expansion joint base and the reinforcing is carried through the 30 foot dummy joints, adding necessary structural continuity to the concrete pavement.

The last two pavement sections, in Fig. 12, are examples of new pavements constructed in 1958 with profiles that give some measure of the high quality of pavement construction that can be produced when the effort is made to do so. The section in the top half of the figure is a reinforced concrete pavement on a divided lane highway, M-20 between Bay City and Midland, with a 12 inch granular subbase consisting of 9 inches of sand and 3 inches of selected gravel used for subgrade stabilization. This pavement would be rated as exceptionally smooth, with roughness indexes of 40 and 27 in the two wheel tracks. In the lower half of the figure is an example of a heavy duty flexible pavement constructed on the Muskegon - Grand Haven Expressway and it is representative of the type of heavy duty flexible pavement construction proposed for use on the Interstate Highway System on certain selected projects. The riding quality of the pavement is exceptional, with roughness indexes of 18 and 15 in the two wheel paths of the pavement.

Having obtained the basic profiles immediately after the pavements were constructed, these projects will be watched with considerable interest over the next few years and will provide the type of data needed to record the complete history of pavements during their useful period of service.

FROST ACTION ON PAVEMENTS

During the winter of 1958-59, a special investigation was made of selected sections of pavement in southern Michigan and in the Upper Peninsula to determine the effect of a one year cycle of freezing and thawing. The selected sections included all four classifications of both rigid and flexible pavements, with the primary objective of determining the effectiveness of pavement design in protecting pavements from the detrimental effects of frost action. The results of this investigation were very interesting and somewhat surprising and will be illustrated in the next few figures.

In Fig. 13 are shown some typical sets of profiles of sections of a flexible pavement on Highway M-100 in southern Michigan. In the top part of the figure is a well designed flexible pavement built on an old grade which had previously been inadequate for more than light traffic. There are several

weak sections in this pavement, shown in the bottom part of the figure, which have led to its being identified as a Class 3 pavement even though its design should justify a Class 2 rating. The good sections are actually Class 2 pavement which are completely adequate for year-round service.

The heavy full line on these profiles is for the late Summer or Fall survey, representing the relatively stable condition of the pavement surface. The good section is exceptionally smooth from the standpoint of riding quality, with a roughness index of 34 in the outer wheel path and 40 in the inner wheel path. The second, lighter, full line is the late winter profile taken at a time presumed to be close to the period of maximum frost action. The roughness indexes of 130 in the outer wheel path and 98 in the inner wheel path show a significant amount of frost displacement which is also apparent from the profile itself. The amount of frost displacement in a well designed road with presumably free draining, granular subbase and base courses was somewhat surprising, but these results were confirmed on similar sections throughout the investigation. The differential between the outer and inner wheel paths indicates the source of water as infiltration from the shoulder, although on this and other sections it was also apparent that water was moving through the granular base course in vapor phase and condensing under the surface to produce the moisture that upon freezing resulted in the displacement of the pavement surface.

The next profiles in the series, shown by the dashed line in Fig. 13, were run in the Spring after all frost had left the ground and had as their objective the determination of the permanent or residual displacement resulting from the winter's freezing and distortion of the pavement surface. In the good section, the results are rather satisfying as they indicate that the pavement surface went back to very nearly its stable Summer profile, with roughness indexes of 40 and 44 in the outer and inner wheel paths, respectively, as compared to 34 and 40 in the previous summer. While the residual or permanent displacement due to the single cycle of freezing was measurable, it was very small in this particular case which was exceptional. In most of the pavement sections included in this investigation, the residual displacement was substantially greater than in the examples shown in Fig. 13.

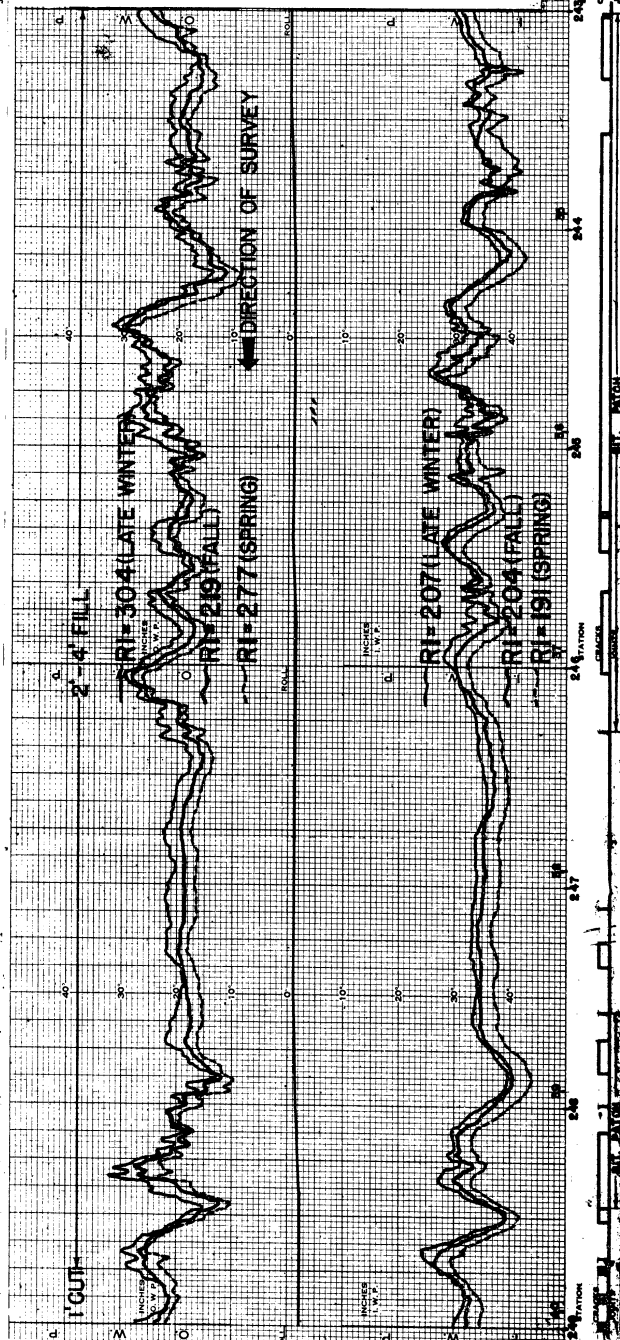
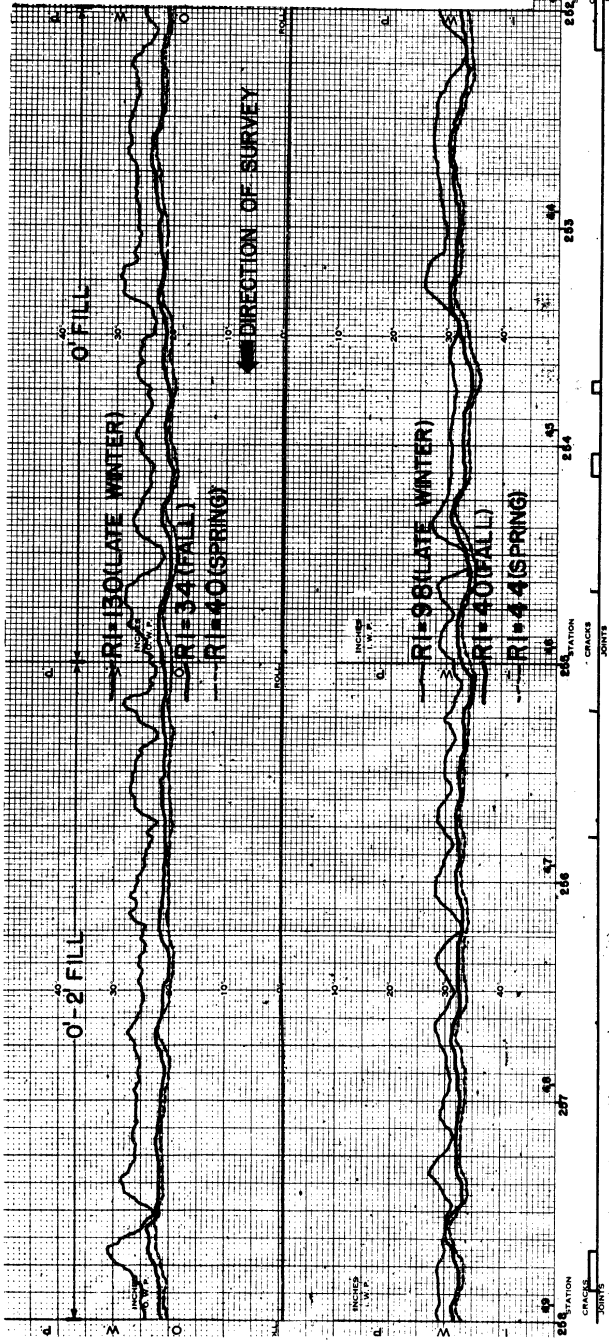
The rough section in the lower part of the figure requires some special comment. The frost displacement is again maximum in the outer wheel path with no significant change in the inner wheel path. One may surmise that after a pavement gets very rough it can get no worse and reaches some limiting value about which it may continue to fluctuate. The Spring profiles indicated that a substantial part of the frost displacement in the outer wheel path becomes permanent or residual displacement. In the inner wheel path, the Spring profile actually had a smaller roughness index than in the initial survey during the previous summer. The negative differential is small and may be experimental error in recording. This could result from a change in periodicity of the wave-like displacements, sluggishness in the recording mechanism, or failure to follow precisely the same wheel path. On the other hand, similar results were obtained on a number of the very rough pavements, so it may actually represent a true change in the pavement profile. It is not unreasonable to suppose that traffic may have a tendency to iron out the larger and more abrupt pavement displacements in very rough sections. In passing, it should be noted from the standpoint of design that the rough section resulted from settlement over peat trapped beneath the roadway and temporary patching to fill in the depressions.

2 1/2" BITUMINOUS AGGREGATE - 1953
 5" GRAVEL BASE - 1953
 12" SUBBASE: SAND
 4" OLD GRAVEL ROAD (1917-1946)
 SUBGRADE: SANDY CLAY
 DRAINAGE: POOR

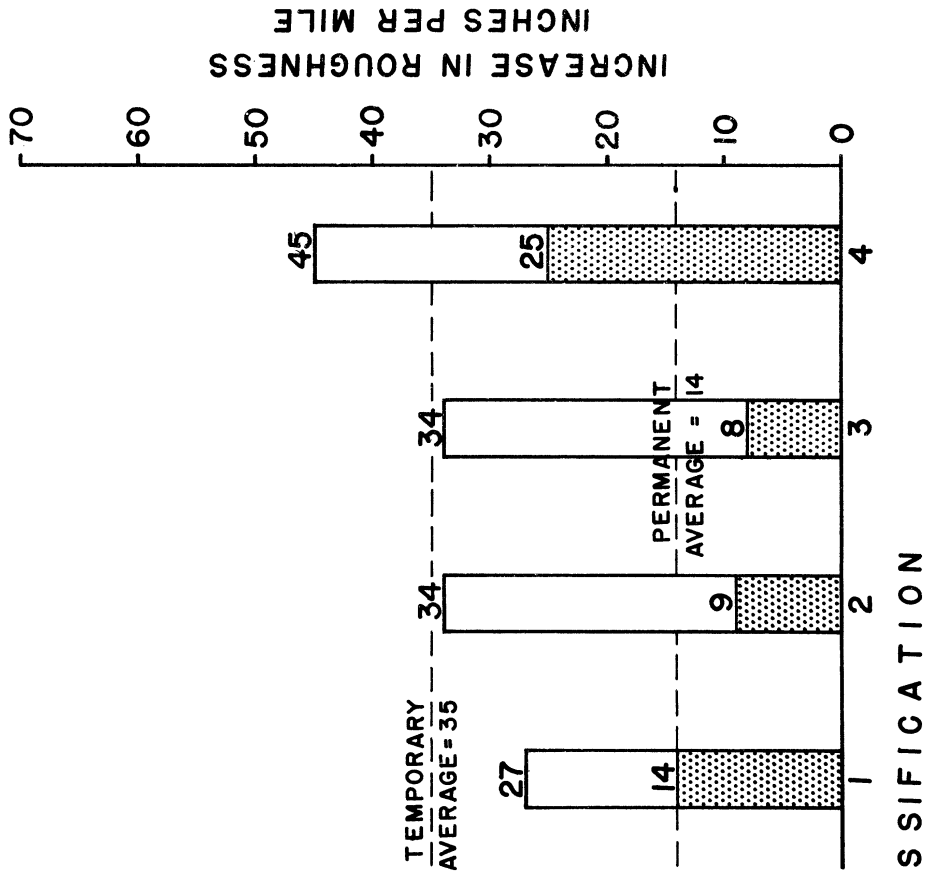
FIGURE - 15 C
 CLASS-III FLEXIBLE
 M-100,1953 SERVICE 5 YEARS
 TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
 DATE OF SURVEY: DEC. 4, 1958 & FEB. 20, 1959

2 1/4" BITUMINOUS AGGREGATE - 1953
 4" GRAVEL BASE - 1953
 12" SUBBASE: SAND & GRAVEL
 4" ORIGINAL GRAVEL ROAD
 2'-4" SANDY CLAY
 PEAT
 DRAINAGE: FAIR TO POOR

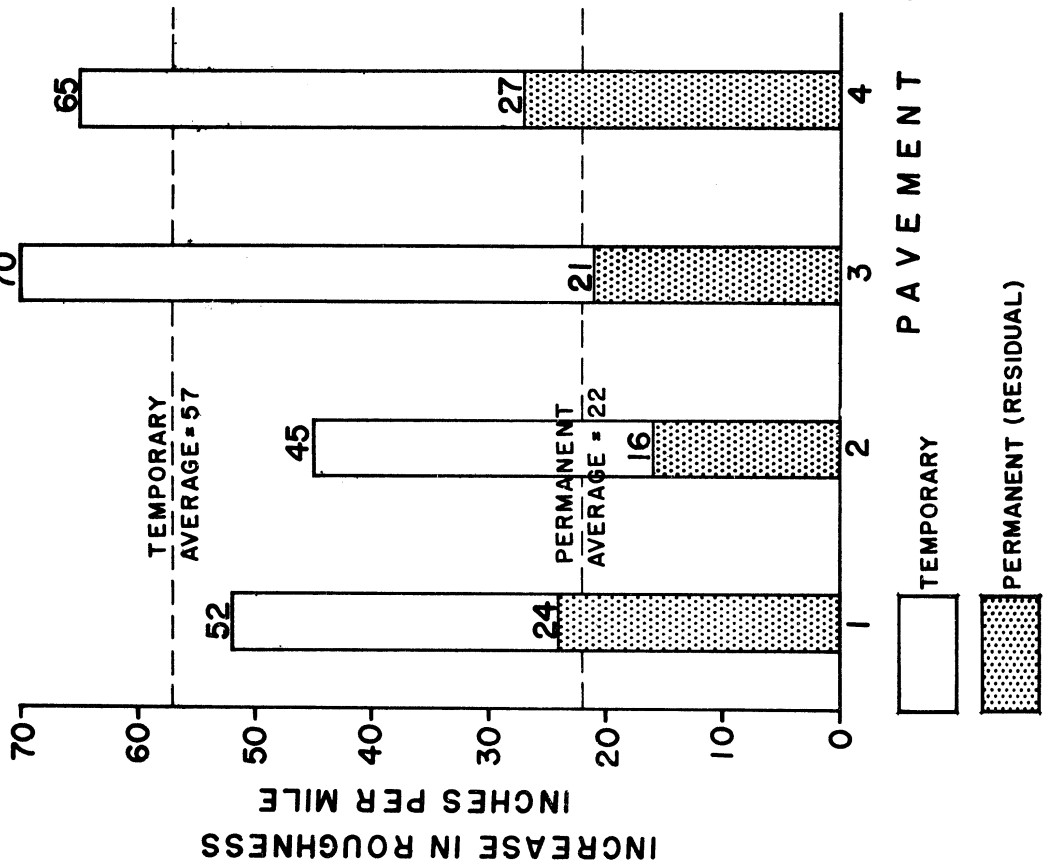
FIGURE - 15 D
 CLASS-III FLEXIBLE
 M-100,1953 SERVICE 5 YEARS
 TRAFFIC: TOTAL 800, COMMERCIAL 160, DHV 200
 DATE OF SURVEY: DEC. 4, 1958 & FEB. 20, 1959



INNER WHEEL PATH



OUTER WHEEL PATH



SEASONAL FLUCTUATION IN PAVEMENT PROFILE

FIGURE - 14

Fig. 14 is a graphical presentation of the average frost displacement on sixteen pavement sections of various classifications in southern Michigan. The vertical ordinate is the increased roughness in inches of vertical displacement per mile, greater than the normal, summertime profile. The classification of the pavement is indicated on the horizontal axis and the left hand group of bar graphs is for the outer wheel path and the right hand group for the inner wheel path. The maximum ordinates in both cases are the maximum frost displacement averages in late winter. It may first be noted that adequacy of the design from the standpoint of load carrying capacity makes little difference in the susceptibility to frost displacement. On the lower part of each bar is shown the permanent or residual pavement displacement caused by the distortion during the period of maximum freezing but remaining in the pavement after the frost has left the ground. The permanent or residual displacement is shown by the cross-hatched portions of each bar and indicates the increased roughness remaining in the late spring profile. The subsequent late summer profile indicated some minor changes in these figures, but no significant trends from the data now available. One rather definite difference is shown in Fig. 14 in the differential between the outer wheel path and the inner wheel path, which, as previously mentioned, indicates that the maximum heaving in the outer wheel path suggests that the source of displacement is the freezing of moisture which has infiltrated from the shoulder of the pavement. This observation also identifies the moisture as being in the base and subbase rather than deep-seated heaving in the underlying subsoil.

Cumulative Roughness over a Period of Years

In the analysis of the tremendous volume of data included in 2000 miles of pavement profile, there are obviously a great many studies and comparisons that may be made. While these data are still under examination, there are certain significant trends that immediately become apparent and some of these will be presented in the next few figures. In Fig. 15 is shown the roughness index values for all of the Class 1 and Class 2 rigid pavements included in the 1958 survey. The roughness index in inches of vertical displacement per mile is the ordinate and the number of years in service is plotted as the abscissa. In the center of the figure is shown a scale indicating the rating of the pavement in accordance with the roughness scale in Fig. 6. The data on the left hand side of Fig. 15 are for Class 1 pavements which are presumed to be the highest rating from the standpoint of structural adequacy of any of the pavements on the trunkline system. In general, they are pavements built over natural sand and gravel subgrades of high internal stability and excellent drainage conditions. Class 2 pavements, on the right hand side of the figure, are also structurally adequate, being designed for year-round service. However, they required subgrade improvement as represented by the addition of a granular base course over clay soils with drainage improvement through raises in grade, removal of frost susceptible soils, and subdrainage where required.

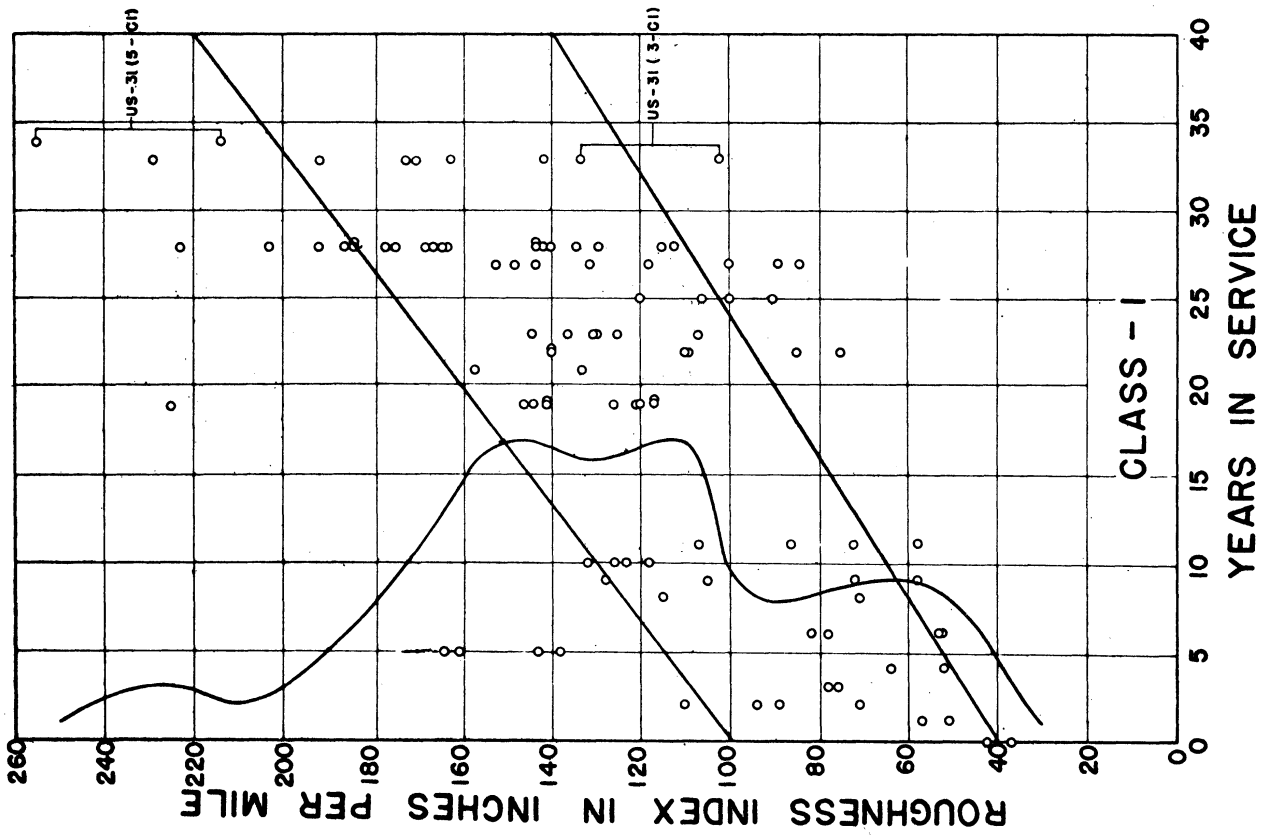
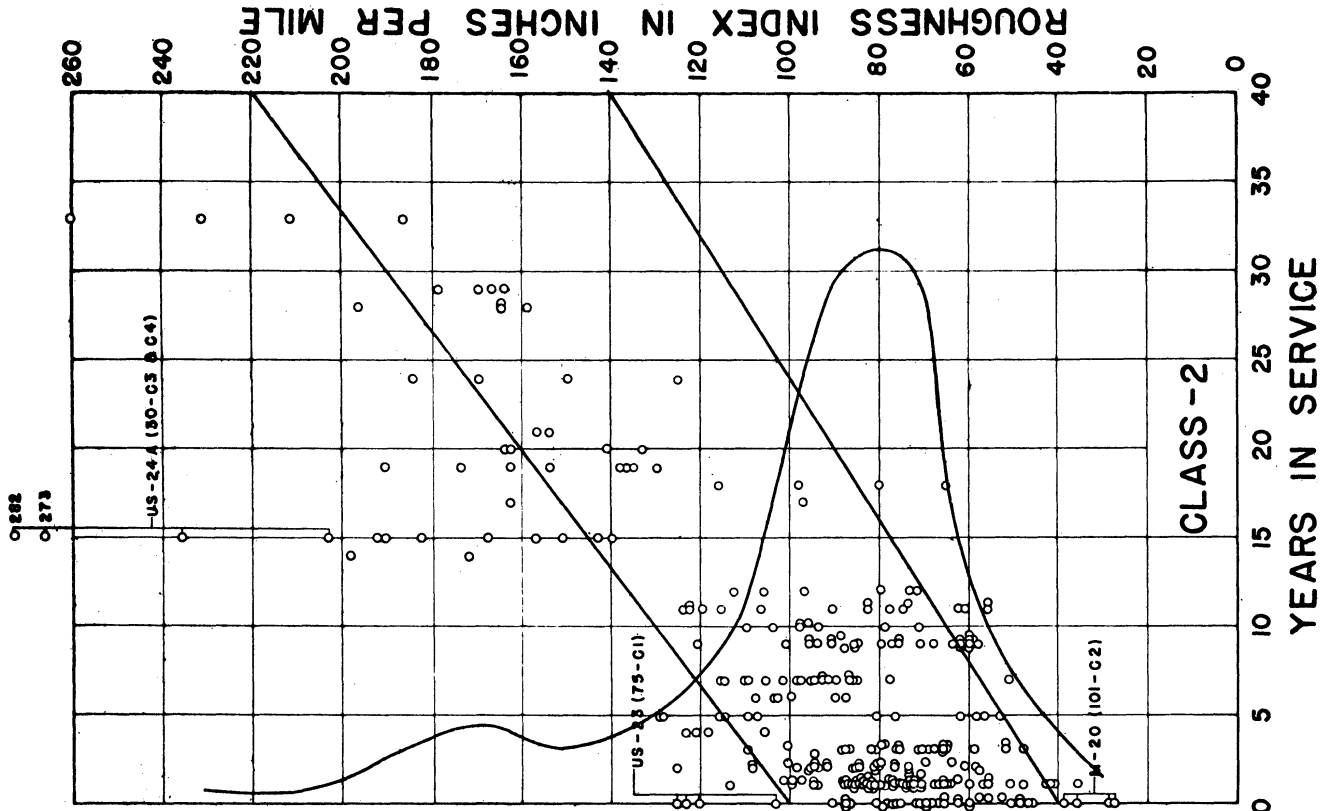
In spite of their structural adequacy, it is perhaps surprising to find that these pavements continue to lose their riding quality at a more or less uniform rate throughout their entire period of service. There are, on the chart, pavement sections from those most recently built to some that have a period of service approaching 35 years. If this is true for those pavements which are rated as structurally adequate for legal axle loads at all times of the year, the question naturally arises, "What is the normal performance of a pavement, and what is to be expected of a pavement that is structurally

adequate in every sense of the word?" While it is too early to draw conclusions from presently available data, further analysis of these observations may suggest some concepts of service behavior of pavements not generally recognized.

Several rather tentative procedures have been attempted in an effort to represent trends in the data in Fig. 15. As a first attempt, a band has been selected on the chart between the two straight lines shown, within which approximately 80% of the roughness index data will fall, with 10% of the more scattered points lying both above and below the band. If normal performance may be said to lie within this band, it would indicate that these well designed pavements lose riding quality or become rougher at a rate of from 2 to 3 inches of vertical displacement per mile for each year of service. This does not seem like a rapid rate of deterioration; but, nevertheless, it means that pavements which may be built within a range of roughness from 40 to 100, rated as good or very good in terms of riding quality, will in 40 years develop a roughness index from 140 to 220 which would rate them as poor to extremely rough in terms of riding quality. This change, which may presumably take place in spite of good design and superior construction, would appear to be consistent with the cumulative residual or permanent displacement of the pavement surface observed in the annual cycle of freezing and thawing already discussed in connection with frost displacement. This first attempt to define what may be normal behavior of a structurally adequate pavement will be left as food for thought while attention is directed to some other interesting features of the data in Fig. 15.

In addition to the suggested band of normal behavior, two distribution curves have been sketched in on the two charts in Fig. 15 and these show quite a contrast in distribution of the pavement sections with respect to their roughness indexes. Class 1 pavements on the natural sand and gravel subgrades are spread out over a much wider range of roughness index than the Class 2 pavements, where there is a distinct grouping of points or peak in the curve at roughness indexes from 40 to 120. This suggests that the natural sand subgrades represent an uncontrolled condition insofar as compactness of the supporting subgrade is concerned, producing a much wider range of riding quality. Class 2 pavements, it may be recalled, are designed and built with sand subbases and granular base courses providing an opportunity for greater uniformity in texture and for improved compaction which has been practiced particularly in more recent years. The result, in terms of riding quality, is naturally enough much better and much more consistent, as should be expected under reasonably good construction control. As a matter of general observation and construction experience, road designers in the Michigan State Highway Department have come to the conclusion that it would be desirable to undercut the pavement structure itself and mix and compact the natural subgrade materials for a depth of some 18 inches in order to eliminate some of the variations due to nonuniformity in soil textures and soil densities. The data in Fig. 15 present positive evidence in support of such a construction procedure and it is being adopted by the Department.

In passing, it should be noted that there is a fairly uniform distribution in terms of years of service for the Class 1 pavements in Fig. 15; while, in the Class 2 pavements, the great majority of projects are concentrated in the first 15 years. This relates to the fact that the Department for the past 15 or 20 years has been correcting deficient subgrades to provide year-round service by the use of sand subbases and improved drainage. Prior to that time, there were very few Class 2 pavements built and they did not come up to present



ROUGHNESS VS. YEARS IN SERVICE
RIGID PAVEMENT
BASED ON 1958 SURVEY

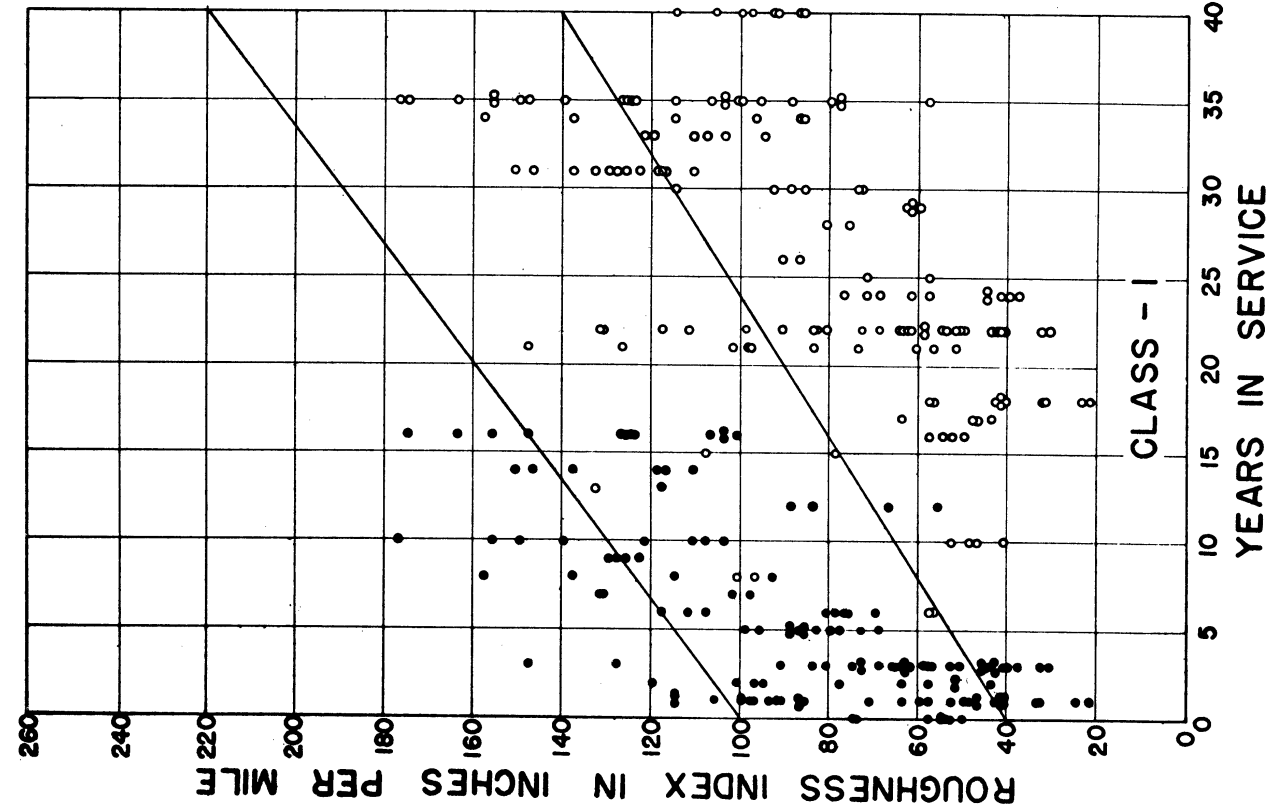
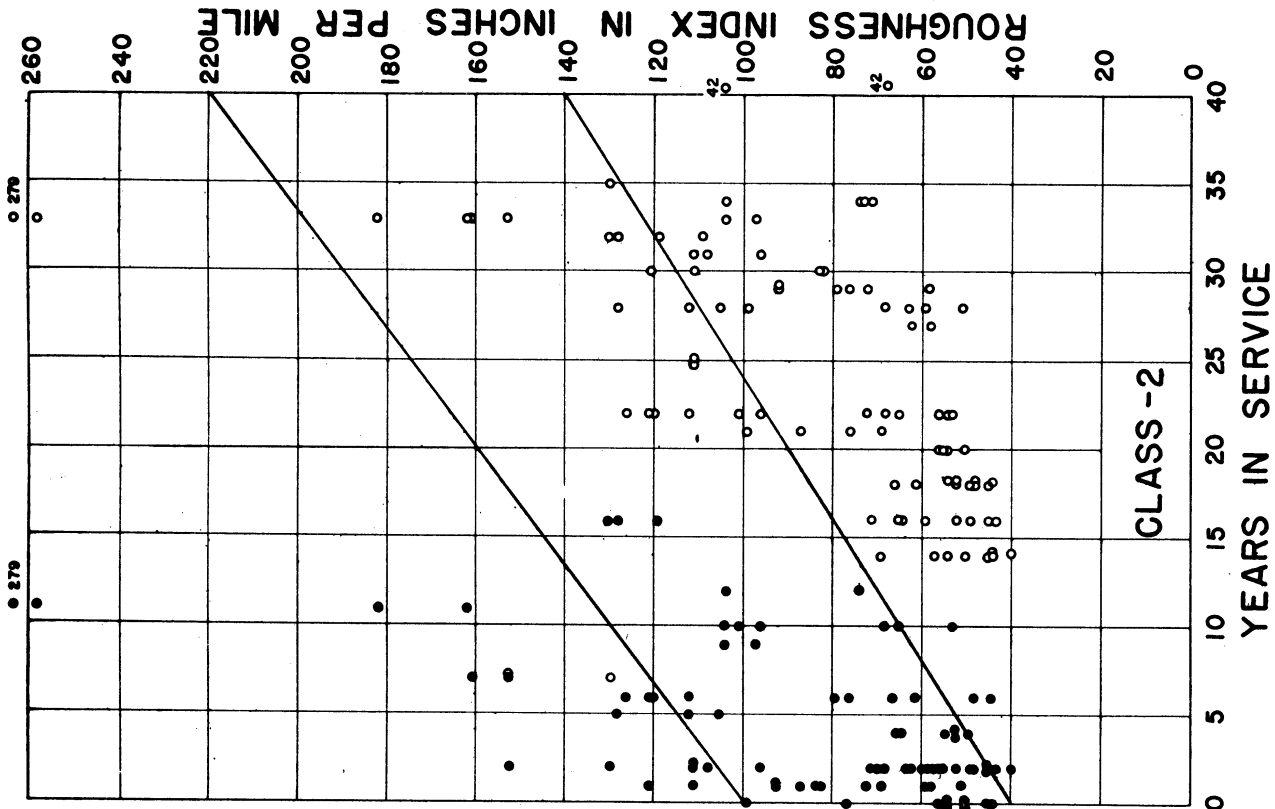
FIGURE - 15

day standards, which accounts for their poor service behavior. The group of projects with service periods from 15 to 20 years represent wartime construction under emergency conditions where steel reinforcement was not being used and there was a general relaxation of the more rigid construction specifications which has clearly left its mark in terms of poor service behavior.

To further emphasize the direct correlation between performance and design and construction conditions, several projects on the charts in Fig. 15 have been identified by contract. In the Class 1 pavements, two such contracts on US-31 provide a significant comparison. One of them, after 33 years of service, shows excellent performance falling at the lower limit of the band of normal service, with riding quality which is still rated as fair to acceptable. The other contract, after 34 years of service, is extremely rough and well above what has been suggested as normal behavior. These two pavement sections are within a few miles of each other and have practically the same traffic count. They were classified as Class 1 as the subgrade was a Plainfield sand which is one of the superior subgrades with high internal stability and excellent drainage conditions. This classification is correct for the project showing the superior performance, but is incorrect for the project showing poor performance. The latter is at a transition in soil types and the major portion of the section is over a silty clay loam with inferior drainage conditions and actually should be rated as Class 3 or Class 4.

Among the Class 2 pavements, three contracts have been selected for special comment. US-24A with roughness indexes which fall off the chart is the pavement of 20 foot, unreinforced concrete slabs without load transfer at the joints that has already been commented on in connection with Fig. 11. The other two projects, built in 1958, are M-20 and US-23. M-20 has an initial roughness index below 40, which would be rated as exceptionally smooth, and is the result of special care in placing and finishing the concrete slab. It too has already been commented on in connection with Fig. 12, and represents riding quality which can be produced under construction conditions when the effort is made to do so. The other section, US-23, represents a poorly built job with roughness index which would be rated only fair, and represents poor performance on the part of the contractor. When it is considered that riding quality is the basic commodity which is being purchased when a pavement is built, it seems fairly obvious that a large proportion of the investment has been lost by poor construction practice. "Built-in roughness" of this character is something that has not been given the attention it deserves, and one immediate benefit of the statewide recording of profiles has been to call attention to this deficiency in pavement construction practice and to provide a quantitative basis for correcting it.

In Fig. 16 are shown two similar charts comparing roughness index with years in service for older concrete pavements that have been recapped with a bituminous surface. The bands that have tentatively been used to represent normal behavior of structurally adequate pavements are shown on these charts largely for a basis of comparison. Roughness indexes for the pavement sections in question have been plotted against the years in service of the original concrete pavement and also in terms of years of service of the resurfacing. In this discussion, comment will be limited to the groups of points falling above and below the normal service band as they may provide the clue to the more important design conditions which are involved in service behavior.



EXCEPTIONALLY SMOOTH	VERY GOOD	GOOD	FAIR	ACCEPTABLE	POOR	VERY POOR	EXTREMELY ROUGH
----------------------	-----------	------	------	------------	------	-----------	-----------------

○ FROM YEAR BUILT
● FROM YEAR RESURFACED

ROUGHNESS VS. YEARS IN SERVICE
RECAPPED PAVEMENT
BASED ON 1958 SURVEY

FIGURE -16

In both the Class 1 and Class 2 recapped pavements, those points which fall well above the norm represent recapped pavements which have become just as rough as the original pavements in a relatively short period of service. Several of these projects have been resurfaced more than once and their rapid loss of riding quality demonstrates that they are structurally inadequate for one reason or another, unless they were built just as rough as the original pavement which is not likely judging by experience with construction practice in recapping old pavements. To determine the source of this inadequacy may require a field investigation and a determination of the soil conditions and environment of each of these pavements and this has not been done as yet. However, the accuracy with which the characteristics of the pavement profile, expressed in terms of the roughness index, have identified the source of difficulty in other pavements showing abnormal service behavior indicates that the weakness of the pavements under discussion can be reliably determined.

The other group of points in Fig. 16 which are of particular interest are those that fall well below the band of normal service. When the roughness index on these projects is plotted in terms of the service period of the original pavement, it must be granted that the low roughness index of the resurfaced pavement is not truly representative of the service behavior of the original pavement. On the other hand, in this group of points there are a number of projects in which the resurfaced pavement has retained its superior riding quality over a number of years in service, indicating exceptionally good performance which must be evidence of superior design or unusually favorable soil conditions and environment.

CONCLUSION

The preceding examples of pavement performance derived from studies of the pavement profile have been presented to demonstrate that service behavior of existing pavements is a most promising source of design information and a reliable basis for isolating the important factors in building adequate pavements. Analysis of the large volume of data gathered in the pavement performance surveys has just begun and there is much yet to be learned. In such a state of affairs it is not appropriate to draw conclusions with any thought that they constitute final answers. However, there is enough evidence available and enough analysis has been made to indicate the value of this approach to pavement design. While there may be much more to do in extracting the full value of these data, pertinent evidence already points to the fact that pavements of all classifications, not excepting those judged most adequate, become progressively rougher with age, confirming the implications of the one year's study of frost displacement. This brings realization that pavement performance cannot be measured in terms of static equilibrium of a beam resting on an elastic foundation subjected to static loads with strength controlled by a direct proportionality between load, deflection, and stress.

On the contrary, an objective viewpoint sees the pavement slab expanding and contracting with changes in temperature; curling and warping with temperature differentials between the top and bottom; growing and shrinking with moisture changes; and, distorted by frost displacement, only partially relieved by thawing of the frozen substructure. All of these effects superimposed on stresses due to load make the life of a pavement an everchanging cycle of

dynamic effects which seems to require a new and more realistic concept of pavement performance. This may pose another set of questions. How does an adequate pavement react to these changing conditions? How long does it retain an acceptable riding quality? What is normal behavior, in terms of which abnormal behavior can be identified and defined? Only when these questions have been answered can the responsible factors which control pavement performance be isolated and logical relations between cause and effect be determined.

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MICHIGAN PAVEMENT PERFORMANCE STUDY

BY

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PROJECT OF THE
MICHIGAN HIGHWAY PLANNING SURVEY
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IN COOPERATION WITH
U.S. DEPARTMENT OF COMMERCE
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CUMULATIVE CHANGES IN RIGID PAVEMENTS
WITH AGE IN SERVICE

By

William S. Housel*

INTRODUCTION

While there has been no attempt in the preparation of this paper to make a documented study of the history of highway pavements, it seems to be only stating the obvious to say that from the very beginning pavements have been built simply to provide durable surfaces with improved riding qualities for the safety, comfort, and convenience of the highway user. With the advent of automotive vehicles with constantly increasing speed of travel, smoothness of the pavement surface or riding quality has become increasingly important in pavement design and construction.

Also from the very beginning, pavement builders must have gauged the success of their endeavors by experience and direct observation of their pavements under the service conditions to which they were subjected. Thus, pavement condition surveys, while they may not have been formalized as they are today by systematic procedures, are as old as the oldest pavements. In the development of modern highway systems, the importance of permanence of riding quality or durability has focused increased attention on the strength of the pavement structure and its ability to maintain structural continuity under the increased loads and mounting volume of modern traffic, in

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combination with the stresses and strains associated with environmental conditions.

The increased use of rigid concrete pavements to provide high quality surfaces has paralleled the rapid development of automotive transportation. The physical characteristics of such rigid pavements have led highway engineers naturally and logically to judge their performance by the rate at which they become rough and lose their riding quality and the rate at which they crack and lose their structural continuity. The point of these introductory remarks is simply to emphasize that the changing condition of rigid pavements, as reflected in cracking and roughness, has always been a natural and realistic measure of pavement performance. The major contribution of recent years has been the introduction of refinements in procedures for making pavement condition surveys and development of more precise criteria for evaluating pavement performance.

Recognition of this approach must have been in the minds of the Highway Research Board Committee on Rigid Pavement Design when they formulated, in January, 1959, a proposed high priority research project entitled "Investigation of Existing Pavements". At that time they listed a number of significant changes in rigid pavement design in recent years and made the following pertinent statement:

"It is believed to be highly important to determine the effects of these changes in order to avoid the possibility of constructing many miles of pavement which might otherwise fail prematurely. 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."

Well-organized pavement condition surveys in Michigan date back to the middle twenties, when the late V. R. Burton organized a group of research workers who started a series of state-wide pavement condition surveys including comprehensive data on soil conditions and climatic environment.^{1,2,3} This work has been carried on over the years by a number of individuals whose names have become well known among highway engineers and soil scientists, including Kellogg, Benkelman, Stokstad, and Olmstead.

These investigators early found significant correlation between pavement performance and environment, including soil type, drainage, and climatic factors, a viewpoint which has continued to exert a dominant influence on pavement design in Michigan. Improvements in this approach to pavement design have led to more accurate evaluation of soil conditions, drainage, and climatic environment; the utilization of local soil materials of favorable characteristics; and, the selection of 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 the integrated results of these uncontrolled variables could be measured quantitatively by more accurate field surveys and objective analysis of the results. Furthermore, it was felt that pavement performance, in terms of changes in the pavement profile and cracking pattern, could be expressed numerically by a roughness index and a continuity ratio. In this respect, pavement performance studies have followed the observation

made by Lord Kelvin that, "When you can measure that of which you speak, and express it in numbers, you know something about it."

The first attempt in Michigan to measure pavement performance quantitatively in terms of pavement roughness and structural continuity was a cooperative investigation, initiated in 1952, involving the Michigan State Highway Department, the University of Michigan, and the Wire Reinforcement Institute. The data obtained from that investigation, carried on over a period of five years, indicated that pavements with steel reinforcement were measurably smoother and that cracking was measurably less than in the unreinforced concrete pavements involved in that investigation. Of perhaps more importance to the present discussion was the fact that pavement performance was evaluated in terms of a continuity ratio related to the cracking pattern and a roughness index based on measured vertical displacement in the pavement profile.

MICHIGAN PAVEMENT PERFORMANCE STUDY

In further pursuit of these objectives, a cooperative investigation was next undertaken in 1957 by the University of Michigan, the Michigan State Highway Department, and sponsors representing the trucking industry.* Pavement performance studies under this sponsorship continued for approximately two years, and in July, 1959, were taken over by the Michigan State Highway Department as part of the Michigan Highway Planning Survey - Work Program financed by HPS funds, under the supervision of the Bureau of

* The Michigan Trucking Association, the American Trucking Associations, Inc., and the Automobile Manufacturers Association

Public Roads. This program has continued to date under a contract with the University of Michigan. In the first two years, equipment for recording pavement profiles was developed and tentative procedures were established for evaluating the data obtained.

A truck-mounted profilometer for accurately recording pavement profiles was built, following closely similar equipment used for some time by the California State Highway Department. Electronic recording equipment and integrators were added to provide a chart-recorded profile of the pavement in both wheel paths and to compile the cumulative vertical displacement in inches per mile. Means were also provided to record pavement cracks and joints. Early results from these studies were presented and discussed in a paper presented to the Highway Research Board in January, 1959.⁴ In passing, it is interesting to note that there is nothing new about profilometers and measuring roughness as the sum of vertical displacements per mile as a roughness index. In his paper to the Highway Research Board in January, 1960, Francis Hveem presented a most interesting review of this subject and described a number of such devices, the earliest one in available records dating back to before 1900.⁵

Pavement Profiles for 1958 - 1960

During the three years 1958, 1959, and 1960, close to 6,000 lane miles of pavement profile were accumulated. The analysis of these data has proceeded concurrently, insofar as personnel and facilities would permit; it is the purpose of this paper to present some of the significant results presently available. As indicated by the title of this paper,

attention is directed to cumulative change in rigid pavement profiles with age in service; it is felt that the data do reveal significant trends in pavement performance and direct relationship to controlling design conditions. In presenting these data, it must be recognized that there is a tremendous volume of information involved in some 6,000 miles of pavement profile; the present discussion will be limited to several classifications of rigid pavement which have been sampled in sufficient quantity to provide a reasonable basis for analysis. All pavements included in the profile survey are part of the Michigan state trunkline system of some 9,435 miles, including 8,050 miles of two lane pavement, 135 miles of three lane pavement, and 1,250 miles of divided four lane pavement. The trunkline system thus amounts to 21,500 lane miles of pavement; thus, the three years of profile surveys discussed in this report provide a sample of approximately 27.5 per cent of the total trunkline mileage.

Most of the data obtained are for pavements rated as Class 1 and Class 2, although there will be some data presented from surveys of Class 3 and Class 4 pavements. In this connection, it is necessary to define these four pavement classes as they have been incorporated in the Michigan trunkline surveys. The first pavement evaluation of the Michigan trunkline system was presented as of January 1, 1958, and was compiled from the Michigan State Highway Department's records, including design data, pavement condition surveys, maintenance records, and soil surveys. At that time, 55 per cent of the state trunkline system was rated as Class 1 and Class 2 roads, adequate for legal axle loads at all seasons of the year;

45 per cent was rated as Class 3 and Class 4, inadequate and requiring Spring load restrictions. In this road classification for legal axle loads, the four levels of adequacy selected are 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 data selected from 6,000 miles of pavement profile, for the present discussion, are shown on a series of charts developed as a standard format after considerable "cut-and-try" experimentation. In these charts, as shown in Fig. 1, the roughness index in inches of vertical displacement per mile is plotted as the horizontal abscissa and the years in service as the vertical ordinate. The data presented in Fig. 1 are for the traffic lane of Class 1 rigid pavements and represent pavement profiles of some 556 miles of pavement. At the top of the chart is shown a tentative roughness

rating that has been in use for several years.⁴ Each plotted point represents an individual pavement profile in the outer or inner wheel path of a specific construction contract. There are 123 such contracts and 556 miles of pavement included in Fig. 1: 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. In general, it has been found that the outer wheel path is rougher than the inner wheel path; and, while special studies have been made of this variation, these studies will not be discussed in detail as part of this paper.

Significant Variations in Pavement Profiles

In spite of the wide scattering of roughness index values in Fig. 1, there are a number of characteristics of these data, the analysis of which indicates significant trends in pavement performance which may be enumerated as follows:

1. There is a general increase in roughness with age which will be discussed as evidence of cumulative changes in rigid pavements with age in service.
2. There are a number of specific projects which exhibit roughness indices much less than the general trend and others with values much greater, both of which may be related to controlling design or construction conditions responsible for this abnormal behavior.
3. There is a discontinuity in average roughness indices at a service period of approximately 25 years which may be related to construction changes in the trunkline system and indirectly influenced by Michigan design procedure.

Cumulative Changes in Pavement Profiles

Evaluation of cumulative changes in rigid pavements with age in service is the major objective of this paper. The method of evaluation proposed after considerable study is to establish a band of normal behavior as shown in Fig. 1. 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 a triangle on the chart, through which the central line of the normal behavior band is drawn. For the first 25 years shown in Fig. 1, the average results fall consistently along a line with an intercept of 65 inches per mile on the horizontal axis and a slope of 4.5 inches per year as the average increase in the roughness index. After 25 years there is a discontinuity or displacement in this average line which will be discussed later.

The width of this band of normal behavior has also been established by trial and error as parallel lines which bracket 85 per cent or more of the observations and balance the excluded observations, indicating abnormal performance, on either side of the band. For example, in Fig. 1, for observations in a period of less than 25 years, approximately 7.5 per cent of the pavement profiles have roughness indices less than normal and the same approximate percentage, greater than normal. It may be observed that projects falling outside this band represent abnormal behavior, which may provide the most informative data available to relate pavement performance to design and construction conditions. Examples will be cited in a further discussion of abnormal behavior.

The important deduction from these data at this point in the discussion is that rigid pavements represented by this sample suffer a continuing or cumulative increase in roughness at an average rate of about 4 to 5 inches per mile per year. The fact that these pavements have been rated as Class 1 pavements implies that they are adequate or more than adequate for legal axle loads at all seasons of the year; thus, load carrying capacity is not a controlling factor in this progressive loss of riding quality. From this it follows that environmental factors, mainly associated with seasonal cycles, dominate this type of pavement deterioration, and there is other evidence to support this conclusion.

Older Pavements Showing Abnormal Behavior

There are a number of projects in Fig. 1 with roughness indices substantially less or greater than those within the band selected to represent normal behavior. Attention will be directed first to pavements more than 25 years old. In the first place, as previously mentioned, there is the definite discontinuity in the five-year averages, influenced by a considerable grouping of projects with roughness indices much less than those within the band of normal behavior deduced from pavements less than 25 years old.

The first and perhaps major factor in this shift arises from the fact that many of the older pavements built before 1936 have been retired from service by reconstruction, recapping, or a change in classification. With a few exceptions which may be encountered, only those projects exhibiting superior performance are still in service and have been picked up

in the profile surveys. A review is in progress to trace the history of all concrete pavements built before 1936, but complete results are not available for this report. Consequently, only a few examples can be cited at this time to illustrate this point and to indicate the close correlation between design and construction conditions and unusual pavement performance.

One such project, US-31 (3-C1), 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. Another project, US-31 (5-C1), after 34 years of service, is rated extremely rough, with a roughness index falling near the upper limit of the band of normal behavior. Both were rated as Class 1 pavements on the basis of an area soil survey identifying the soil type 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 classification of Plainfield sand is correct for the project showing superior performance, but incorrect for the second project, which has become extremely rough. In the latter case, the project is located at a transition in soil types, the major portion being on a 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 types and marked changes in pavement performance are accurately identified on the pavement profile.

Another revealing example that may be cited is a 33 year old project, US-31 (5-C2), which appears to have shown much better than

average performance. A review of the pavement profile shows that this contract covers an area of well-drained sands of the Plainfield or closely related series, but with several smaller areas of low-lying poorly drained soils. These areas became extremely rough after some 30 years of service, but were recapped with a bituminous surface in 1956. The balance of the pavement had become quite rough, with considerable cracking shown by a reduction in the continuity ratio from 6.66 to 1.35. The roughness indices for this pavement, which is still in service, were reported in Fig. 1 as the average for the entire project. Segregation and reclassification of the sections which became very rough and have been resurfaced and correction of the roughness index for the balance of the pavement would bring it more nearly within the band of normal behavior, with slightly better than average performance.

Another group of points showing better than normal performance are identified as Contract M-25 (37-C2). This pavement is on a shoreline road along Lake Huron built along a beach ridge on soil identified as the Eastport series, another high quality subgrade. This is the clue to its superior performance, but it has nevertheless increased in roughness, although at a reduced rate, as residual displacement resulting from seasonal frost action, details of which will be discussed later.

Another example of interest, shown in Fig. 1, is Contract US-131 (5-C2), the only one of the projects in the survey more than 25 years old which is still in service, even though its roughness index is much greater than those within the band of normal behavior. Roughness indices of 248 in the inner wheel path and 307 in the outer wheel path show that it has

become extremely rough and is actually beyond the limits of the tentative rating scale. This project is a plain concrete pavement built in 1929 just south of the city limits of Cadillac. In addition to being rough, it is badly cracked with its continuity ratio having been reduced from 6.67 to 0.85. In terms of average slab length, this is a change from 100 feet between joints to an average of about 12 feet between cracks.

As a whole, this old project is the product of outmoded design standards that no longer represent Michigan practice. The rougher sections are over inferior type soils that would now be compensated for by raises in grade and a free-draining granular subbase. A substantial part of this project was in a relocation over generally good subgrade soils, but was built without adequate control of subgrade compaction and without the advantage of stage construction to condition the subgrade before paving. This project is on a major route and would doubtless have been rebuilt some time ago except that its relocation, long planned, was postponed until the advent of the major improvement program of the past several years. This relocation is now in progress and only this unusual set of circumstances found it still in service when the profile survey of this route was made in 1959.

Younger Pavements Showing Abnormal Behavior

Pavements less than 25 years old shown in Fig. 1 also include about an equal number of projects with roughness indices substantially above and below the limits of normal behavior. In the earlier years, the conclusion is quite inescapable that the wide range in riding quality must

have been produced during construction and is thus initial or built-in roughness.

Looking first at the group of eight projects less than 15 years old with superior riding quality, there are three projects, US-31 (13-C1), US-12 (17-C1), and US-12 (18-C1), surveyed the year they were built or one year later, with roughness indices of 50 or less, which would be rated as exceptionally smooth. There are five additional projects, US-23 (49-C3), US-27 (4-C2), US-27 (7-C5), US-31 (41-C2), and US-31 (7-C9), which, allowing for normal increase in roughness, must have been built with an initial roughness in the same range and rated as exceptionally smooth. Five of these eight projects were built by two contractors who have gained special recognition for high quality workmanship. The same may be said for the other projects giving evidence of good workmanship, even though the illustration lacks the emphasis of repetitive coincidence of contractor and excellent performance.

Attention is next directed to a group of five projects less than 15 years old with roughness index values greater than those within the band of normal behavior. Again it may be assumed that normal increase in roughness would leave built-in roughness as the major source of decreased riding quality. Project US-12 (16-C2) was built in 1960 and surveyed in December of the same year. The low temperature may have produced some curling; but, other than this, there are no known job conditions contributing to the increased roughness other than construction methods.

Project US-112 (43-C3) presents a particularly interesting comparison in that its westbound lane has a roughness index of 167 in the

outer wheel path and 161 in the inner wheel path, rated as "poor". The eastbound lane, on the other hand, had a roughness index of 102 in the outer wheel path and 92 in the inner wheel path, which, while not outstanding, would at least be rated as "good". A report from the project engineer on this contract reveals that there were special job conditions which account for the abnormal result. The westbound lane was paved late in the year to provide for traffic during the winter, until the project was completed, and a considerable portion of it was hand finished. The eastbound lane was completed the next spring and was machine finished. Incidentally, the paving was done by the contractor who paved three of the eight projects cited above as evidence of high quality workmanship. The pavement was a short stretch at the intersection of two major trunklines where a future grade separation was planned. Actually, it could be considered in the category of a temporary roadway for a limited service of a few years; this may have been a contributing factor in its construction.

The other three projects rougher than normal, US-23 (18-C8), US-27 (29-C3), and US-23 (4-C7), are all by different contractors. There are no special job conditions presently known to affect these results other than construction methods as the common denominator.

Comparison of 1959 and 1960 Roughness Indices

While it is not considered a significant variation in pavement performance, there is a rather definite shift in Fig. 1 in two groups of observations, for projects built in 1959 and 1960, which does require some explanation. It was first thought that this might be evidence of a change

in built-in roughness, reflecting an accelerated construction program and, possibly, less effective construction control due to overload of inspection facilities and personnel. However, a more searching analysis of these observations points to the conclusion that this shift in roughness observations has been produced by a combination of factors, none of which can be demonstrated to be primarily responsible.

It may first be noted that the 1959 projects have an average roughness index of approximately 60, somewhat less than normal, while the 1960 projects have an average roughness index of 75, somewhat above normal. Practically all of these two groups of projects were surveyed in 1960, with the 1959 projects being surveyed in the spring and early summer and the 1960 projects in the fall, after September 1 and some as late as December 15. After September 15, Michigan specifications have required 1 inch expansion joints every 400 feet; this may have had some influence on the results.

One of the factors which may have affected some of these observations is the temperature differential from June to December, with a similar but reduced differential between the top and bottom of the slab producing curling at the joints. In May and June, 1960, when a number of the 1959 projects were surveyed, air temperatures ranged between 60 and 80 degrees. A number of the 1960 projects were surveyed in December, 1960, when the air temperature ranged from 20 to 30 degrees, mostly in the low twenties. Unfortunately, there were no parallel surveys on the same projects in June and December which would have provided a direct comparison between the two groups of observations under discussion.

However, examples of curling due to temperature differentials are available from special studies where abnormal increases in roughness were

reported on two recently constructed pavements of similar design. These examples are shown in Fig. 2. On the bottom of the chart are shown pavement profiles on a short section of pavement on I-75, near Flint, where curling at the joints is the major source of the increase in roughness. This increase in roughness may include the residual deformations from two winters of cyclic change as well as some frost displacement in the subbase, which reaches a maximum about the time of the second survey in the middle of March.

A similar example is shown on the top of the chart, with the difference that the timing of the two surveys is reversed. The profile in March, 1960, shows maximum roughness as the combined effect of curling and frost action. The second profile, in May, 1960, shows the recovery of the pavement from the maximum temporary displacements of the seasonal cycle. Temperature differentials between the two surveys are comparable to those under discussion.

Aside from the temperature effects that may be involved, the possibility of experimental error and some effect of the accelerated construction program cannot be completely dismissed. With respect to the latter, there is no further comment except to make the obvious observation that inspection control is an ever present problem with results in some proportion to the attention that can be devoted to it.

With reference to experimental error in the rather complex instrumentation involved in recording and integrating pavement roughness, there are always problems to be met, particularly under the requirements for mass production of pavement profiles. There were such problems during

the summer of 1960, and, as a matter of fact, the profilometer was out of service for several months for a general overhaul and recalibration. Some changes in electrical circuits and mechanical details were made to improve operating characteristics and to facilitate frequent calibration. A review of the frequent calibrations during this period indicates that the data under discussion could have been affected in terms of average roughness by as much as 5 per cent, or a maximum of 10 per cent in individual cases. This is considered to be within the normal operating accuracy of the profilometer; it is the grouping of the two sets of observations at the two time periods involved that makes such equipment error a possible contributing factor.

Passing Lane of Class 1 Rigid Pavements

There are a number of other examples of trends in pavement performance that may be selected from the 6,000 miles of pavement profile covered by the three-year survey. In Fig. 3 are shown the roughness indices from some 290 miles of the passing lane of dual lane divided highways. Practically all of this mileage is less than five years old, having been constructed as part of the Interstate system. The same band of normal pavement performance used in Fig. 1 has again been used as a basis of comparison. In this case, it is obvious that the volume of data and the short period of service is not sufficient to establish any basis to differentiate between the traffic lane and passing lane. At the same time, it may be noted that Class 1 pavements are not expected to show any significant effect of wheel load applications, being rated as adequate or more than

adequate for legal axle loads at all times. While the available data cannot be considered conclusive, more than 90 per cent of the observations in Fig. 3 do fall within the band of normal behavior; the cumulative change in roughness of the limited number of older projects also follows the same trend, within the indicated limits.

For a direct comparison, two specific projects have been noted, M-21 (35-C9) and M-21 (35-C10), each with a service period of 19 years. These same projects have also been identified on Fig. 1 to show that the roughness indices of the traffic lane and passing lane fall in the same range, within very narrow limits. There is one exception to this statement: 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 investigation of this section revealed that a storm sewer had been laid along the edge of the pavement, and backfill settlement undoubtedly produced the high roughness index.

It is indicated again in Fig. 3 that there is a shift in roughness indices in the passing lane of 1959 and 1960 projects that has already been commented upon in connection with Fig. 1. The conditions under which these projects were surveyed and the probable contributing factors are identical with those in the traffic lane. The fact that the results are the same needs no further comment. Finally, any conclusion that could be drawn from the comparison between the traffic lane and passing lane of the Class 1 pavements would be that the available data indicate no measurable difference due to wheel load applications.

Class 2 Rigid Pavements

The next two charts, Fig. 4 and Fig. 5, show the roughness indices on Class 2 rigid pavements. It may be recalled that these pavements have been designed to compensate for seasonal loss in strength due to subgrade deficiencies and less favorable environmental conditions. From the standpoint of load carrying capacity, they are considered adequate for legal axle loads at all seasons of the year. Referring first to Fig. 4, for the traffic lane, the data shown cover 244 construction contracts and some 1275 lane miles of pavement. The same band representing normal behavior has been shown in Fig. 4, as there is no evidence to support any change in these limits. Approximately 7 per cent of the data show roughness indices less than normal, and 8 per cent have roughness indices above normal limits.

As a whole, the data in Fig. 4 are quite comparable to those in Fig. 1, and do show that Class 2 pavements follow the same trends in behavior exhibited by the Class 1 pavements. The cumulative change in pavement roughness with years in service shows observation points fairly well balanced around the average line or norm. The major difference in Class 2 pavements has to do with the change in Michigan design standards over the years and relates to the fact that for the last 15 or 20 years all construction on the state trunkline system has been designed and built to be adequate for legal axle loads at all times of the year. Thus, there is only a scattering of Class 2 pavements that have been in service for periods of more than 15 or 20 years. The bulk of the data in Fig. 4 are consequently

concentrated in the first 15 years, with a great preponderance of the mileage having been constructed as part of the major construction programs of the past four or five years. Many of the comments made with respect to performance of the Class 1 pavements also apply to the Class 2 pavements; hence, many of the details previously discussed need not be repeated.

Attention may then be directed to differences in performance of the different classes of pavement as revealed by pavement profile data. The first such difference in Fig. 4 is the fact that Class 2 pavements built in the first two or three years show no evidence of any shift in built-in roughness commented upon in discussing the Class 1 pavements, including possible changes in construction control or inspection procedure. Inasmuch as the construction of the Class 2 pavements is coincident with the construction of Class 1 pavements, it may then be deduced that construction control was not a factor in the shift of roughness indices for Class 1 pavements. There are some significant data in Fig. 4 with respect to built-in roughness, however, these being related to several special projects that have been identified. By 1959, the emphasis on pavement profiles and roughness data had been given sufficient publicity within the state, which, coupled with some competitive endeavor related to different types of pavement, stimulated contractors on certain contracts to make a special effort to build smooth pavements. Project M-20 (101-C2) is a reinforced concrete pavement on the Bay City - Midland expressway where special efforts were made to produce superior riding quality. The fact that the average roughness index on all four lanes of this project was less than 40 is an indication of what can be done with respect to built-in roughness when sufficient

effort is applied. Another project, I-75 (75-C1), built in 1958 and surveyed the same year, shown in Fig. 4 represents the top of the range in initial roughness. Construction reports from the project revealed that poor subbase compaction left the paving contractor with inadequate support for his forms. He reported this difficulty at the time of construction as affecting the quality of his work.

While all of the projects in Fig. 4 which showed abnormal performance are being investigated, only one other example has been identified for discussion in this report, that being Project US-24A (30-C3 and C4). This project stood out from the standpoint of poor riding quality in the earliest days of the pavement performance study, so it received immediate attention and has, as a matter of fact, been commented on in previous reports.⁴ This contract shows the result of using short, 20 foot slabs of plain concrete without load transfer at the joints in an effort to control pavement 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; then, an 18 to 24 inch sand subbase was added to provide more adequate subgrade support and to eliminate pumping action. The crack control was successful in that very few of the 20 foot slabs have cracked, but this design produced one of the roughest riding pavements in southern Michigan due to tilting of the slabs and faulting at the joints.

Passing Lane of Class 2 Rigid Pavements

In Fig. 5, roughness indices for the passing lane of Class 2 rigid pavements have been assembled. These data cover 139 construction contracts and 277 lane miles of pavement. There are only a few projects in the survey more than 10 years old, for reasons already cited, and a high proportion of the roughness data are from projects built in the last 5 years. More than 90 per cent of the data fall within the same band of normal behavior, with 2 per cent of the points indicating roughness indices less than normal, and 6 per cent greater than normal. There is no evidence from these limited data of any differential in the cumulative change in roughness between the passing lane and traffic lane of Class 2 pavements.

There are fewer projects showing evidence of abnormal behavior outside the band of normal performance and these are being investigated for special conditions which may have produced these results. With respect to built-in roughness, there are several projects that are exceptionally smooth and several that are rougher than normal. These include the passing lane of projects already cited in connection with Fig. 4 for the traffic lane of Class 2 pavements, so no further comment is required.

Class 3 Rigid Pavements

The performance of Class 3 rigid pavements, insofar as they are covered in the three-year survey, is shown in Fig. 6. The data are from 70 construction projects and 353 lane miles of pavement. Most of the projects are more than 20 years old and represent outdated design and construction standards which do not compensate for seasonal loss of strength, so may be

regarded as deficient for legal axle loads, at least during the spring breakup.

There are a few projects less than 15 years old still rated as Class 3 pavements, a fact which appears to be inconsistent with present Michigan design standards. Further investigation of the two projects identified in Fig. 6 determined that they have been rebuilt to present standards and should now be reclassified, but this correction had not been made at the time of the profile survey. M-46 is an east-west trunkline across the state, running west from Saginaw, and carrying fairly heavy truck traffic generated by the petroleum industry in this area. The other two projects identified were 34 years old at the time of the survey in 1959 and had become extremely rough, requiring heavy maintenance to keep them in service. These two sections have subsequently been strengthened and resurfaced, but have not yet been resurveyed. These recent construction contracts have been betterment projects consisting of overlays to reinforce the present pavement without complete rebuilding to correct the basic deficiency in subgrade support and inferior drainage, the results of which are still considered debatable. While the pressure to bring the rest of this road up to present day standards has been heavy, the cost of rebuilding, involving complete relocation, has postponed its programming in competition with other important routes also requiring attention.

Further discussion of the performance of Class 3 pavements would point to the much wider range in performance of those projects more than 20 years old. This wide variation in behavior, as compared to what has been selected as normal performance, suggests less attention in design and

construction to those factors most responsible for pavement performance; namely, soil conditions, drainage, and environment. In this connection, it may be noted that 25 per cent of the roughness indices are less than normal and 14 per cent are greater than normal.

It is probable that further investigation of projects showing abnormal performance will lead to reclassification of a number of the projects on both sides of the band of normal performance. It is almost certain that those projects which have become extremely rough would require complete reconstruction or extensive correction of those deficiencies which have lead to poor performance to bring them up to present day standards. One project in this group, US-23 (29-C3), can be used for illustration. This project, which was 27 years old when surveyed in 1959, had roughness indices of 330 and 335 inches per mile and is built over a complex of poorly drained heavy clays and silty sands. It has carried heavy traffic in later years and has required heavy maintenance to keep it in service. It was resurfaced in 1959, shortly after the profile survey, and has now been replaced by a modern expressway, I-75 of the Interstate system. The other project identified, US-12 (37-C2), was built in 1937 and was extremely rough when surveyed in 1959 after 22 years of service under heavy traffic. It, too, was built over poorly drained soils of heavy clay and some areas of muck and swamp-border soils in low-lying areas. The concrete pavement was reinforced and of standard thickness, but lacked the granular subbase now used to compensate for subgrade deficiency and to eliminate pumping. It was resurfaced shortly after the profile survey and has now been replaced in the trunkline system by a modern expressway, I-94 of the Interstate system.

Class 4 Rigid Pavements

A few Class 4 pavements were included in the profile surveys and are shown in Fig. 7 more to complete the picture of pavement performance than to provide design data of current interest. There are only 7 contracts and 20 lane miles of pavement, all of which are more than 30 years old. All of these pavements are rated as extremely rough and fall in the upper range or above the band of normal behavior which has been shown for comparison. This performance is consistent with the Class 4 rating, indicating pavements inadequate for legal axle loads at all times. Special field investigation of these projects has not been made and may not be required as they can be identified with a history of heavy maintenance and substandard conditions.

SOURCES OF PAVEMENT ROUGHNESS

The foregoing review of cumulative changes in rigid pavement with age in service has been based almost entirely on an analysis of 6,000 miles of pavement profiles. It may be desirable to supplement these data by some discussion of special studies which may serve to emphasize several important factors contributing to pavement roughness. These factors may be briefly described as temporary and permanent roughness due to frost displacement, built-in roughness, and deflection due to wheel load applications.

Frost Displacement

In the first two years of Michigan Pavement Performance Study, a number of special pavement sections were selected throughout the state to measure the seasonal fluctuation in pavement profiles due to frost action. Some of the data have been previously reported and will be only briefly presented here.⁶ In Fig. 2 were shown several pavement profiles illustrating temporary displacement due to curling at the joints combined with frost displacement in the granular subbase. As shown by a comparison of pavement profiles at different times of the year, much of this displacement at the time of maximum frost action was temporary and disappeared in the summer profile. However, there was some residual or permanent displacement remaining after each cycle to which the pavement was subjected.

In Fig. 8 is shown a seasonal fluctuation in pavement profiles through the rather severe winter of 1959 and the succeeding summer. The frost displacement is shown for four classifications of pavement in terms of the average increase in roughness in inches per mile for all pavement sections in each class. The shaded areas represent the permanent or residual roughness contributing to the cumulative increase in roughness over a period of years. There was no consistent correlation between frost displacement and pavement classification from the standpoint of structural adequacy, although there was some trend in that respect.

This type of frost displacement is to be distinguished from the deep-seated frost-heaving which was a problem in former years, but which has now been largely eliminated from the state trunklines. Such frost displacement has been found to be associated with freezing of moisture

accumulating in the granular bases and subbases through infiltration at the shoulder or through joints and cracks, or condensation under the surface of moisture entering in the vapor phase. Thus, the highest classification of pavements, from the standpoint of load carrying capacity, were affected, as well as the less adequate roads.

From the standpoint of cumulative change, there are several specific projects shown in Fig. 9 for which profile data are available over several successive years to illustrate the rate of increase in roughness the first few years after construction. There are both Class 1 and Class 2 rigid pavements in this example, and data are shown from profiles of an experimental section of continuously reinforced concrete pavement on I-96. The rates of increase in roughness as shown are an approximate average, excluding the first winter, as the residual roughness for the initial cycle of displacement does not appear to be representative. There is also considerable variation depending on the severity of winter from the standpoint of frost action. In general, the rate of increase, varying from three to seven inches per mile per year in these several pavements in their early years of service, may be expected to level off over a considerably longer period of service.

Built-in Roughness

It would seem that there has been enough said about built-in roughness in the discussion of pavement profiles covering a considerable mileage of Class 1 and Class 2 rigid pavements previously discussed. However, there is shown in Fig. 10 one source of built-in roughness involved in pavement finishing in which the relation between cause and effect is so

specific that it needs no particular comment. The excessive roughness of bridge decks and short lengths of pavement slab where hand finishing is employed has been a problem of much concern to the Michigan State Highway Department for some time. Fig. 10 shows direct comparison between sections of pavement or bridge deck, as the case may be, where hand finishing has been employed and adjacent pavement which has been machine finished. Transverse finishing of bridge decks is one of the possibilities under investigation and it has shown some promise.

Roughness Due to Wheel Load Applications

Pavement deflection under wheel load applications and the permanent displacements caused by excessive load are a source of loss in riding quality that has always been of major concern to highway engineers. In the roughness data from some 6,000 miles of pavement profile reviewed in this report, two important facts stand out, qualified necessarily by the limitations in the volume of supporting data available. First, there were measurable increases in the cumulative roughness over a period of years in the Class 3 and Class 4 rigid pavements that were known to be deficient in load carrying capacity. This deficiency was emphasized in specific projects that were cited where their poor performance had been more definitely related to known deficiency in subgrade support.

Second, there was no measurable difference in the cumulative roughness of the traffic lane and passing lane of Class 1 and Class 2 pavements in the periods of service covered by the profile data presented. The necessary qualification in this statement is in recognition of the short period of service which is related to two developments in highway

construction in Michigan. First, the construction of dual lane divided highways, where such a comparison could be made, has largely taken place in the last 10 years. Second, it is only in the last 15 or 20 years that Michigan design standards have required that all trunkline construction be made adequate to carry legal axle loads at all times of the year.

These qualifications notwithstanding, the evidence accumulated is entirely consistent with the statement made by Commissioner Curtiss of the Bureau of Public Roads in discussing damage caused by highway loads when he said, in effect, that properly built roads would not be damaged by the loads for which they were designed. Evidence bearing on the same point is provided by pavement profile data from Class 1 and Class 2 pavements showing that the cumulative increase in roughness with years in service is closely related to soil conditions, drainage, and climatic environment. This in turn illustrates the point made by the late Commissioner MacDonald of the Bureau of Public Roads when he said: "The roads are more destroyed really by climatic and soil conditions than they are by any use that is made of them."

CONCLUSION

The Michigan Pavement Performance Study has now been in progress for four years and has accumulated some 8,100 miles of pavement profiles, including those from the 1961 surveys. This paper presents a substantial part of the data on rigid pavements for the first three years, covering

966 construction contracts and some 6,000 lane miles of pavement in all parts of the state. The analysis of these data is still in progress; it is felt that additional information of value in pavement design and construction is yet to be obtained from the data, particularly in the study of all of those projects whose performance varies considerably from normal behavior.

This study has been predicated on the belief that accurate recording of the condition of existing pavements after years of service under actual field conditions and an objective analysis of the resulting data was a most promising approach for evaluating pavement performance and relating it to pavement design and construction. In more specific terms, the procedure involved, first, accurate recording of the profile of the pavement surface which has been evaluated quantitatively in terms of a roughness index in inches of vertical displacement per mile, supplemented by a study of the magnitude and character of these displacements as revealed by the recorded pavement profile. Second, the cracking of the pavement has been recorded and evaluated quantitatively as a continuity ratio directly related to the average interval between cracks and joints.

From these data the following tentative conclusions, subject to further study and possible revision, may be presented to summarize the results.

1. Pavement performance and cumulative change in rigid pavements, under Michigan environment and service conditions, can be expressed in terms of a band of normal performance which brackets approximately 85 per cent of the data, excluding those projects which have shown abnormal performance.

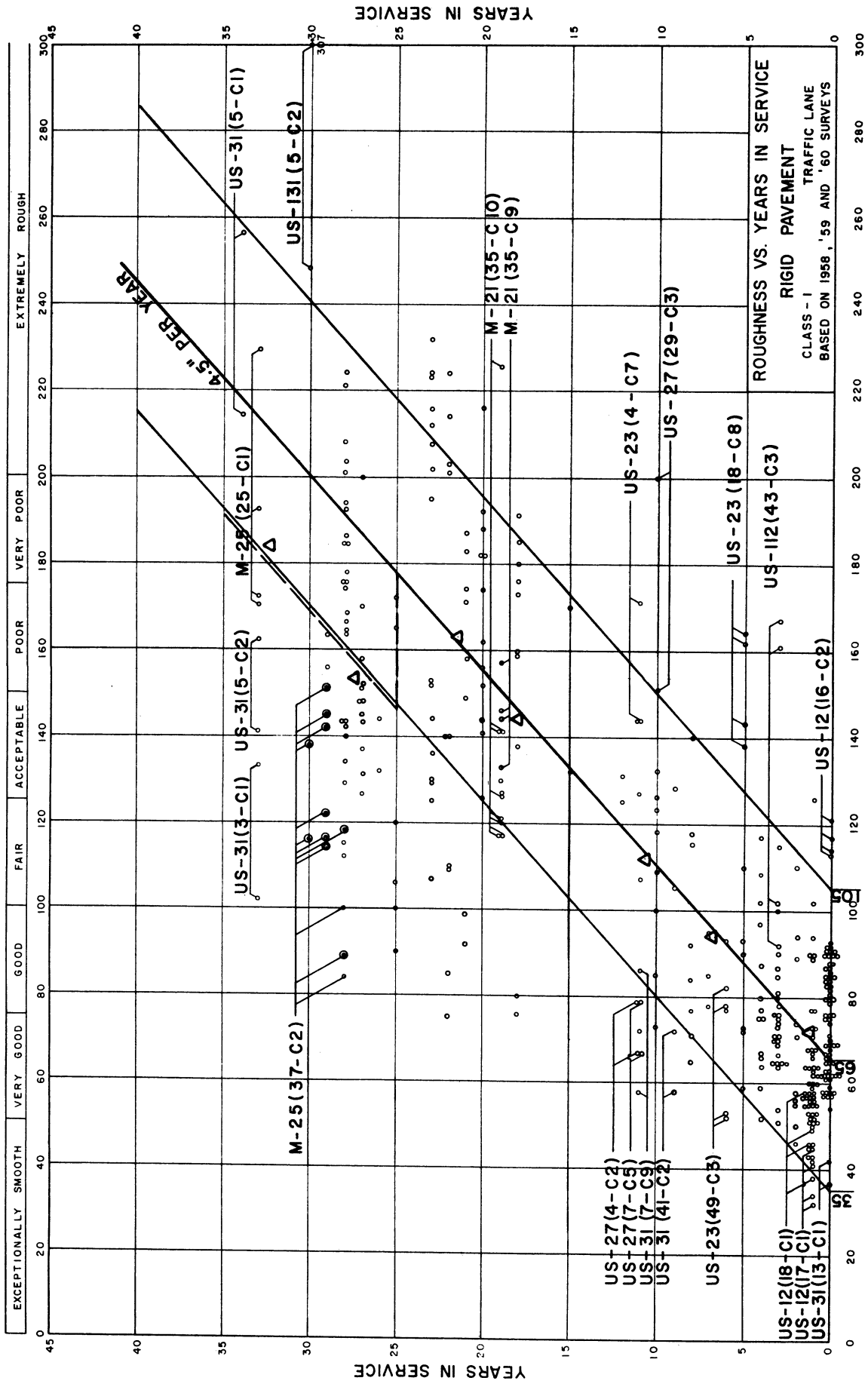
2. The width of this band of normal performance has been taken as 70 inches of vertical displacement per mile as representative of the limits within which variations in riding quality have been and perhaps can be controlled by design and construction practices.
3. Initial or built-in roughness, in terms of inches of vertical displacement per mile, ranges from 35, which is considered exceptionally smooth, to 105, rated as only fair. From a general consideration of this range and from results on specific projects which were cited, this is one factor in riding quality which appears to offer a promising opportunity for substantial improvement in the upper limit and any variation above this limit.
4. The progressive increase in pavement roughness, or cumulative change with age in service which has been characterized as normal under Michigan climatic environment, average 4.5 inches per year and has been based largely on the performance of Class 1 and Class 2 pavements designed and built to carry legal axle loads at all times of the year.
5. Comparison of the traffic lane and passing lane of Class 1 and Class 2 pavements, within the limits of available data, and analysis of specific projects showing abnormal behavior both indicate that an average cumulative increase in roughness of a magnitude of four to five inches per mile per year results from the characteristics of rigid pavements subjected to Michigan climatic conditions, rather than from load applications. This finding is confirmed by special studies of frost displacement and effect of temperature differentials and appears to identify this range as one that may not be effectively controlled by present design and construction practices.
6. The data from Class 3 and Class 4 pavements, recognized as deficient in load carrying capacity, do show evidence of an added increase in roughness due to wheel load applications. These data come from older roads, not representative of present design standards in Michigan and consequently provide a limited basis for drawing specific conclusions. However, when supplemented with more

definitive results from specific projects showing abnormal behavior and viewed in the light of favorable results from Class 1 and Class 2 pavements, there is clear indication that deficiency in subgrade support is the primary factor in their inferior performance.

7. Finally, with respect to procedure for measuring pavement performance and relating it to design and construction conditions, it is found that the projects showing abnormal behavior provide the most significant information. Analysis of all of these projects has not been completed, but examples that have been presented in this report do, in the writer's opinion, provide convincing evidence that the investigation of existing pavements under actual service conditions and environment fulfills the promise it was felt to hold for a realistic appraisal of pavement design and performance.

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ROUGHNESS VS. YEARS IN SERVICE
 CLASS - I
 RIGID PAVEMENT
 TRAFFIC LANE
 BASED ON 1956, '59 AND '60 SURVEYS

ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

FIGURE - I

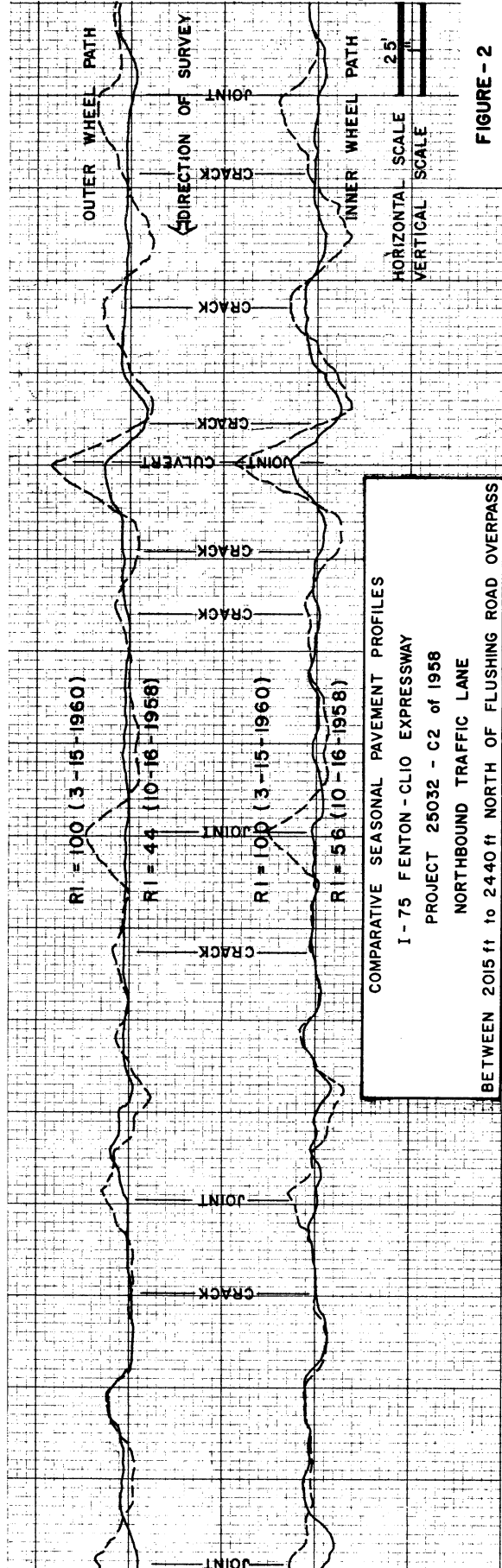
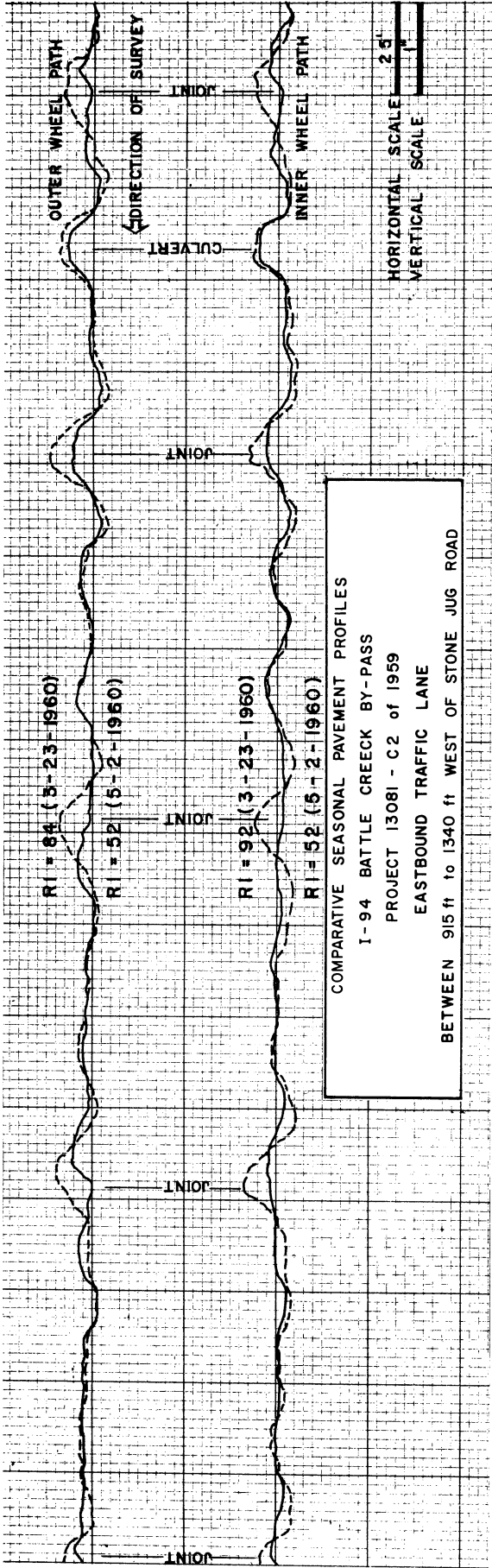
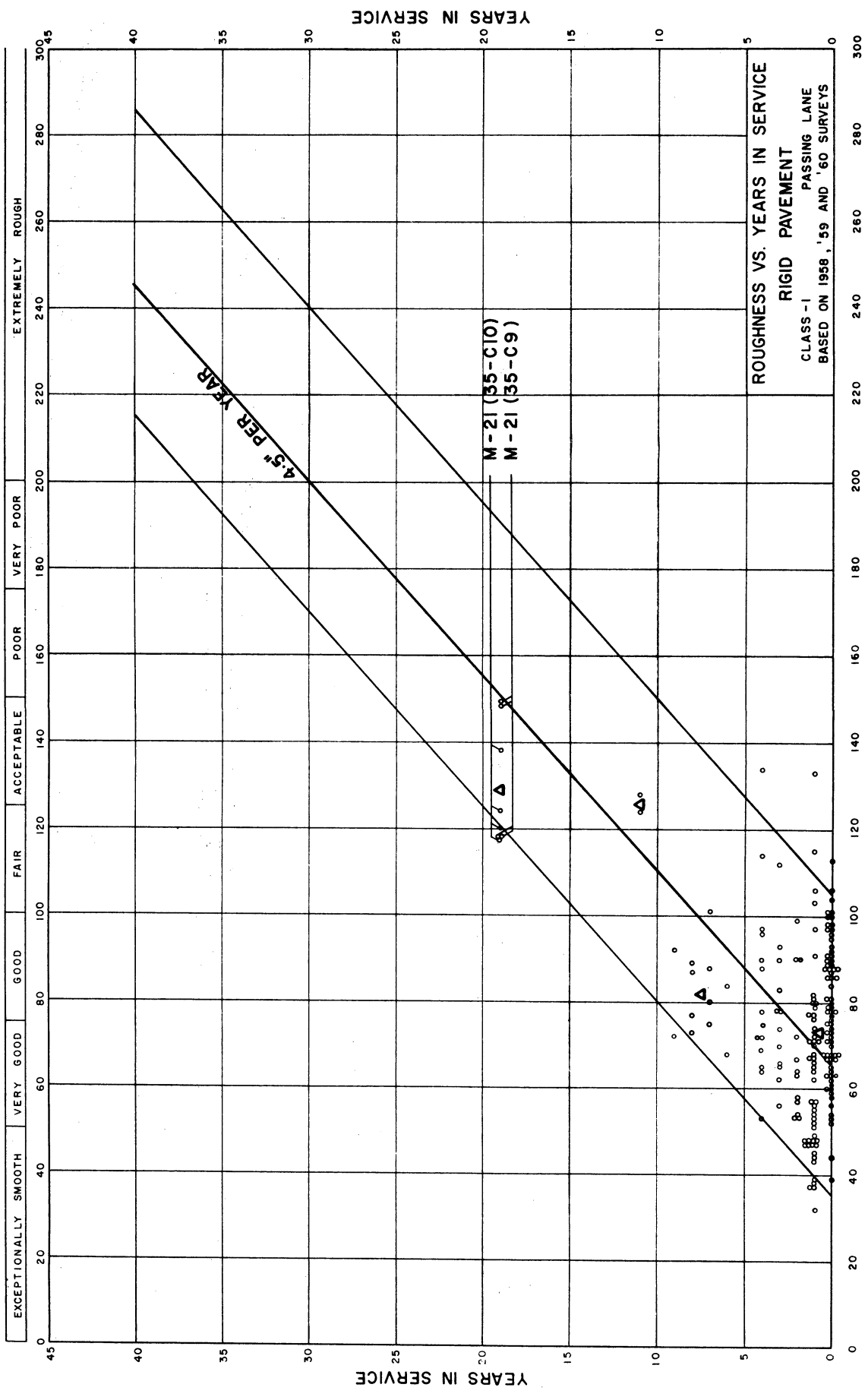


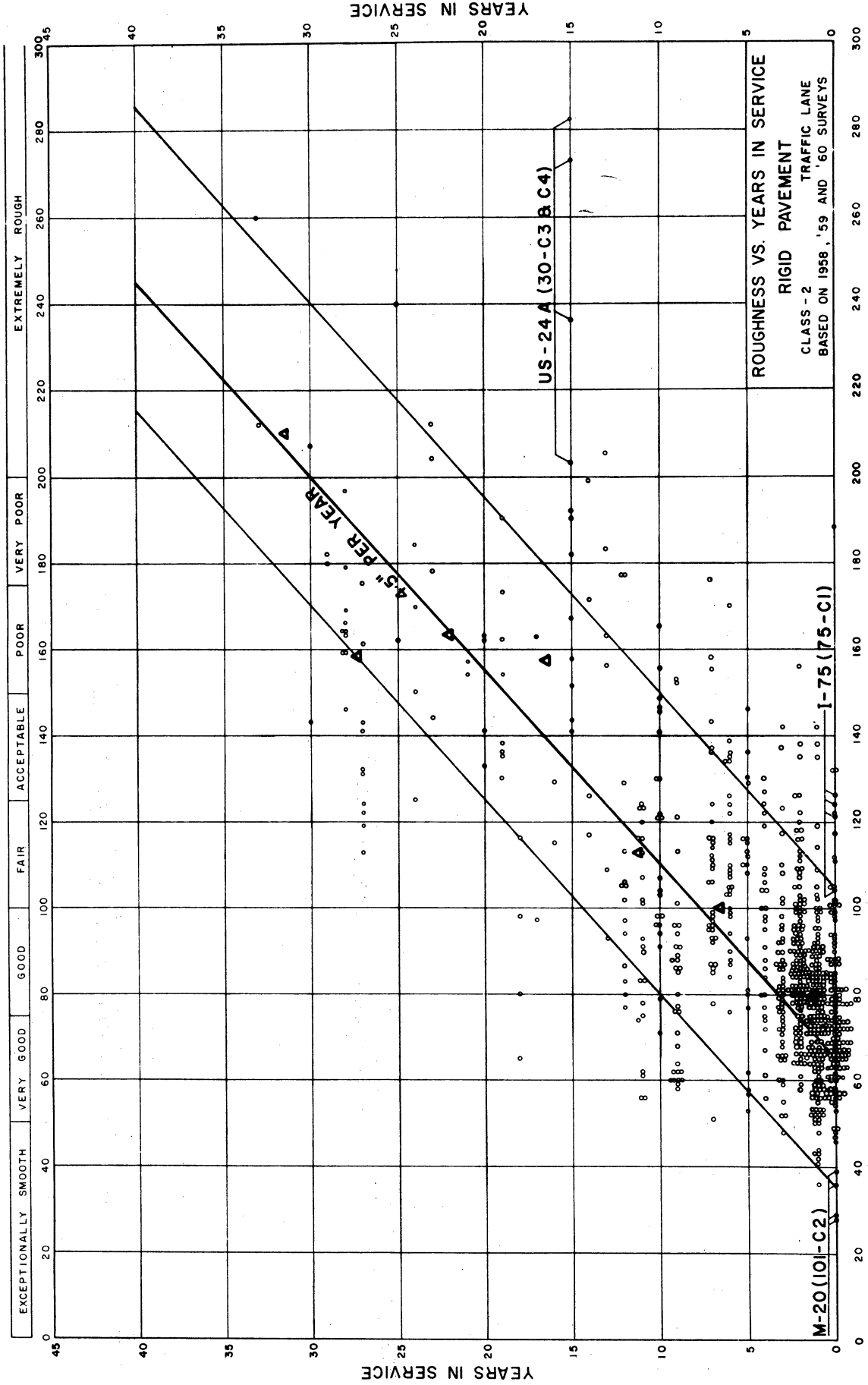
FIGURE - 2



ROUGHNESS VS. YEARS IN SERVICE
 RIGID PAVEMENT
 CLASS - I
 PASSING LANE
 BASED ON 1958, '59 AND '60 SURVEYS

ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

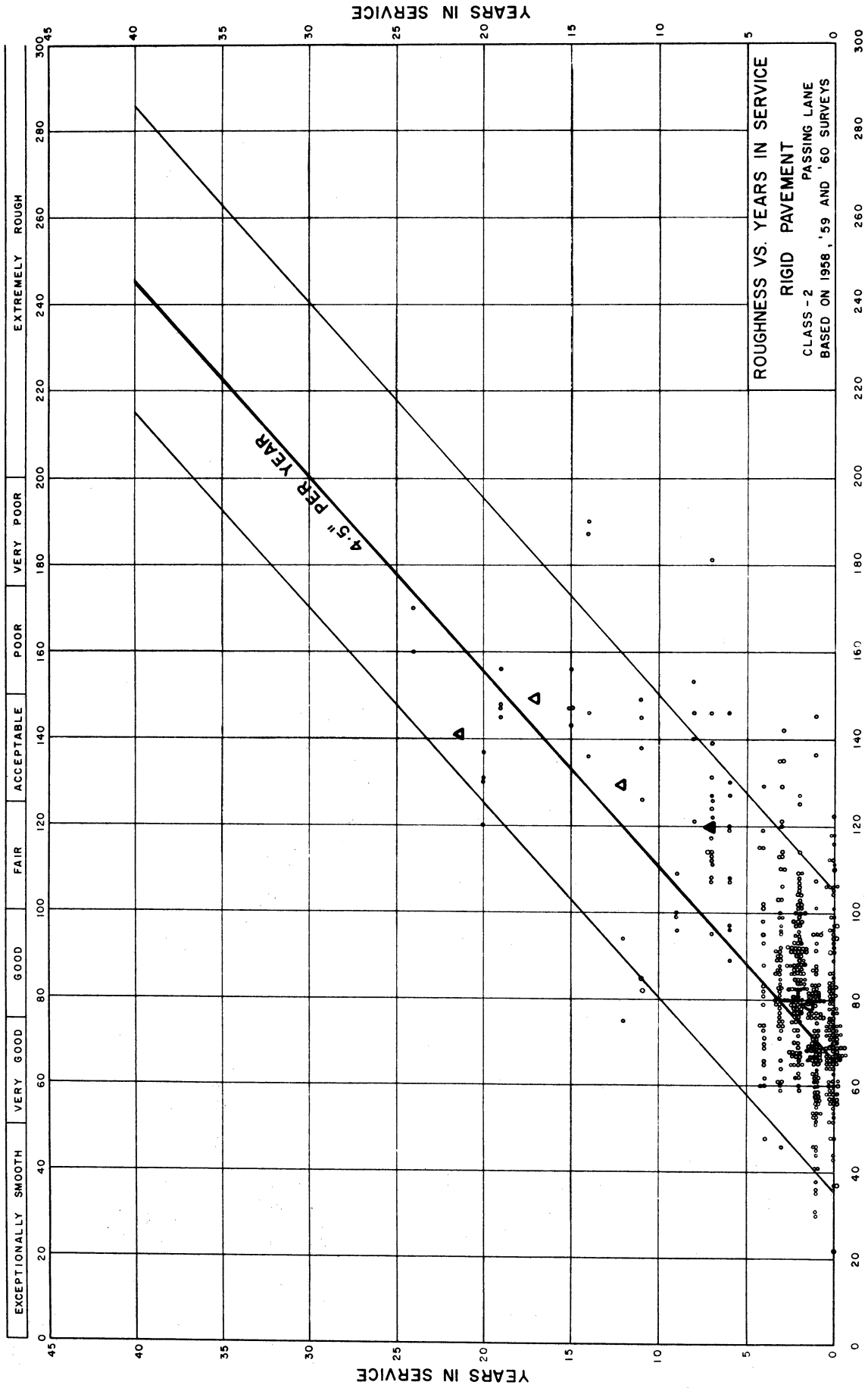
FIGURE - 3



ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

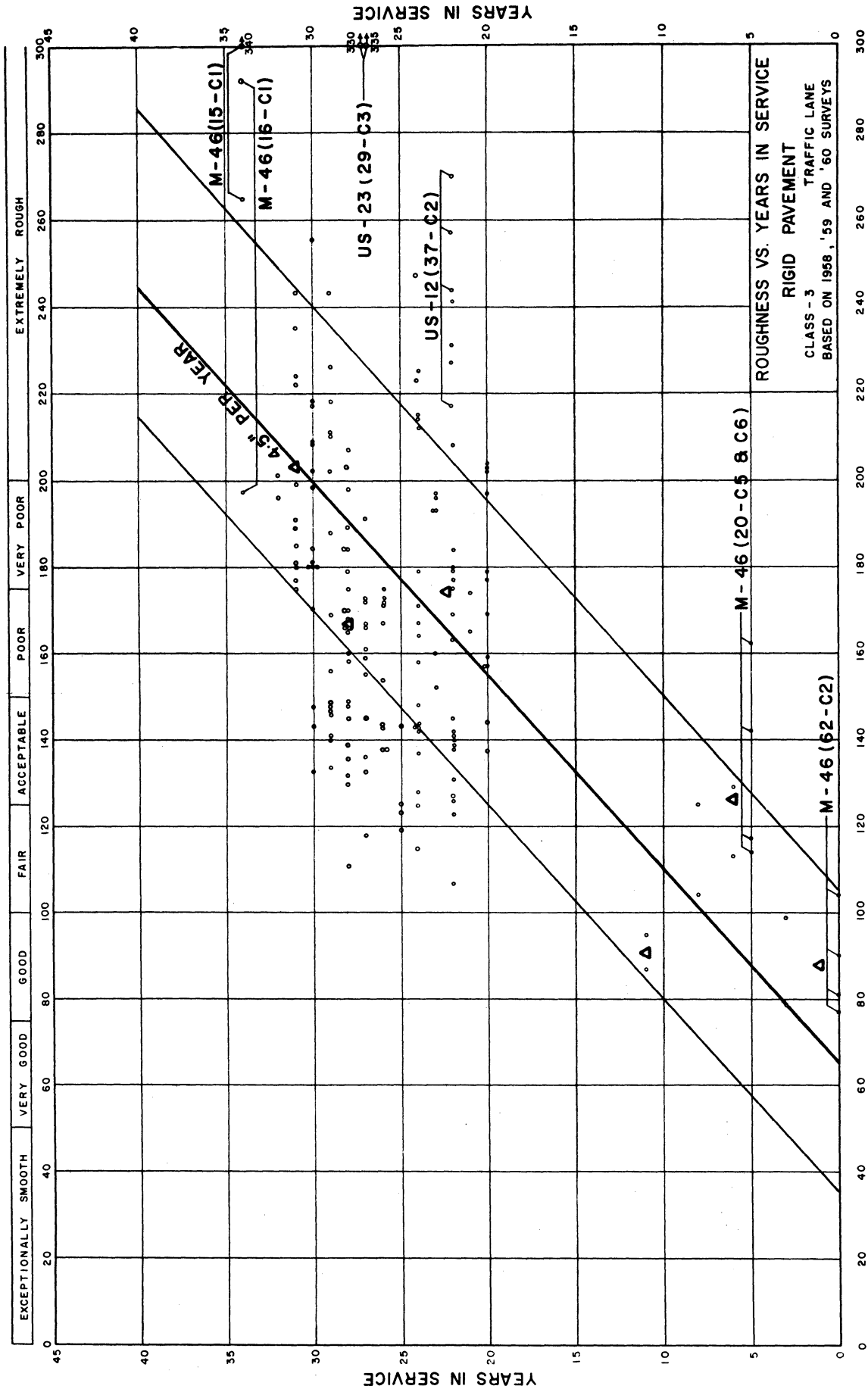
ROUGHNESS VS. YEARS IN SERVICE RIGID PAVEMENT CLASS - 2 TRAFFIC LANE BASED ON 1958, '59 AND '60 SURVEYS

FIGURE - 4



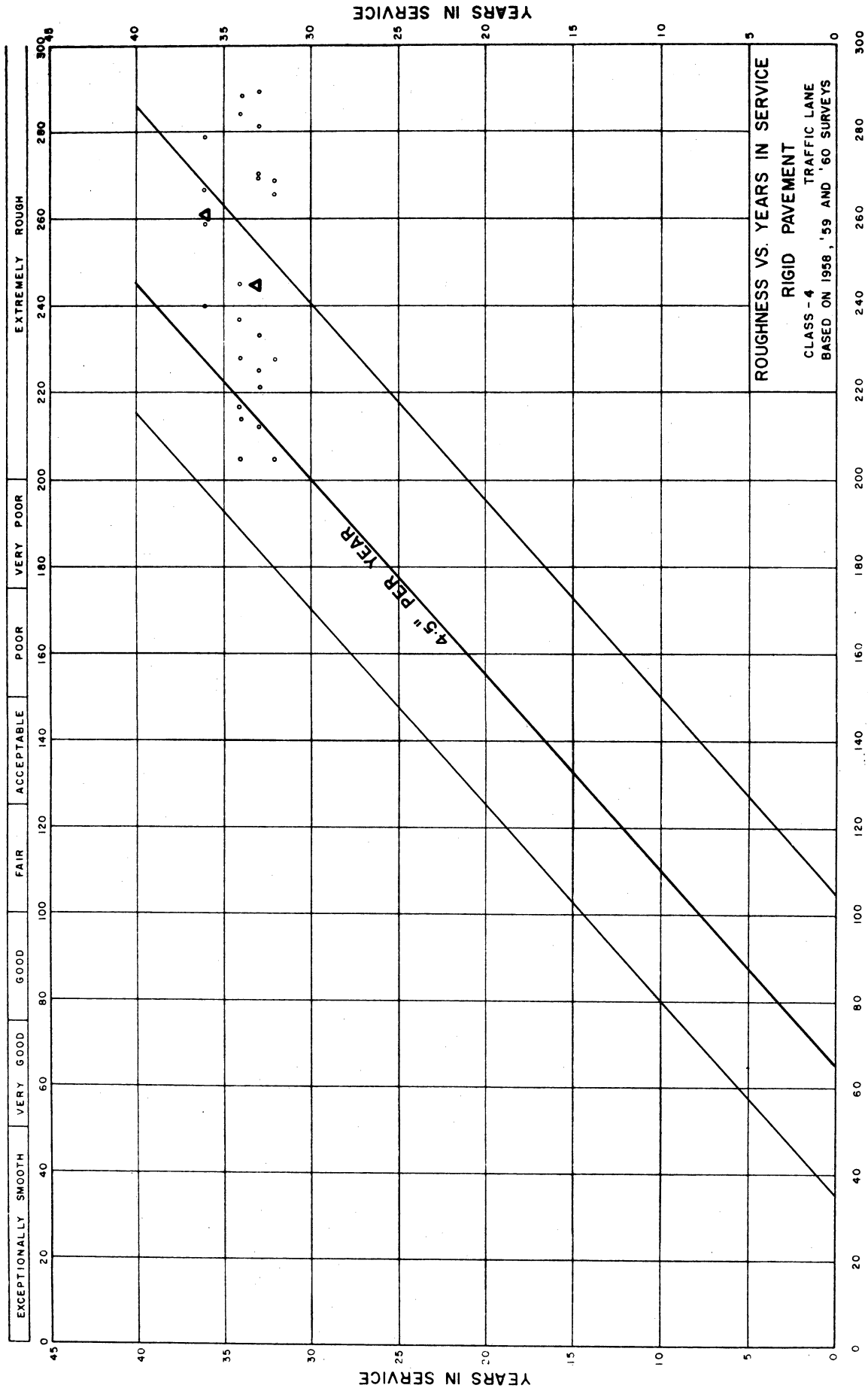
ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

FIGURE - 5



ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

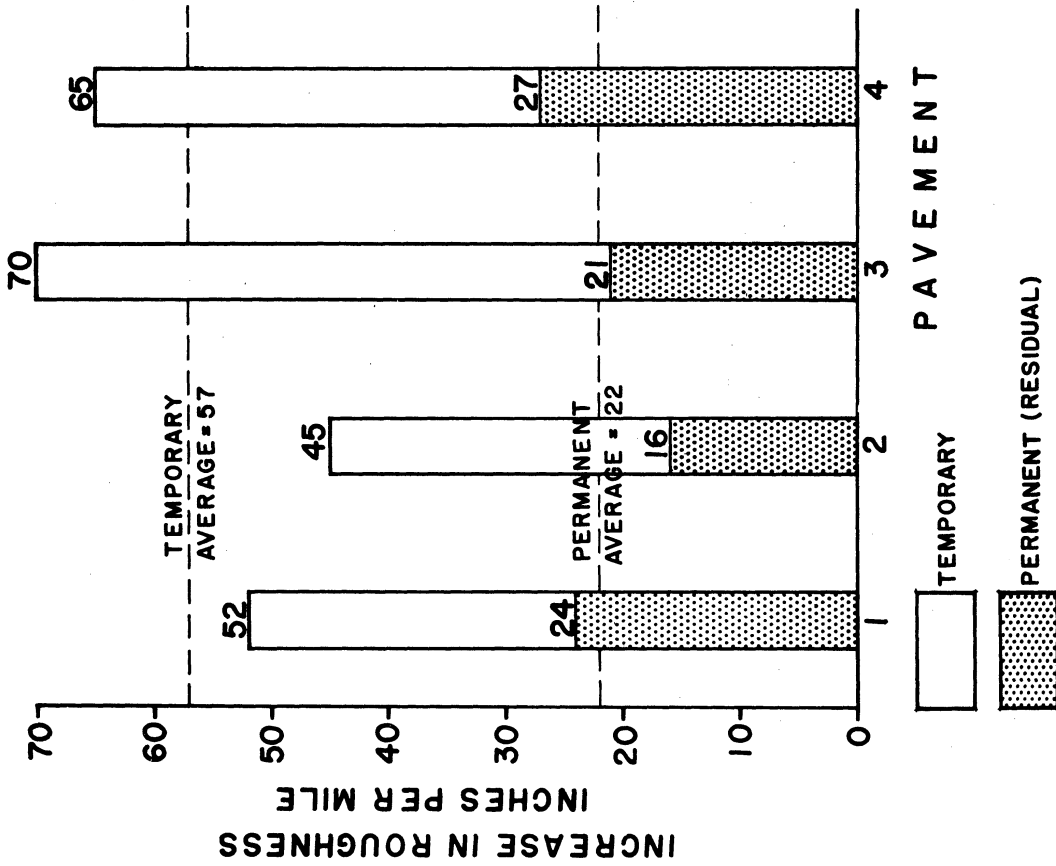
FIGURE - 6



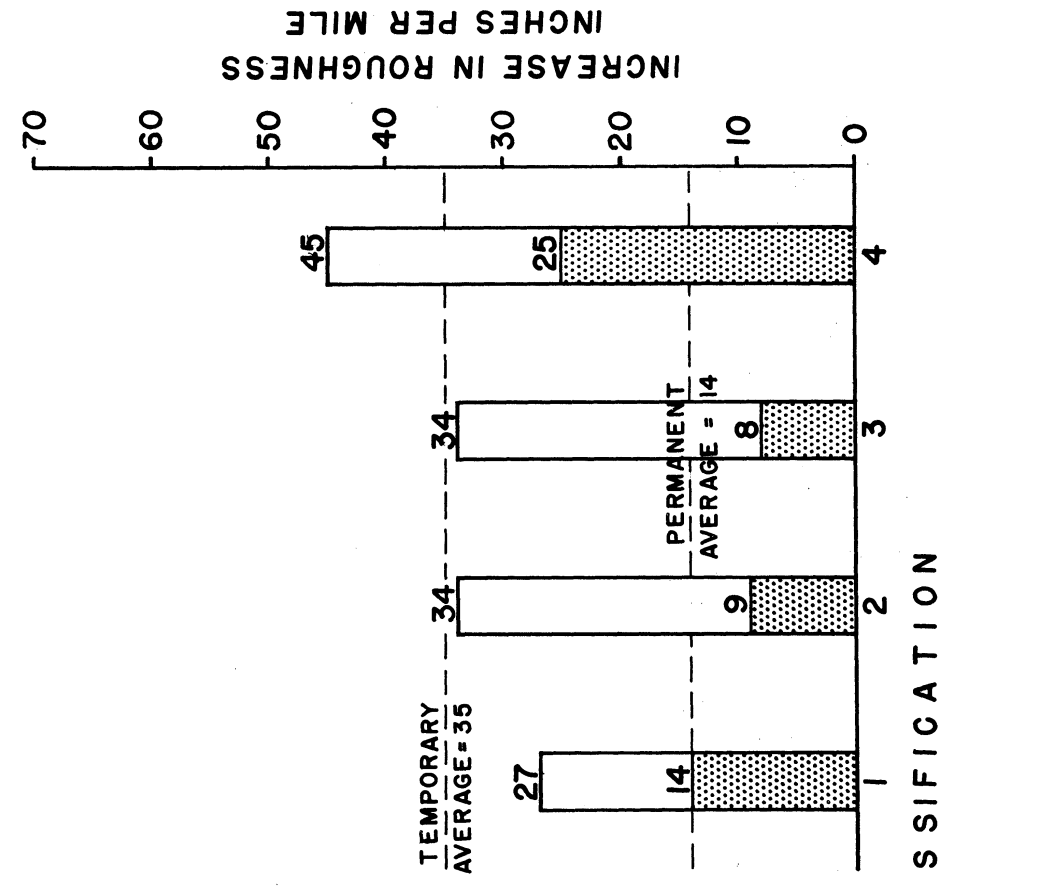
ROUGHNESS INDEX IN INCHES OF VERTICAL DISPLACEMENT PER MILE (R.I.)

FIGURE - 7

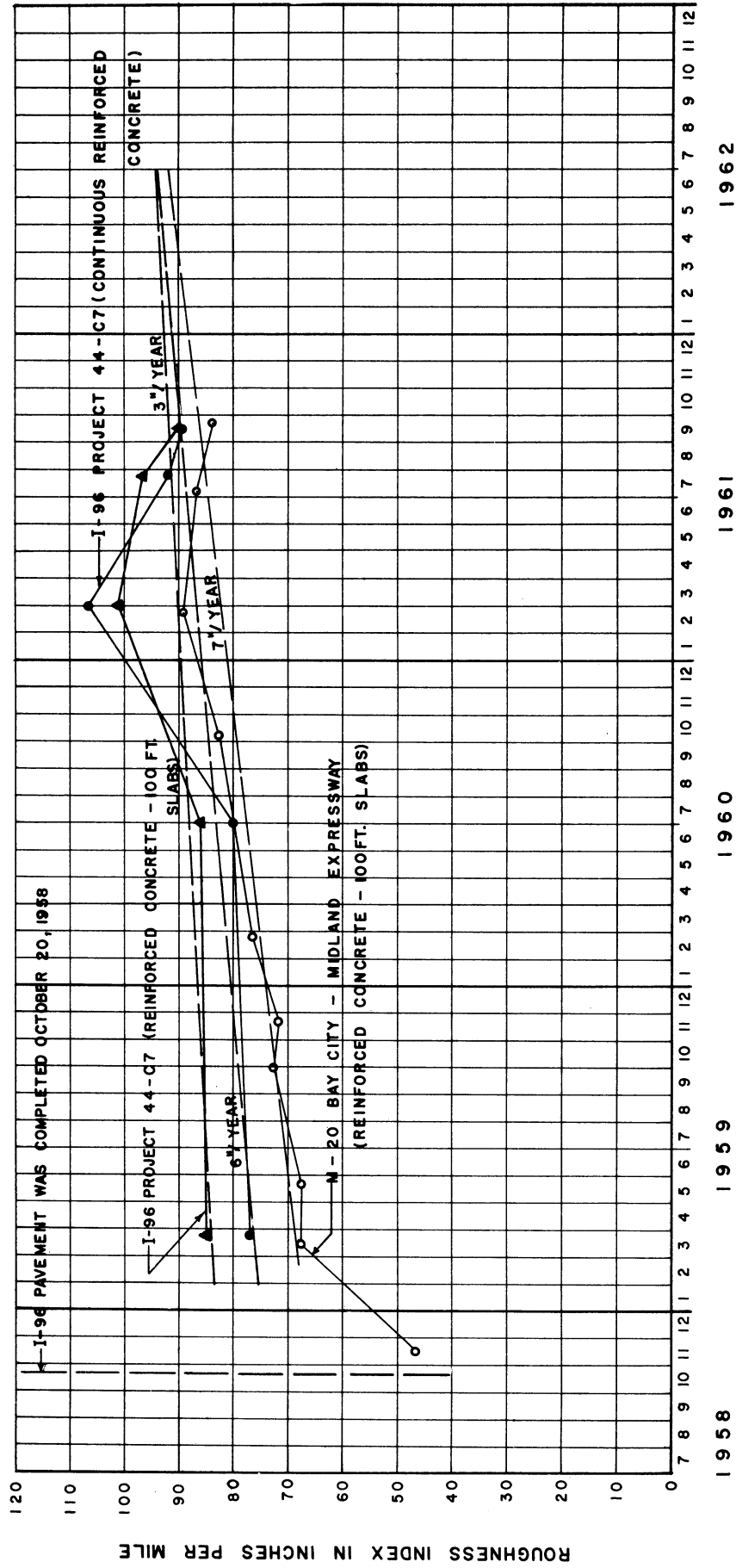
OUTER WHEEL PATH



INNER WHEEL PATH



SEASONAL FLUCTUATION IN PAVEMENT PROFILE



CUMULATIVE CHANGE IN ROUGHNESS
ON SPECIFIC PROJECTS

FIGURE - 9

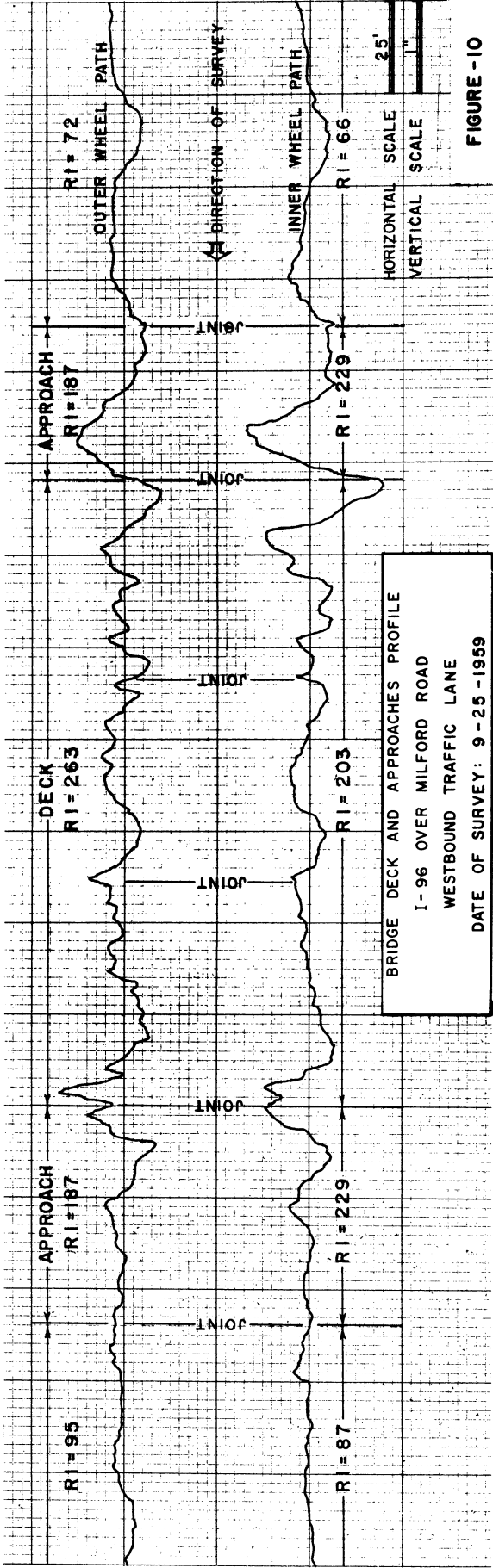
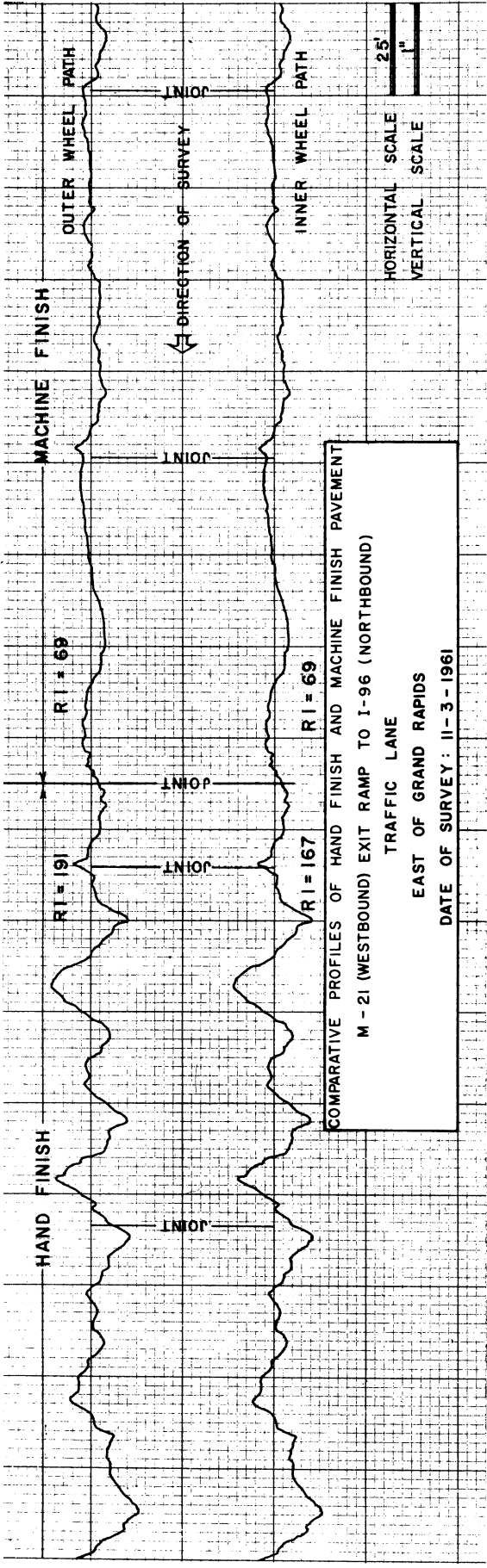


FIGURE - 10

DEPARTMENT OF CIVIL ENGINEERING

THE UNIVERSITY OF MICHIGAN

ANN ARBOR, MICHIGAN

MICHIGAN PAVEMENT PERFORMANCE STUDY

DESIGN, MAINTENANCE, AND PERFORMANCE OF RESURFACED PAVEMENTS

AT WILLOW RUN AIRFIELD

BY

WILLIAM S. HOUSEL

Professor of Civil Engineering

JANUARY, 1962

ACKNOWLEDGMENT

During the past fifteen years that Willow Run Airfield has been used as a field laboratory for studying pavement design and performance, the University of Michigan has had the support and cooperation of a number of agencies who have contributed to that program. These include the Federal Aviation Agency, Michigan Department of Aeronautics, Airlines National Terminal Service Company, Inc., the Wire Reinforcement Institute, and the past* and present sponsors of the Michigan Pavement Performance Study, now being conducted as part of the Highway Planning Survey Work Program in cooperation with the Bureau of Public Roads. G. R. Ingimarsson, Research Fellow of the National Petroleum Refiners Association, is assisting in the current studies.

* Michigan Trucking Association
American Trucking Associations, Inc.
Automobile Manufacturers Association

DESIGN, MAINTENANCE, AND PERFORMANCE OF RESURFACED PAVEMENTS

AT WILLOW RUN AIRFIELD

By

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INTRODUCTION

Inasmuch as the original design and construction is an important factor in the subsequent maintenance and performance of concrete pavement, it is appropriate to start this paper with a brief history of the pavement construction at Willow Run. 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 airfield was originally designed as part of a major plant for the manufacture of B-24 long range bombers, to be used for the operation and testing of these planes. In 1942, a training facility consisting of an apron, taxiways, and extension of the runways was built at the east end of the field under the supervision of the U. S. Army Corps of Engineers. In 1943, the factory apron at the west end of the field was enlarged and several additional taxiways were constructed by the Defense Plant Corporation.

The entire field is on an outwash plain of sand and gravel, varying in depth from a few feet to as much as 30 or 40 feet, deposited on a

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waterworked clay till plain, within the limits of the post-glacial Lake Maumee, now Lake Erie. Subgrade conditions over most of the field were almost ideal, although in the north-central portion there was a lowlying area of virgin hardwood with a heavy accumulation of forest debris and organic material and a water table close to the surface. Subdrainage was provided to lower the water table beyond the normal depth of frost penetration that would affect the paving, but it proved difficult under the emergency construction conditions then in effect to enforce effective controls of the grading operation that should have been recognized. Failure to remove topsoil and organic matter within the paved areas and to make more adequate provision for surface drainage were shortcomings that affected pavement behavior in later years; their influence was clearly shown in the subsequent performance of the pavement.

In 1946, after the war, the University of Michigan acquired title to Willow Run Airfield as war surplus, with the primary objective of developing the facility as a research center. Concurrently with acquisition of the field, arrangements were made to lease it to a group of large commercial airlines serving Detroit; and, since that time, it has served as the major airport terminal for the City. The operation and maintenance of the airport was subsequently placed in the hands of the Airlines National Terminal Service Company, Inc. (ANTSCO), an arrangement which has remained in effect up to the present time. Acquisition of the airfield property placed on the University of Michigan a certain responsibility, as stated in the provision ". . . that the entire landing area . . . , and all improvements, facilities, and equipment of the airport property shall be maintained at all times in good and serviceable condition to insure its efficient operation." This responsibility was in turn delegated to ANTSCO as part of the rental agreement

under which they have operated the field. This responsibility was taken seriously by all parties concerned; in addition to normal maintenance required for operation of the field, a periodic pavement condition survey has been made to keep an accurate inventory of the physical condition of the paving over the period of years that the field has been 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. From the writer's viewpoint, the airfield has provided an unusual opportunity as a field laboratory for studying pavement design and performance; it is from this program that the basic information for this paper has been drawn.

PAVEMENT CONSTRUCTION AND MAINTENANCE AT WILLOW RUN

Fig. 1 is an aerial photograph of Willow Run Airfield, with the manufacturing plant in the left foreground and the main west apron and hangar in the central foreground with the runways and taxiways extending beyond, to the training facility at the east end. There were approximately 1,500,000 square yards of concrete pavement, roughly equivalent to 115 miles of 22 foot highway pavement. As noted in the introduction, there were three stages of construction during the period from 1941 through 1943. All of the concrete pavement was unreinforced, with the main apron, runways, and taxiways built in 1941 with an 8-6-8 thickened edge section. The east apron and connecting taxiways, built in 1942, have been little used other than for training and experimental purposes and will not be discussed in detail in this report. Additions to the main west apron, in 1943, were

built with an 8-6-8 thickened edge section; this construction also included the outer taxiway and a second apron at Hangar No. 2 on the right hand side of the photograph.

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 pour, 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. Paved runways were 160 feet wide and taxiways 80 feet wide. In general, the dummy joints were formed by paper inserts rather than being grooved as shown on the plans. The only steel called for on the plans was $1/2$ inch round steel dowels 24 inches long at 30 inch centers in the longitudinal dummy joint for 15 feet on both sides of the transverse expansion joints. Subsequent slab replacements have revealed some departures in the "as-built" pavements from the details on the plans, but none of any particular significance in the performance of the pavement as a whole.

However, there were some significant variations in construction practice, between the 1941 and 1943 construction, which developed some sharp contrasts in performance, revealed by subsequent pavement condition surveys. Fig. 2 is an aerial photograph taken at the south end of the main west apron showing typical sections of the 1941 apron and the 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 discussed in this paper. Although the pavement at this time had been subjected to almost negligible service in terms of load

repetition, the 1943 additions and the curved taxiway are beginning to show a considerable amount of transverse cracking, with virtually no cracks having developed in the 1941 construction.

Fig. 3 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 already shows serious crack development, with transverse cracks in the center of a large percentage of the 25 foot slabs and an unusual pattern cracking developing in certain lanes, with some slabs having already 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. A survey was made in 1950 of cracking over the entire airport to make an approximate evaluation of the type of cracking developing in the two different paving projects. Cracks were classified as transverse, longitudinal, or diagonal, as summarized in Table 1, with no attempt being made at this time to isolate the special pattern cracking referred to above.

One of the most interesting developments shown in Fig. 3 is this pattern cracking developing along the edge of every fourth lane in the 1943 apron. The cause of this incipient cracking was traced to the fact that this section of the apron was poured in alternate 20 foot widths and the concrete mixer was permitted to travel on the recently completed 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 had not been completely cured and its strength was not sufficient to carry the concentrated load of the mixer at the edge of the slab. As a result, these

frequently accepted compromises in paving practice contributed more to the deterioration of this badly cracked pavement than any other factors in design or construction.

Fig. 4 is another photograph of the same area taken in 1954, just prior to the first resurfacing project which is the main subject of this report. After some eight years of airline service, the apron built in 1943 has been reduced to a block pavement over a considerable portion of the area, with very little structural continuity in the original slabs. Some of the earliest slab replacements, which were also unreinforced, have also been badly cracked in this area of heavy traffic concentration just off the end of the south loading ramp in a path traveled by a large percentage of the planes going to the main taxiway.

Early Maintenance of Paved Areas

The preceding discussion of the original pavement construction, showing a sharp contrast between the 1941 and 1943 construction is a background for a discussion of pavement maintenance in these areas and emphasizes the importance of sound construction practices, the neglect of which may defeat the most important objectives of planning and design.

During the first eight years of commercial airline operation, ANTSCO carried out an effective and timely maintenance program calculated to keep ahead of the pavement deterioration that was progressing in several critical areas, including the inferior 1943 construction. This maintenance consisted of an annual crack and joint filling program carried out in the fall of the year when cracks had opened up, following replacement of badly cracked slabs in those areas in which it was practicable to do so. Pavement deterioration was being measured by the periodic pavement condition

surveys in terms of the cracking pattern or loss of structural continuity. After each periodic aerial survey of the entire paved area, the structural continuity of the pavement was evaluated in terms of a "continuity ratio", defined as the average length of pavement slab between cracks and joints divided by a selected standard length representing normal subdivision of the concrete pavement due to shrinkage and temperature differentials, regardless of loading and structural strength. The standard slab length, independent of loading effects, was selected as 15 feet. Thus, the initial continuity ratio for a 25 foot slab length would be 1.67, while a continuity ratio of unity, or an average slab length of 15 feet, would be considered satisfactory from the standpoint of structural adequacy. Continuity ratios less than unity, or slab lengths less than 15 feet, would represent excessive cracking and evidence of structural weakness in the pavement itself or in the supporting subgrade.

Summarizing the results of the periodic pavement condition surveys, Fig. 5 shows the change in continuity ratio of typical paved areas during the period from 1946 through 1954. Runway 4L-22R is one of the main diagonal runways of the 1941 construction, considered representative of the well-built pavement with excellent subgrade conditions and good construction control. By 1954, the continuity ratio had only been reduced from 1.67 to 1.50; this good performance can be considered as a basis for comparison with the other areas to be considered. Taxiway B and Runway 9L-27R, with continuity ratios reduced from 1.67 to approximately 1.10, were still in reasonably good condition, but these were areas of known subgrade deficiency due to the inclusion of unstable organic material in the subgrade during the grading operation. The 1941 apron, with a continuity ratio of 1.19, may be showing some influence of greater load repetition, but is still

rated as satisfactory and representative of the better 1941 construction. This performance is in marked contrast to that of the 1943 apron, where the average continuity ratio has been reduced to 0.44, indicative of excessive cracking and loss of structural continuity.

BITUMINOUS RESURFACING

The first resurfacing project, in 1955, included the center taxiway and the most badly cracked portion of the main apron where the annual filling of cracks and joints had become a prohibitive maintenance procedure. While it will not be discussed in detail as part of this paper, it may be noted in passing that additional bituminous resurfacing of the concrete pavement has been done in 1959, 1960, and 1961 to maintain efficient operation in spite of the continued deterioration of the unreinforced concrete pavement and to reduce the cost of annual maintenance. Ultimately, if justified by the continued operation of the field as a major airport, it is planned to resurface most if not all of the concrete pavement.

Subsequent discussion in this paper will be devoted to a survey of reflected cracking in selected test areas and the general performance of the 1955 resurfacing, where several variations in construction details were undertaken on an experimental basis. These test areas have been designated in Fig. 6, with three areas, T-1, T-2, and T-3, on the center taxiway, and two areas, A-1 and A-2, on the 1943 apron. Areas A-3 and A-4, on the main apron, are of more recent origin and represent resurfacing carried out in 1960, in which one experimental area included a wearing course of Epon asphaltic concrete, laid primarily as a protection against spillage of gasoline and oil in the apron area. While it might be termed an unanticipated

dividend, the Epon surface course, in its first year of service, showed a substantially greater resistance to reflected cracking in comparison with the conventional bituminous resurfacing. For this reason it has been included in this discussion as a promising development in resurfacing of airport paving.

1955 Bituminous Resurfacing

The first resurfacing project, in 1955, consisted of a 1-3/4 inch bituminous concrete binder course (CAA Specification P401-A) and a 1-1/4 inch surface course (P401-C). Before laying, the surface was swept with a power broom which removed all loose material from scaling and disintegration at the joints. A bituminous tack coat of AE-2 asphalt emulsion (P-603) was applied to the concrete surface at a rate of 0.09 gallons per square yard. Where practicable, the badly cracked slabs were replaced; but, this was not done in the apron area of 1943 construction, which was badly cracked throughout. Welded wire fabric, 3 by 6 inch No. 10 gauge in both directions, was placed over the area to be resurfaced, with the exception of certain test sections where it was left out for a comparative study on reflected cracking. Installation of the welded wire fabric has been described elsewhere so will not be given in detail in this report.¹ The welded wire fabric was placed directly on the concrete surface, with the transverse wires at 3 inch spacing on the bottom; it was found that it would penetrate by wedging action into the binder course while it was being rolled. With proper handling of the wire sheets, no particular difficulty was encountered in the wires extruding through the binder course or snagging on the

¹ Wakefield, F. G., "The Practical and Laboratory Use of Wire Fabric in Bituminous Resurfacing at Willow Run Airfield", Technical Bulletin No. 215, American Road Builders Association, Washington, D. C., 1956.

finishing machine. In the rare cases in which the fabric was still exposed in the top of the binder course, it was satisfactorily covered by the 1-1/4 inch surface course.

Surveys of Reflected Cracking

The first survey to determine the reflected cracking was made in 1957, approximately two years after the resurfacing. All visible cracks in each test area were outlined with white paint and an aerial photograph then taken to record the cracking pattern. These photographs were then checked in the field by visual inspection. Most of the test areas were sealed in 1960, with exceptions that will be noted. A second crack survey of all test areas was then made in October, 1961, and will be presented in each figure for comparison with the 1957 survey. No aerial photographs were taken in 1961; the crack survey was made by field inspection, the cracking pattern then being sketched on the plan of each test area.

The data on reflected cracking are presented in graphical form on a series of charts, with certain details presented in terms and sequence common to all of the subsequent figures. Each test area is identified by a serial number with letters relating to the location of the test area, which has also been indicated on a small insert plan of the airfield. The reflected cracking is given as a percentage of the total lineal feet of cracks and joints in the underlying concrete pavement. The cracks and joints in the original pavement are shown by full lines on the plan view and are taken from the last aerial photograph, made in 1955, and checked by field survey shortly before the resurfacing project began. The reflected cracking at the time of the 1957 and 1961 surveys has been shown by a series of dots outlining those cracks and joints which have been reflected through the bituminous surface. Variation in the construction details of the bituminous

surface which are being compared is indicated in connection with that portion of each test area to which it refers.

Test Area 1, shown in Fig. 7, is on the center taxiway and was intended to show the effect of varying the thickness of the bituminous surface from 3 inches in Area T-1-a to 4 inches in Area T-1-b, both without mesh or welded wire fabric. In the 1957 survey, shown at the top of the figure, the reflected cracking was 32 per cent in partial Area a and 24 per cent in partial Area b, with an 8 per cent differential in favor of the greater thickness of bituminous surfacing. It should also be noted that a large percentage of the reflected cracking was at pavement joints, with very little of the slab cracking being reflected through the surface. The results of the 1961 survey, after the surface had been sealed in 1960, show a moderate increase in reflected cracking in both areas, with the same differential of 8 per cent in favor of the greater thickness of bituminous surface. The pattern of reflected cracking in both surveys is closely comparable, with the same cracks showing up in 1960 that were observed in 1957 with some increase which is mostly over pavement joints rather than the slab cracking pattern.

Test Area 2, shown in Fig. 8, is on the central taxiway and gives a comparison of reflected cracking in the 3 inch bituminous surface, with and without welded wire fabric or mesh. Reflected cracking in the area with mesh involves a new type of cracking that makes an accurate quantitative estimate difficult and dependent on a matter of definition. The plan view of the area, at the top of Fig. 8, shows a ladder type of cracking along the center of several lanes where there were no cracks in the underlying concrete pavement. This is related to the fact that the welded wire fabric was ordered in sheets 9 feet, 6 inches wide, laid over the

longitudinal joint with a 6 inch clearance between sheets at the center of the 10 foot lanes. This might be regarded as a construction defect which could possibly be corrected by providing continuous reinforcement in some manner between the sheets of wire mesh.

Because of discontinuity in the reinforcement, opening of joints or movement of concrete slabs is consequently translated from the joint to the gap between sheets of wire mesh. Similarly, opening of transverse joints in the concrete pavement is spread along the wire mesh by the transverse wires until sufficient movement has accumulated to cause a visible crack, producing the ladder type of cracking pattern. Whether this is reflected cracking or not is a matter of definition. In the percentages reported as reflected cracking, the attempt has been made, uncertain at best, to segregate the cracks reflected directly from the underlying pavement from that cracking transferred to previously uncracked areas by the wire mesh. The direct reflected cracking is the first percentage shown; the total of both types of cracking is shown in parentheses.

It must be recognized that percentages of the total lineal feet of joints and cracks in the underlying concrete pavement where wire mesh has been used are not a true measure of the benefit to be derived from the use of welded wire fabric in the bituminous resurfacing. The choice must be made in this case between a fewer number of cracks with wider opening and a larger number of cracks with less width. In the surveys here reported, involving relatively large areas, no study was made of crack widths. All visible cracks, including hairline cracks, were mapped and reported. However, it was observed that translated cracks in the ladder type pattern had substantially less opening than the cracks reflected directly above joints in the concrete pavement.

The percentages reported in the 1957 survey of Area T-2, of 25 and (39) per cent with mesh and 32 per cent without mesh, must be viewed with the above qualification relating to crack width. In terms of nothing more than the lineal feet of cracks, one would conclude that there is about a 7 per cent differential, in terms of direct reflection of cracks, favoring the use of wire mesh, but about the same unfavorable percentage in terms of total cracking, including translated cracks. The significance of decreased crack width in translated cracking becomes apparent in the 1961 survey of the same area after it had been sealed in 1960. In the unreinforced area, the reflected cracking increased from 32 to 37 per cent from 1957 to 1961, even with the benefit of sealing in 1960. In the reinforced area, the direct reflection increased from 25 to 27 per cent, a negligible figure in practical terms which is offset by a favorable differential in terms of total cracking. Including the translated cracking in the total percentage, there was a decrease in reflected cracking from (39) to (33) per cent, which credits the wire mesh with a measurable improvement. It is more significant that this change in percentage is related to the translated cracking, which is emphasized by the almost complete disappearance of the ladder type cracking pattern in the 1961 survey. Thus, cracks of decreased width in the reinforced area are more effectively sealed, a factor showing more clearly the benefit of the welded wire fabric in the continued maintenance of the surface.

Test Area T-3, shown in Fig. 9, duplicates the test conditions in Area T-2, providing another comparison between areas with and without welded wire fabric. However, in this case, the benefit of the wire mesh is much more apparent in both the 1957 and 1961 surveys and the differentials in percentage of reflected cracking substantially greater. Again, the pavement

joints are the primary source of reflected cracking which the wire mesh converts to the ladder type of translated cracking, a considerable portion of which is eliminated in the 1961 survey after sealing in 1960.

Area A-1, shown in Fig. 10, is located in the badly cracked portion of the 1943 apron where the cracking pattern is so complex that it practically defies accurate analysis. It must also be kept in mind that this is an area probably subjected to the heaviest concentration of load repetition of any location on the airfield. A large percentage of the planes moving to and from the main runways traverse this area, involving slow moving planes with load application close to the static conditions most severe on airport paving. Under these conditions, it must be presumed that the 1955 cracking pattern in the underlying concrete pavement has also been modified by additional cracks hidden by the bituminous surface.

As a matter of fact, the most pertinent observation that could be made about this 6 inch unreinforced concrete pavement of inferior construction, subsequently reduced to a series of concrete blocks, is that it has made a remarkable showing in terms of general pavement performance. Furthermore, from the standpoint of pavement design, it is hard to imagine a more spectacular demonstration of the dominant role of unlimited subgrade support supplied by the natural sand and gravel subsoil on which the pavement rests.

In spite of the complicating factors noted, the percentage of reflected cracking shown by Area A-1 is comparable to those of the other test areas, particularly when the greater concentration of load repetition is given due weight. The comparison is again between 3 inch bituminous resurfacing, with and without welded wire fabric. It should be noted also that

the area without mesh is relatively small and does not include one of the double lanes of advanced pattern cracking.

In the 1957 survey, shown at the top of Fig. 10, the unreinforced area showed a reflected cracking of 44 per cent; in comparison, the reinforced area had 33 per cent direct crack reflection, or (38) per cent including the translated cracking. In both cases, there was a measurable differential in favor of the reinforced area. At the bottom of Fig. 10, the results of the 1961 survey of the areas sealed in 1960 show the unreinforced area having a reflected cracking of 47 per cent, as compared to a direct reflection in the reinforced area of 29 per cent, or a total of (30) per cent including translated cracking.

The marked differential in favor of the reinforced areas is again related to the effective sealing of the translated cracks of decreased width, illustrating the most apparent benefit of the welded wire fabric. A considerable portion of Test Area A-1 was not sealed in 1960 and thus, as a special case, provides some measure of the total reflected cracking in the six years from 1955 through 1961. The direct crack reflection is estimated as 49 per cent and the total cracking, including translated cracking, as (75) per cent. As previously pointed out, there are several indeterminate factors involved in this complex crack pattern that can not be eliminated. The figures are nevertheless interesting for comparison, even though qualified by unavoidable uncertainties.

Special Experimental Areas

There are two special test areas yet to be discussed where experiments were made which go somewhat beyond the main investigation of reflected cracking. In Test Area A-2, shown in Fig. 11, an experiment was

tried on a somewhat limited scale which might serve as an example of uninhibited exploration typical of the freedom sometimes associated with academic circles. In this case, it was decided to remove the badly cracked concrete in one lane and replace it with a crushed aggregate base which, at any rate, was well compacted to give it every chance for survival. The results are shown in Fig. 11 and really turn out to be quite interesting.

In order to maintain a common basis for comparison, the cracking pattern of the original concrete pavement was retained as a common denominator. Even though this is a rather tenuous hypothesis, it is directly related to the total lineal feet of cracks and retains some relationship to the badly cracked pavement in the lanes with which it is compared. The area involved is supplied with wire mesh and is at a location subjected to heavy load repetition, just off the end of the center taxiway. The results of the 1957 survey indicate that the relative performance of the crushed aggregate base is close to that of the concrete pavement. In the 1961 survey, after sealing in 1960, the comparison is still quite close although there is a moderate increase, 7 to 8 per cent, in the cracking with the aggregate base.

Epon Asphaltic Surface

Test Areas A-3 and A-4 have been brought into the study of reflected cracking to present interesting information on a comparatively recent development. Maintenance of airport paving in service areas, where spillage of gasoline and oil causes disintegration of conventional bituminous mixtures, has always been a serious problem. At Willow Run, several types of sealing have been tried from time to time with reasonable success.

Jennite, a tar derivative developed and widely used for this purpose, was applied to the service area on the main apron that was resurfaced with bituminous concrete in 1955. The bituminous resurfacing, in 1960, of the service area on the main apron was treated with a double seal of a tar emulsion slurry with fine sand in suspension. All of these materials were in liquid form spread with a distributor and squeegeed to obtain more uniform application. The Jennite seal has given excellent performance and is still effective after six years of service. The coal tar slurry has been generally satisfactory, but has shown some defects believed traceable to application of the seal before the bituminous surface had completely cured.

Area A-3, shown at the top of Fig. 12, was supplied with a 1 inch wearing course, using an Epon asphaltic binder in place of the conventional asphaltic material. Representatives of Shell Oil Company participated in the experiment, designed the mixture, and supervised the laying. From the standpoint of resistance to spillage, its performance has been good for the one year it has been in service. There has been some evidence of softening in spots where compaction of the mixture was inadequate during rolling. These spots are generally at the junction between lanes in which the surface mixture was laid. This represents a construction defect yet to be overcome and it has been recognized as such by those interested in its development. The same defect at the junction between lanes shows up in the cracking pattern in Fig. 12.

With respect to reflected cracking, the Epon surface in Area A-3 is compared with the adjoining Test Area A-4 with the conventional 3 inch bituminous resurfacing. No welded wire fabric was used in either area; thus, there is some basis for comparison with other test areas, conditioned by the shorter period of service and the fact that the underlying

concrete pavement laid in 1941 has a very moderate cracking pattern. Most of the cracking in both areas is over pavement joints; the differential in cracking after the first year of service is quite striking. The Epon area has direct reflected cracking of 13 per cent, as compared to 54 per cent in the conventional bituminous surface. If the special cracking between lanes is included in the Epon area, the total cracking would be 30 per cent, still leaving a substantial differential in its favor.

GENERAL EVALUATION OF PAVEMENT PERFORMANCE

The primary objective of this paper has been to present information on the behavior of bituminous resurfacing of old concrete pavements and data on the control of reflected cracking. At the same time, it seems appropriate to comment briefly on the evaluation of pavement performance on the entire airfield in more general terms. During most of the period of some fifteen years that the airfield paving has been subjected to commercial airline operation, pavement performance has been evaluated in terms of changes in structural continuity related to progressive changes in the cracking pattern. Since 1955, with substantial areas being resurfaced, it is no longer feasible to rely completely on cracking to serve as a measure of pavement performance. Periodic aerial photographs may still be useful in the case of paved areas not yet resurfaced, but it will no longer be possible to obtain a reliable measure of the cracking pattern in the concrete pavement in the resurfaced areas. A measure of reflected cracking is not a reliable substitute nor would it be practicable to conduct surveys of reflected cracking in the large areas involved.

During the past four years, the Michigan Pavement Performance Study has been devoted to the development of accurately recorded pavement profiles and a roughness index in inches of vertical displacement per mile as a measure of pavement performance. The roughness index, reflecting progressive changes in the pavement profile, has shown considerable promise as a measure of pavement performance; it is planned to make increasing use of this procedure in evaluating the performance of Willow Run paving. While riding quality in itself is not as vital in airports as it is in highways, it has been shown that roughness and structural continuity are related in such a way that either may be useful in pavement evaluation when the other is not readily available.

The headquarters of the Michigan Pavement Performance Study is at Willow Run and the airfield pavement is constantly being used as a testing ground for development and calibration of profiling equipment. Consequently, some data are already available on pavement roughness; it is hoped that procedures developed for highways can eventually be extended to cover the entire airfield.

Test runs with the truck profilometer on Runway 9L-27R show that the roughness indices on the two outside lanes, which have not been resurfaced, range from 234 to 315, with an average of 267 inches per mile, which is rated as extremely rough. These outside lanes have practically no wheel load application, so this roughness is caused by frost displacement and temperature differentials in the annual cycles of freezing and thawing. Lack of surface drainage from the edge of runways was one of the deficiencies in the original construction, resulting in the edge lane being subjected to severe frost action. The extreme roughness which has resulted is consistent with highway roughness values for comparable conditions.

The four center lanes of the same runway, which were resurfaced in 1959, had roughness indices in 1961 ranging 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 showed roughness indices in 1961 varying from 78 to 88, with an average of 83 inches per mile, which would also be rated good. The center lanes of the runway, and particularly the center taxiway, are subject to heavy load repetitions; and, while the traffic volume cannot be compared to highway traffic, the magnitude of loads is considerably greater.

In this connection, a brief summary of loading conditions at Willow Run 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 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 capacity by 200 per cent, 20 per cent of the traffic exceeded the rated capacity by 100 per cent, while only approximately 15 per cent of the traffic was equal to or less than the rated capacity. Since 1954, gross plane loads have increased rather than decreased; and, even with reduced commercial traffic, scheduled operations in the last two years still exceed the established capacity criterion.

In spite of the fact that during most of the 15 years of service under commercial airline operations the pavement has been substantially overloaded in terms of both gross plane loads and scheduled operations, it has given an excellent performance. As related in this report, inferior construction of the 1943 pavement is directly accountable for the most serious deterioration, which has been measured in terms of pavement cracking. A relatively small proportion of the 1941 construction has shown the same type of distress, but to a reduced degree, again accounted for by poor construction practice. These defects introduced some deficiency in subgrade support in the critical sections, in spite of the best natural soil conditions that could be found in this area.

While faced with these problems, the record shows and the present condition of the pavement confirms that the entire landing area has been maintained at all times in a good and serviceable condition that has assured its efficient operation. In the writer's opinion, there are two major factors in this good record. In the first place, this performance would not have been possible except for the superior soil conditions which provided unlimited subgrade support, requiring only that it be effectively utilized. The second contributing factor was the alert airport management, whose maintenance and betterment program followed the old adage that, "A stitch in time saves nine." This program has been kept consistently ahead of pavement deterioration and has anticipated difficulties before they developed.

When it appeared that routine maintenance would soon be excessive, salvaging of the deteriorating concrete pavement by bituminous resurfacing was undertaken before the situation got out of hand.

CONCLUSION

Inasmuch as the performance of this bituminous resurfacing is the primary subject of this paper, the discussion may be concluded by summarizing the quantitative data available to measure this performance in terms of reflected cracking. Table 2 is a summary of reflected cracking which may be used for reference in connection with these final conclusions.

1. Excessive cracking in portions of the 6 inch plain concrete pavement at Willow Run can be directly related to poor construction practice. The well-constructed pavement is still in good condition, in terms of structural continuity, after 20 years service, 15 of which involve commercial airline operation with loading far in excess of its rated capacity. This outstanding performance is primarily due to superior subgrade support provided by the natural soil conditions existing at the site and a timely maintenance program.
2. In the areas in which there has been excessive cracking of the concrete pavement, bituminous resurfacing has provided an effective means of insuring efficient service and reduced maintenance. In these areas, joint and crack filling had become prohibitive in cost and relatively ineffective as a means of protection from further disintegration.
3. The major source of reflected cracking is at the joints of the concrete pavement, except where slab cracking has reached such an advanced stage that the concrete pavement has been reduced to a series of separated concrete blocks.
4. The use of welded wire fabric as steel reinforcing in the bituminous mixture was of substantial benefit in reducing direct crack reflection. During the period of service reported herein and conditioned by other maintenance of the test areas, the percentage of reduction in direct reflection of cracks in reinforced areas is, in round figures, 10 to 15 per cent of the total lineal feet of joints and cracks in the underlying concrete pavement, or 30 to 40 per cent of the reflected cracking in unreinforced areas.
5. One of the phenomena noted in the use of welded wire fabric at Willow Run was the creation of a new pattern of cracks described in this paper as translated cracks, in which wider crack openings such as joints are distributed by the reinforcing over the area covered in a larger number of finer cracks. Total cracking percentage before sealing, in terms of total lineal feet of joints and cracks, including both direct and translated cracks, was just as great and in some

cases greater than the reflected cracking in unreinforced areas. However, with the finer cracks, the surface can be more effectively maintained and, after sealing, the decreased percentage of reflected cracking in the reinforced areas is practically the same as in the case of direct crack reflection.

6. In one test area, an increased thickness of 1 inch, or 33 per cent, in the bituminous resurfacing reduced the reflected cracking some 8 per cent in terms of the total lineal feet of joints and cracks, and from 20 to 25 per cent of the reflected cracking in the area of standard 3 inch thickness. The percentage of improvement was the same before and after sealing.
7. A special experiment with a 1 inch wearing surface of Epon asphalt, in addition to providing good protection against spillage of gasoline and oil, showed a substantial decrease in reflected cracking, in comparison with that of a comparable area of conventional bituminous resurfacing.

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
SUMMARY OF REFLECTED CRACKING IN PERCENT

Test Area	Thickness	Without Mesh		With Mesh			Differential*		
		Mesh		Direct	Total		Direct	Total	
		1957	1961		1957	1961		1957	1961
T-1	3"	32	39	--	--	--	--	--	--
	4"	24	31	--	--	--	--	--	--
T-2	3"	32	37	25	27	39	33	7	10
T-3	3"	31	41	19	22	34	28	12	19
A-1	3"	44	47	33	29	38	30	11	18
Average	3"	35	41	26	26	37	30	10	16

* T-1 omitted

TEST AREA A-2

Base Course	Thickness	With Mesh			
		Direct	Total		
			1957	1961	1957
Concrete	3"	32	25	36	27
Aggregate	3"	32	32	35	35
Differential		0	7	-1	8

TEST AREAS A-3 AND A-4

Surface	Thickness	Direct Total		
		Bituminous	Epon Asphalt	Differential
Bituminous	3"	54	54	54
Epon Asphalt	3"	13	13	30
Differential		41	41	24

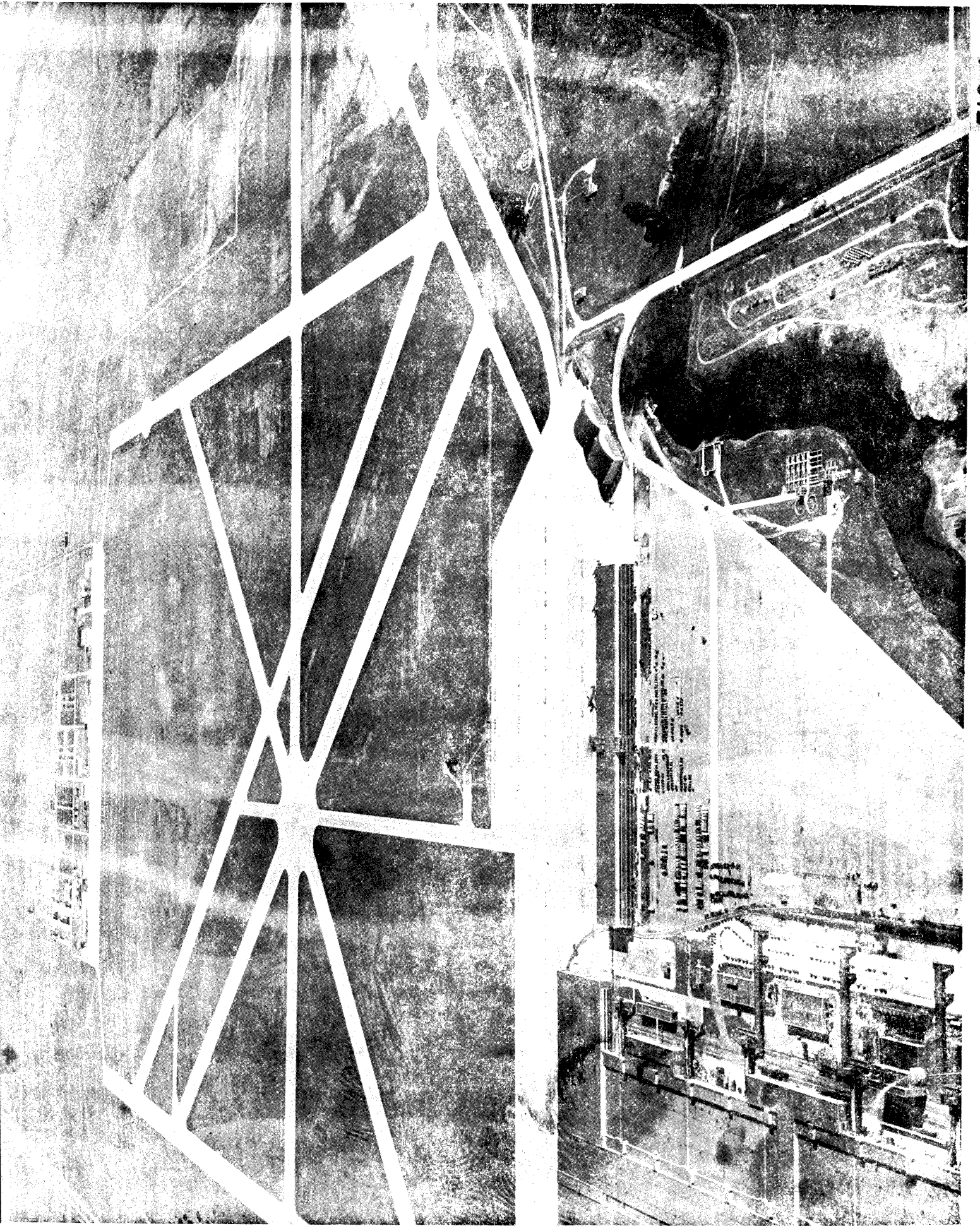


FIG. 1

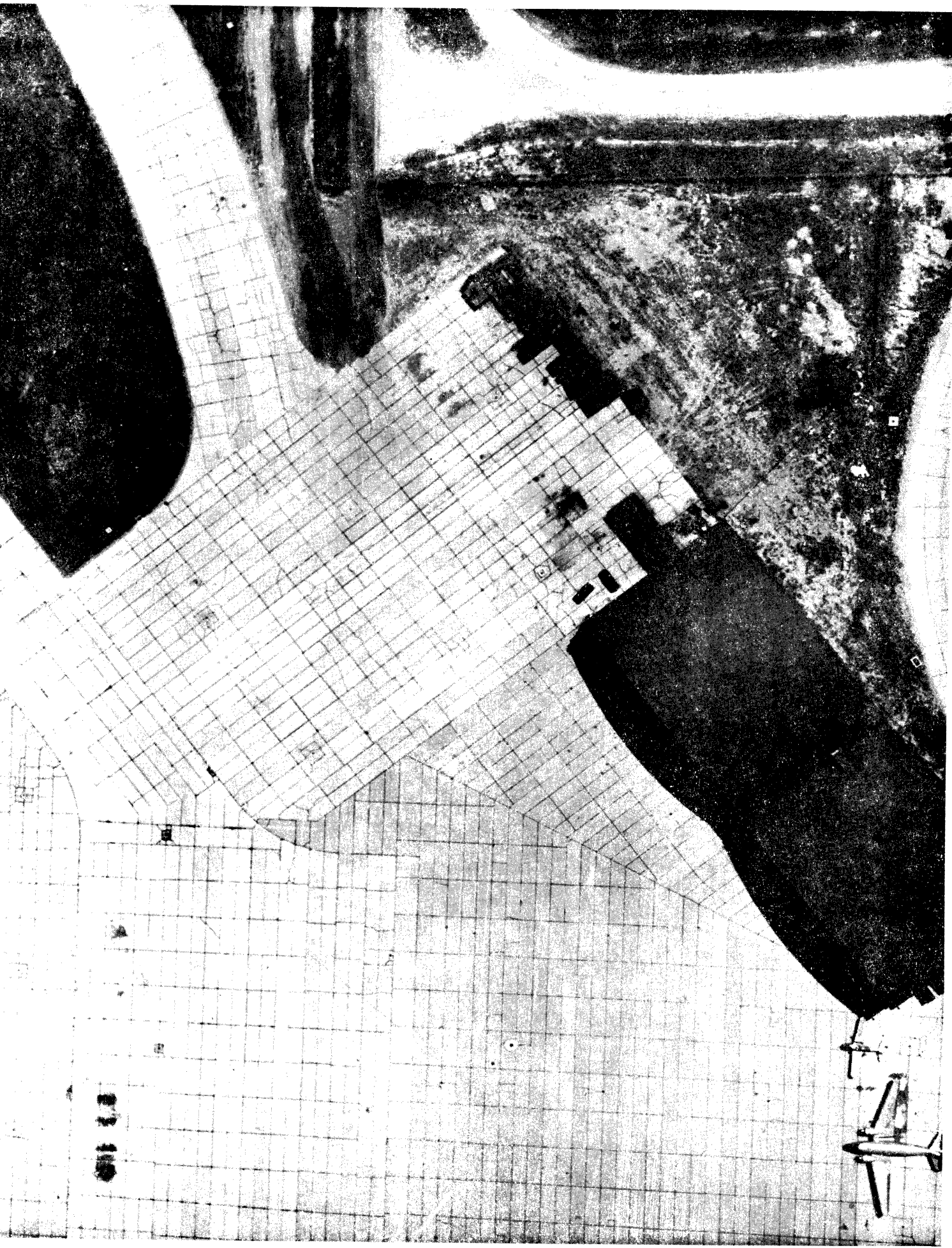


FIG. 2

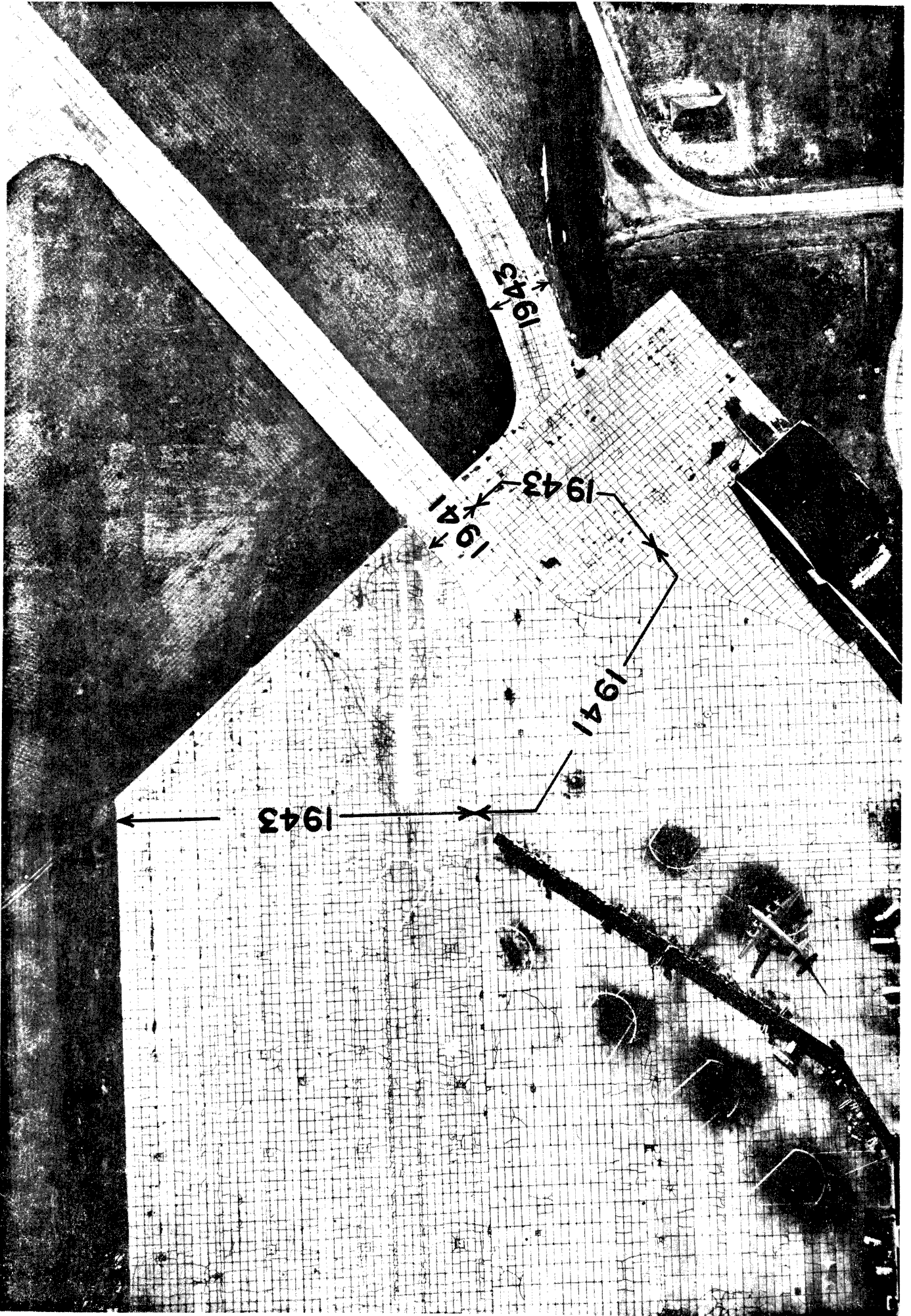


FIG 3

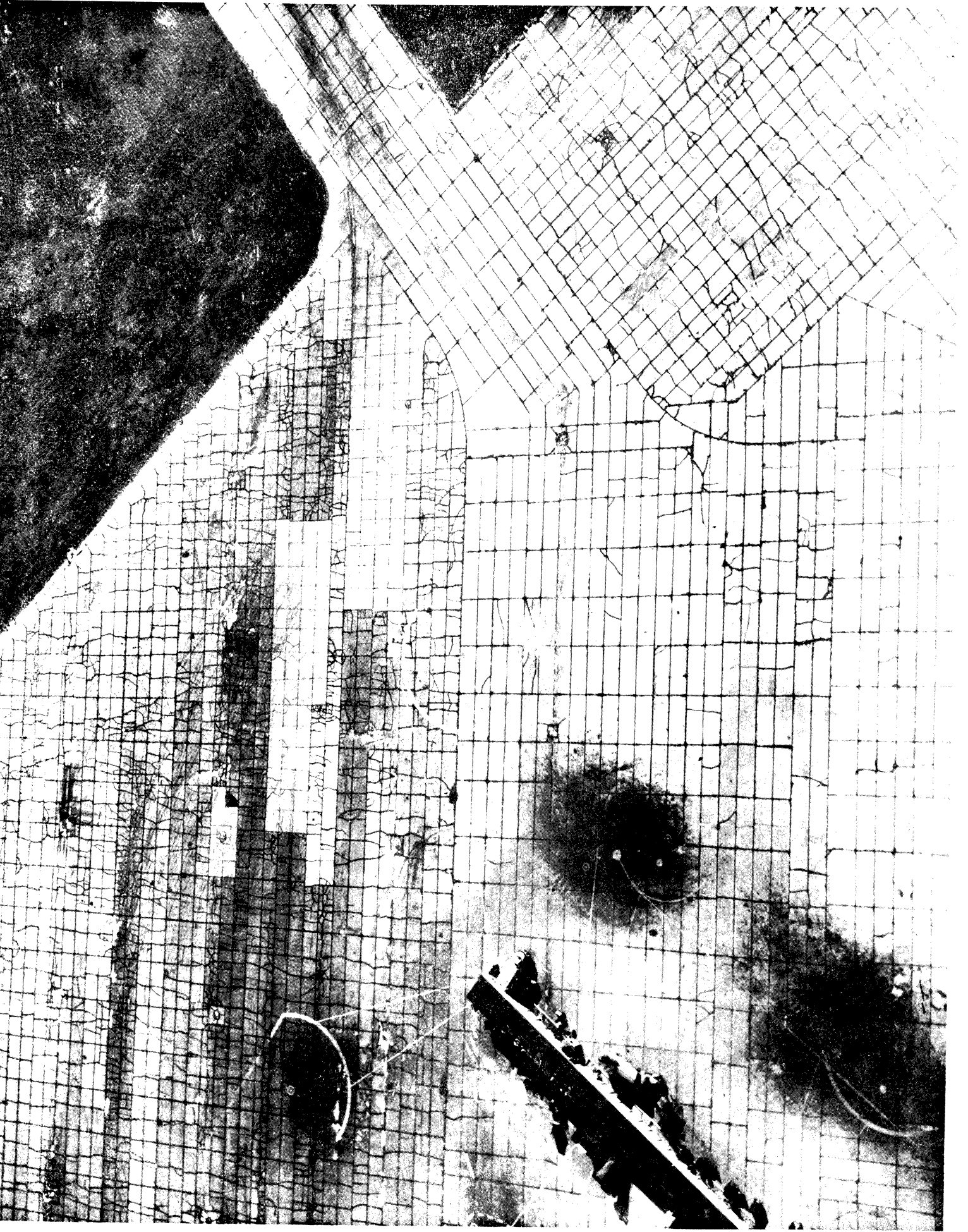
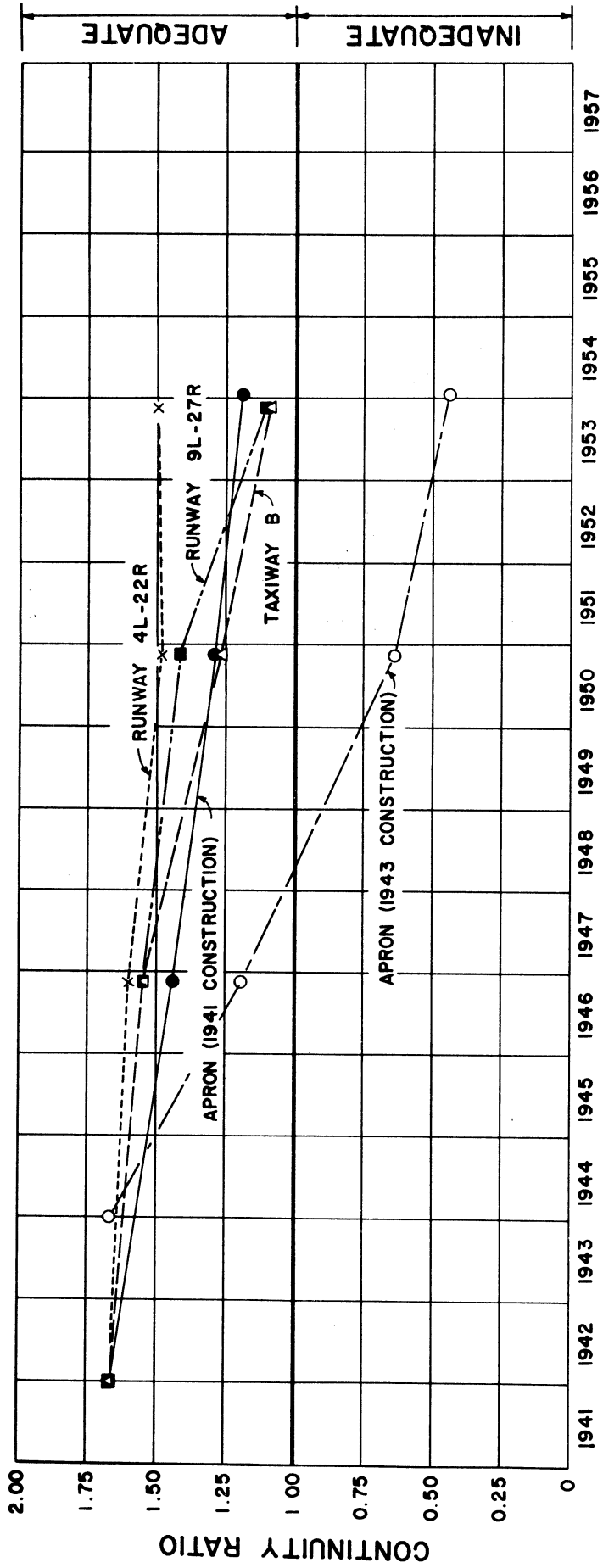


FIG. 4



TIME IN YEARS

FIG. 5

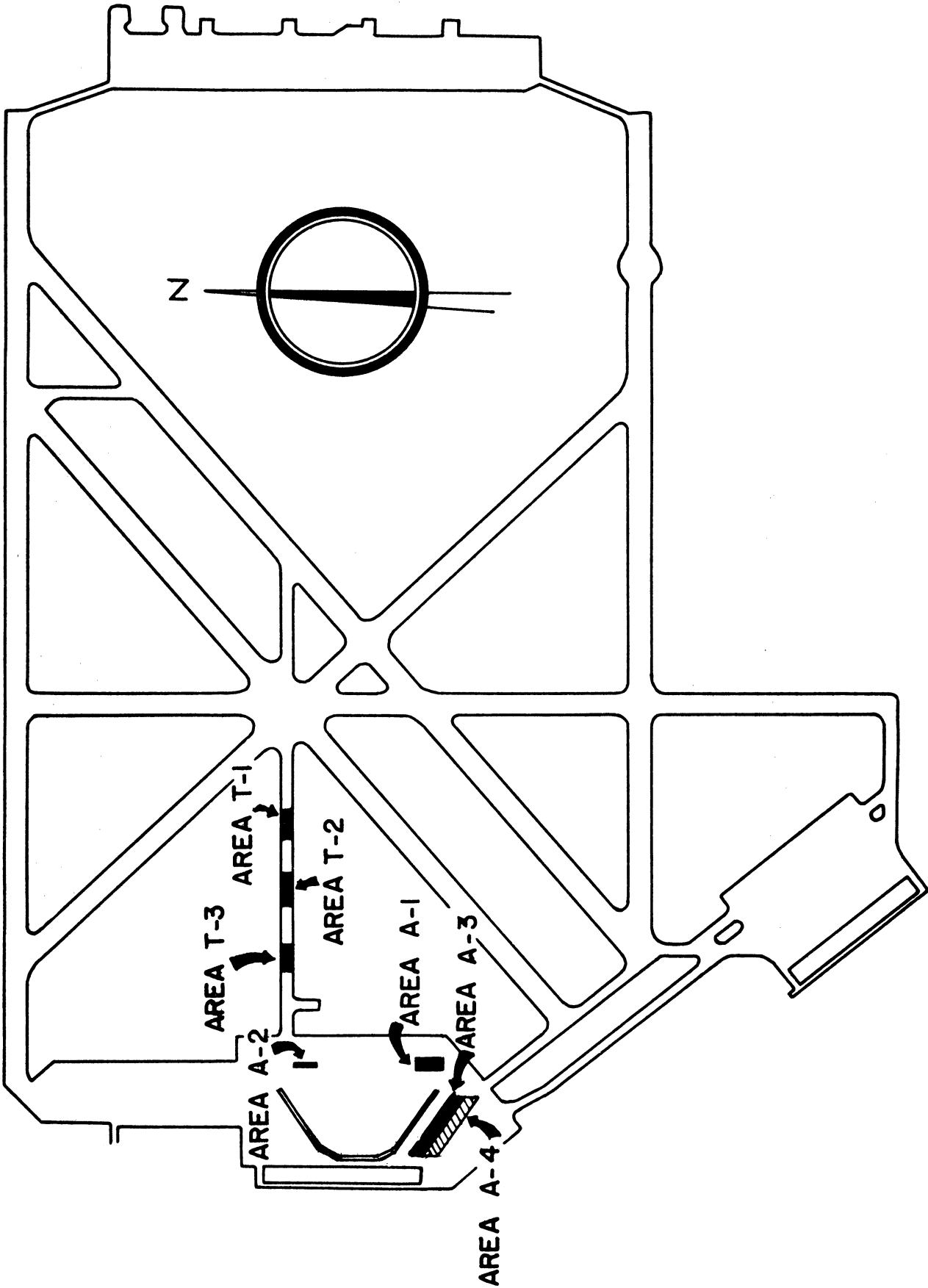


FIG. 6

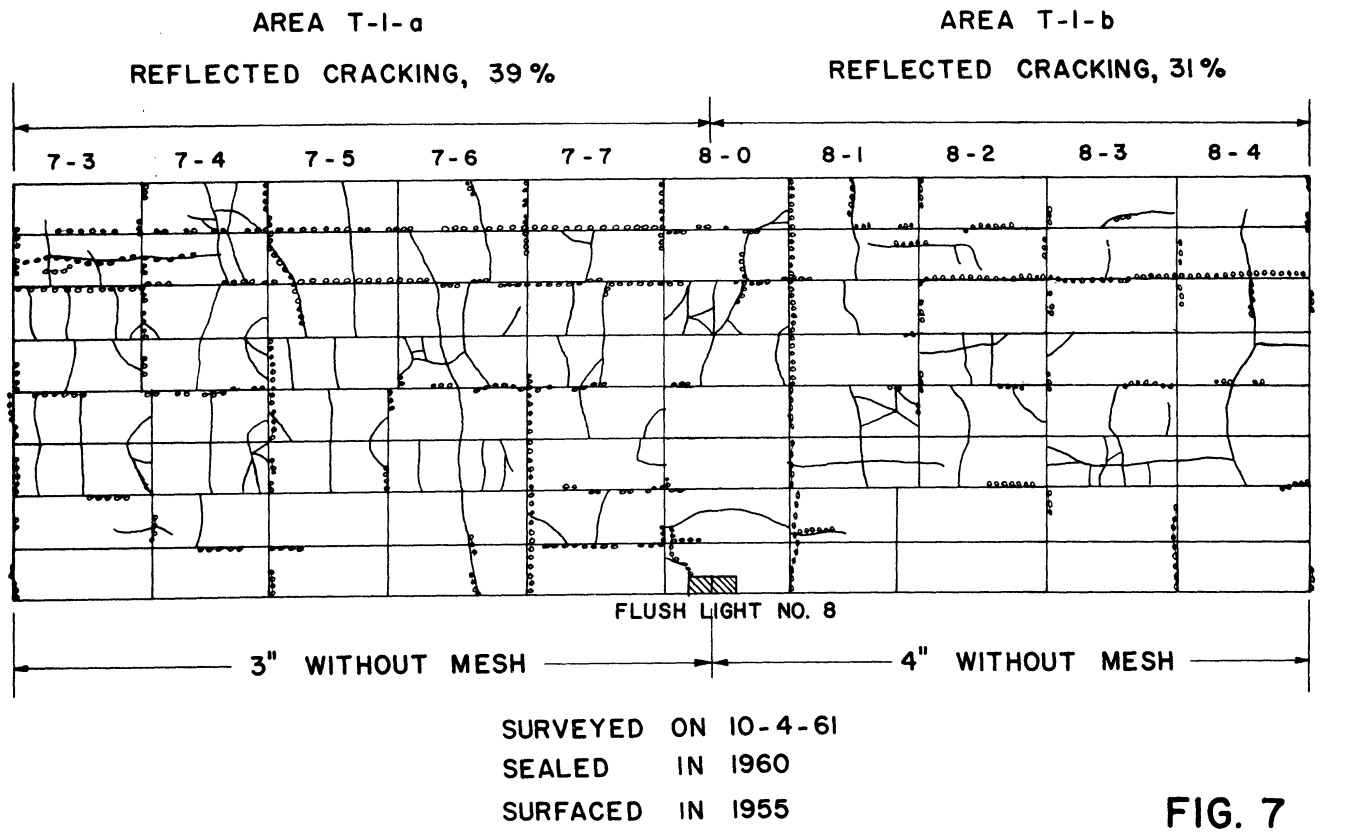
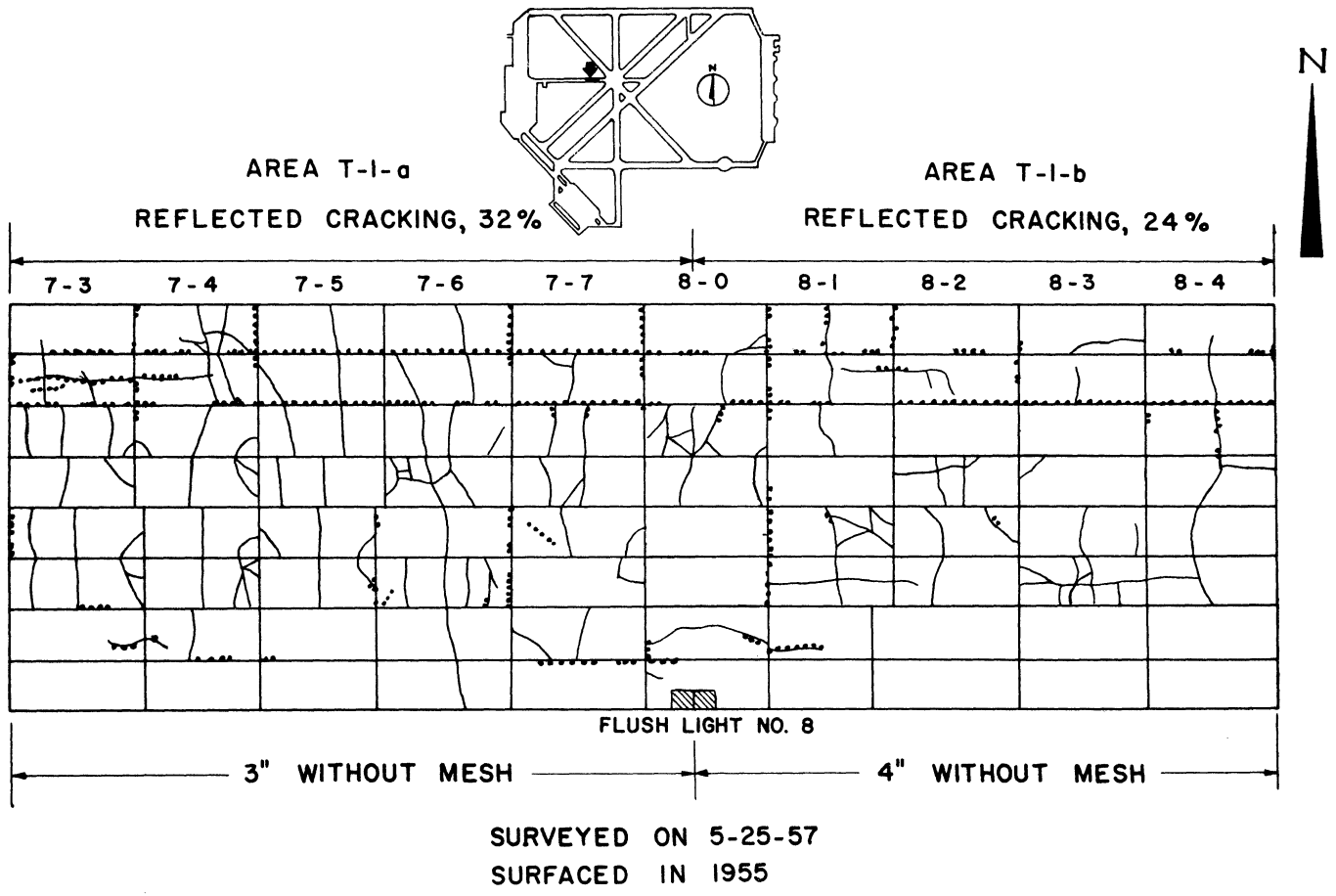


FIG. 7

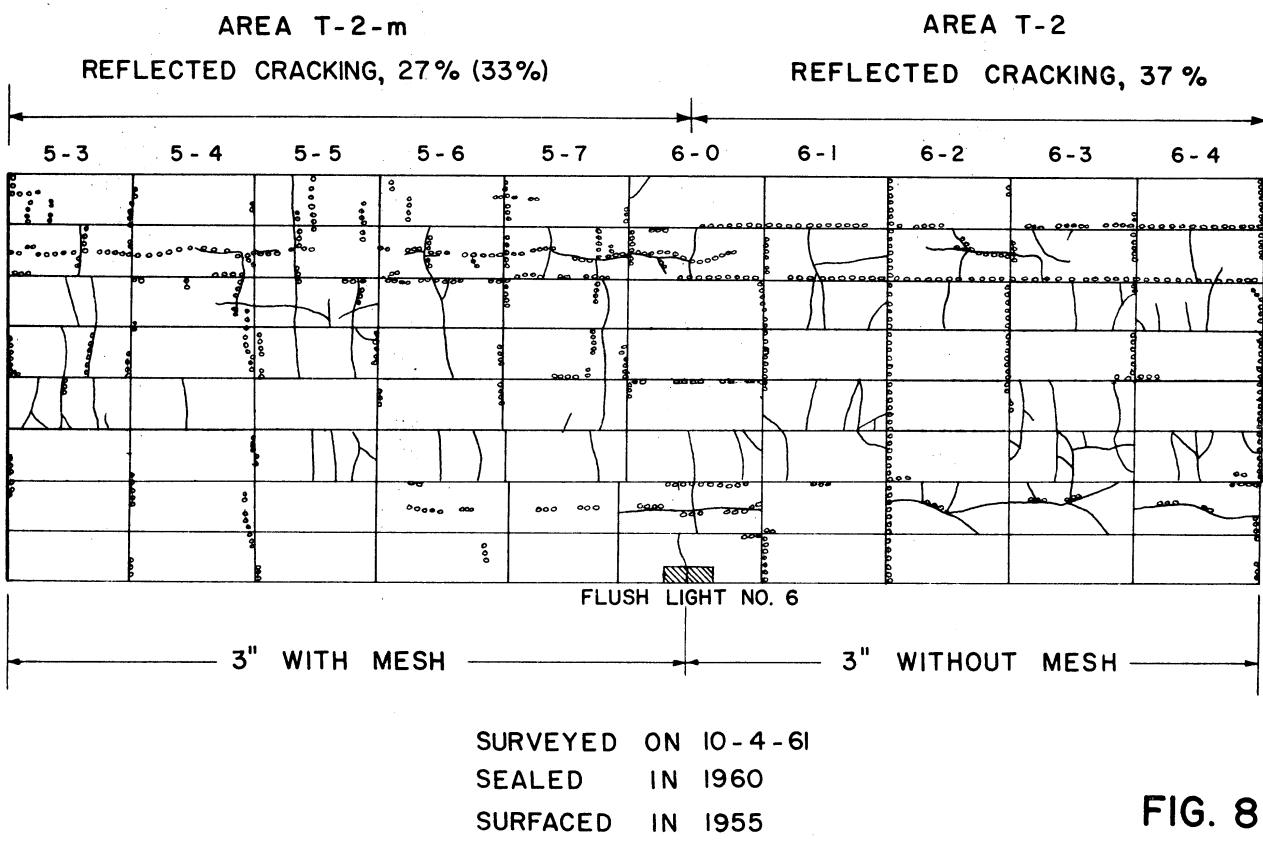
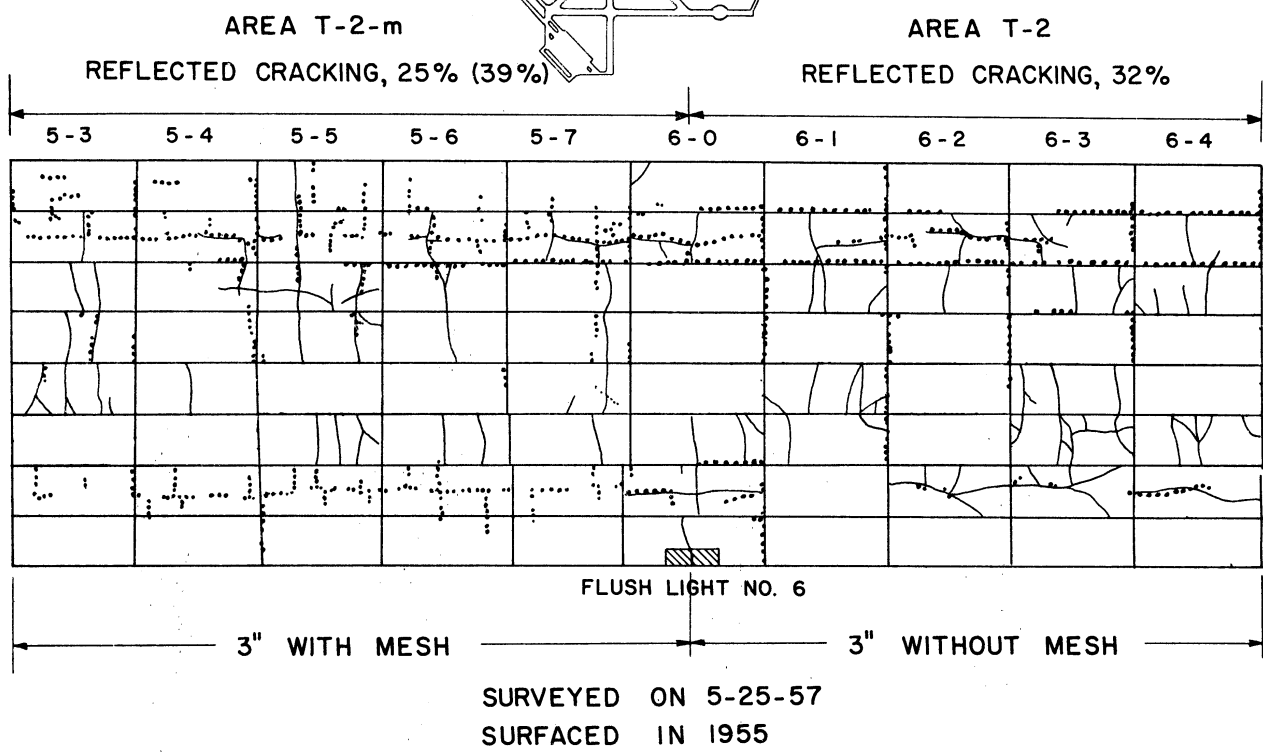
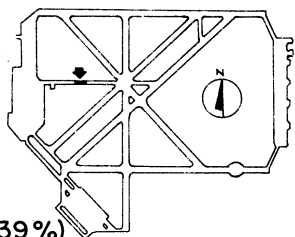
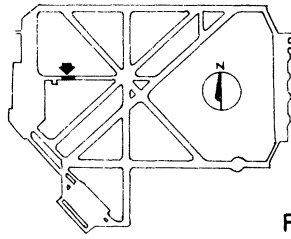


FIG. 8



AREA T-3

REFLECTED CRACKING, 31 %

AREA T-3-m

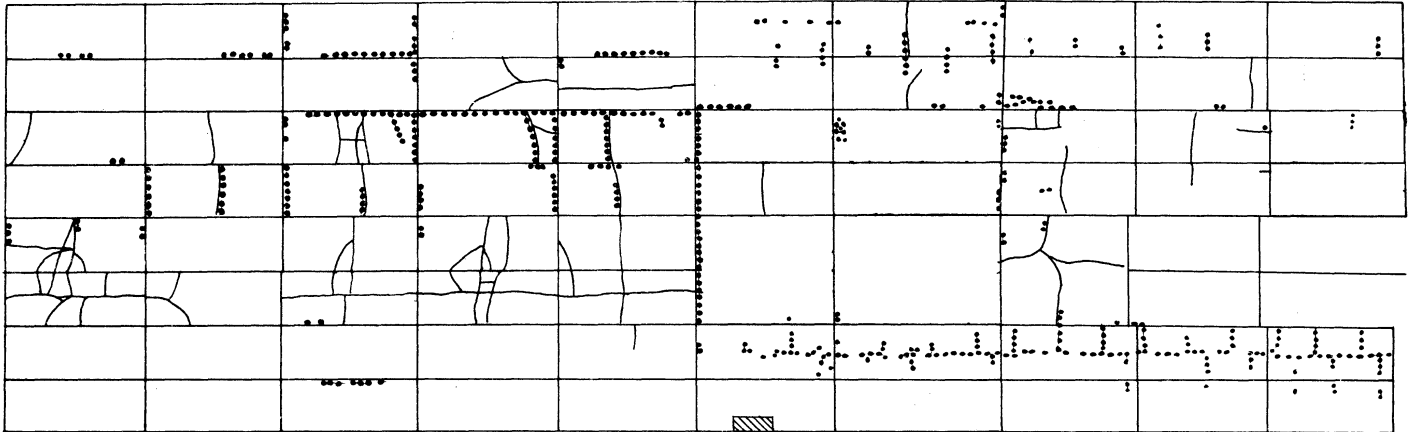
REFLECTED CRACKING, 19 % (34 %)

3" WITHOUT MESH

3" WITH MESH



3-3 3-4 3-5 3-6 3-7 4-0 4-1 4-2 4-3 4-4



FLUSH LIGHT NO. 4

SURVEYED ON 5-25-57

SURFACED IN 1955

AREA T-3

REFLECTED CRACKING, 41 %

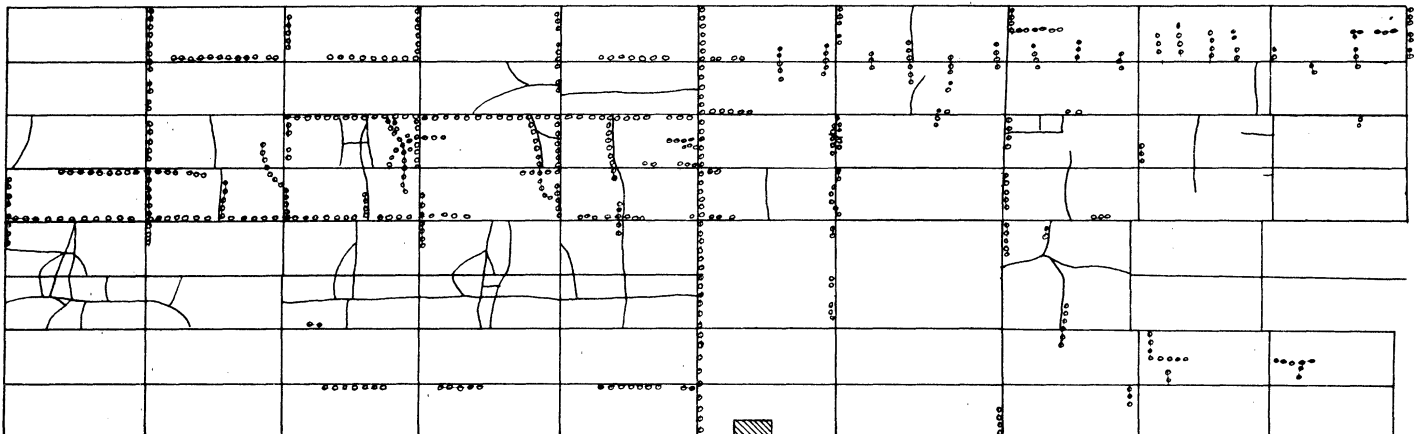
AREA T-3-m

REFLECTED CRACKING, 22 % (28 %)

3" WITHOUT MESH

3" WITH MESH

3-3 3-4 3-5 3-6 3-7 4-0 4-1 4-2 4-3 4-4



FLUSH LIGHT NO. 4

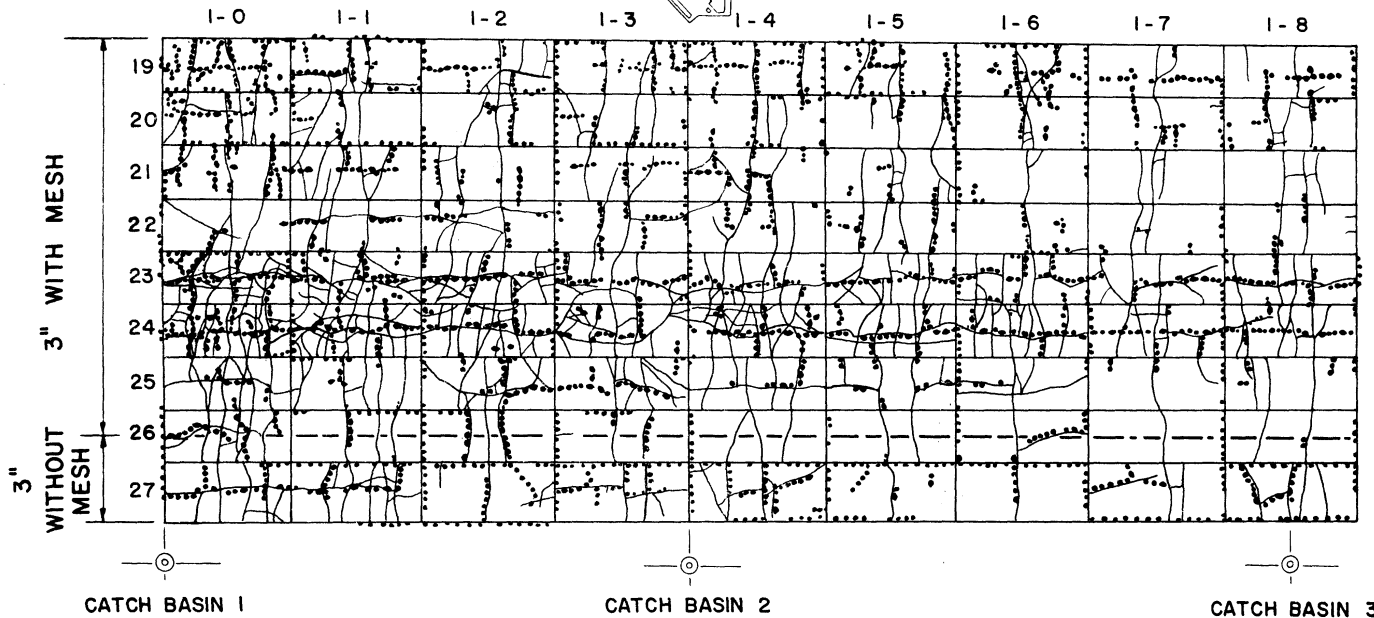
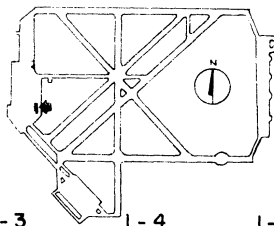
SURVEYED ON 10-4-61

SEALED IN 1960

SURFACED IN 1955

FIG. 9

AREA A-l-m-a
REFLECTED CRACKING, 33% (38%)



CATCH BASIN 1

CATCH BASIN 2

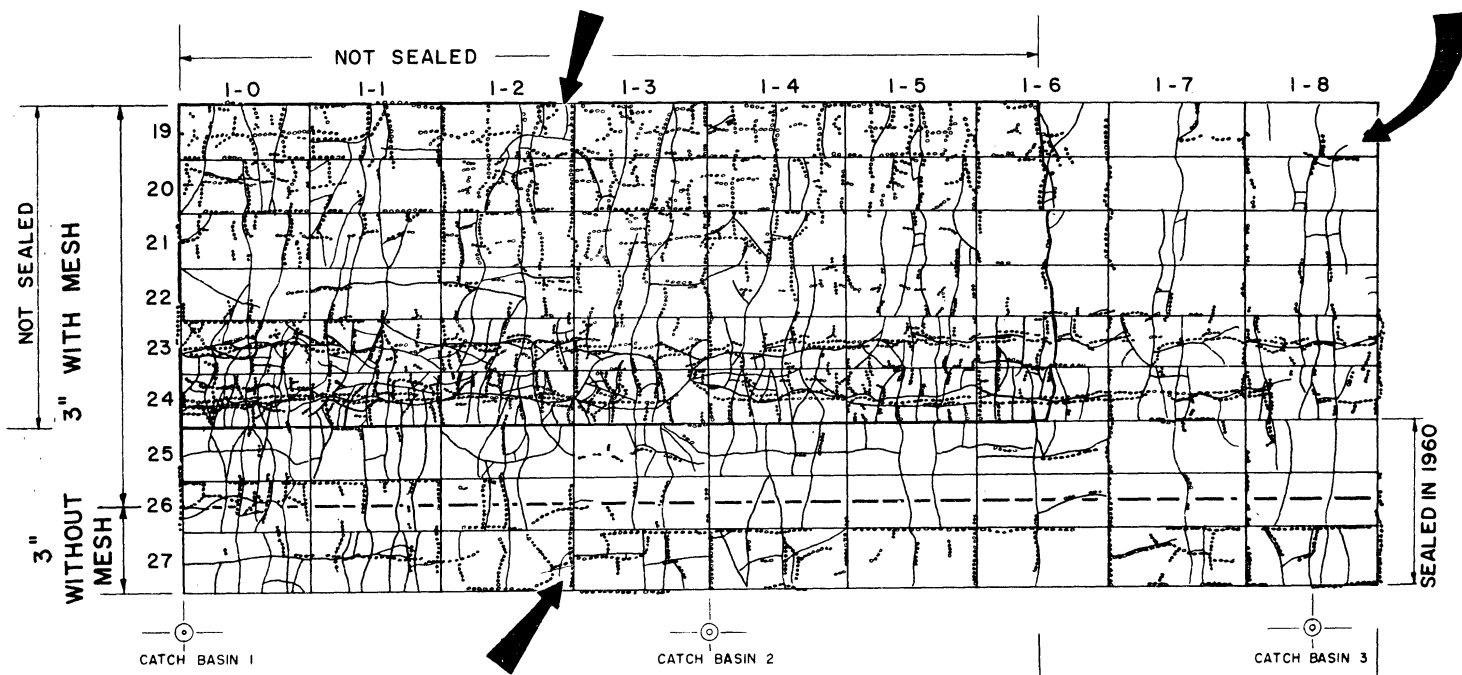
CATCH BASIN 3

AREA A-l-b
REFLECTED CRACKING, 44%

SURVEYED ON 5-25-57
SURFACED IN 1955

AREA A-l-m-d'
REFLECTED CRACKING, 49% (75%)

AREA A-l-m-a''
REFLECTED CRACKING, 29% (30%)



CATCH BASIN 1

CATCH BASIN 2

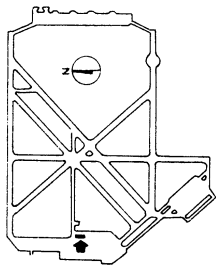
CATCH BASIN 3

AREA A-l-b
REFLECTED CRACKING, 47%

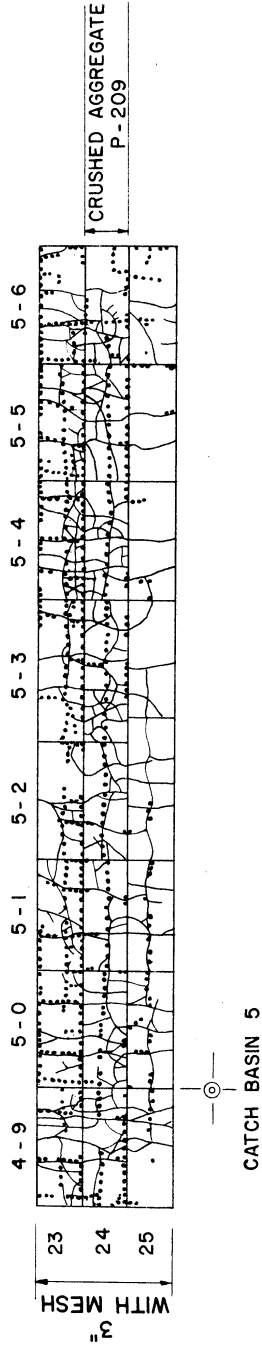
SURVEYED ON 10-4-61
SEALED IN 1960
SURFACED IN 1955

SEALED IN 1960

FIG. 10

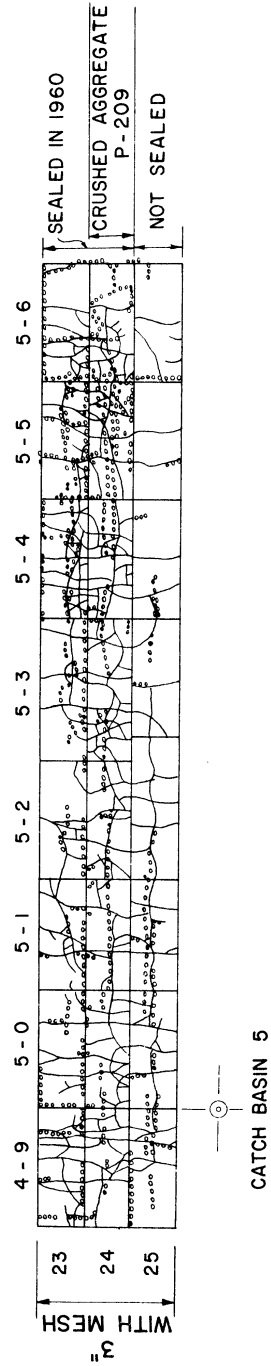


AREA A-2
 REFLECTED CRACKING
 LANE 23, 46% (51%)
 LANE 24, 32% (35%)
 LANE 25, 14% (17%)



SURVEYED ON 5-25-57
 SURFACED IN 1955

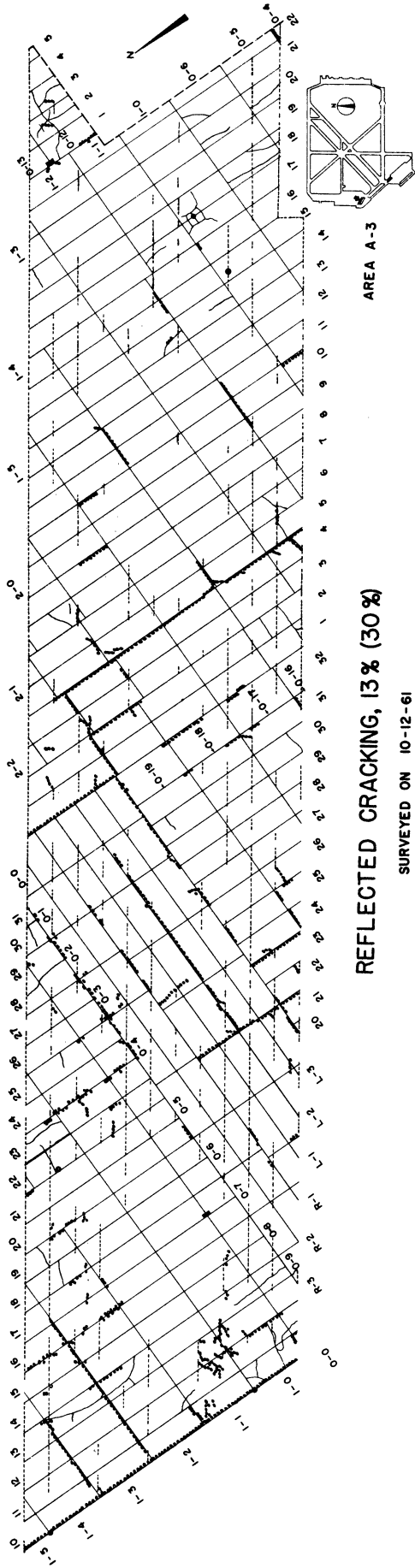
AREA A-2
 REFLECTED CRACKING
 LANE 23, 30% (32%)
 LANE 24, 32% (35%)
 LANE 25, 19% (22%)



SURVEYED ON 10-4-61
 SEALED IN 1960
 SURFACED IN 1955

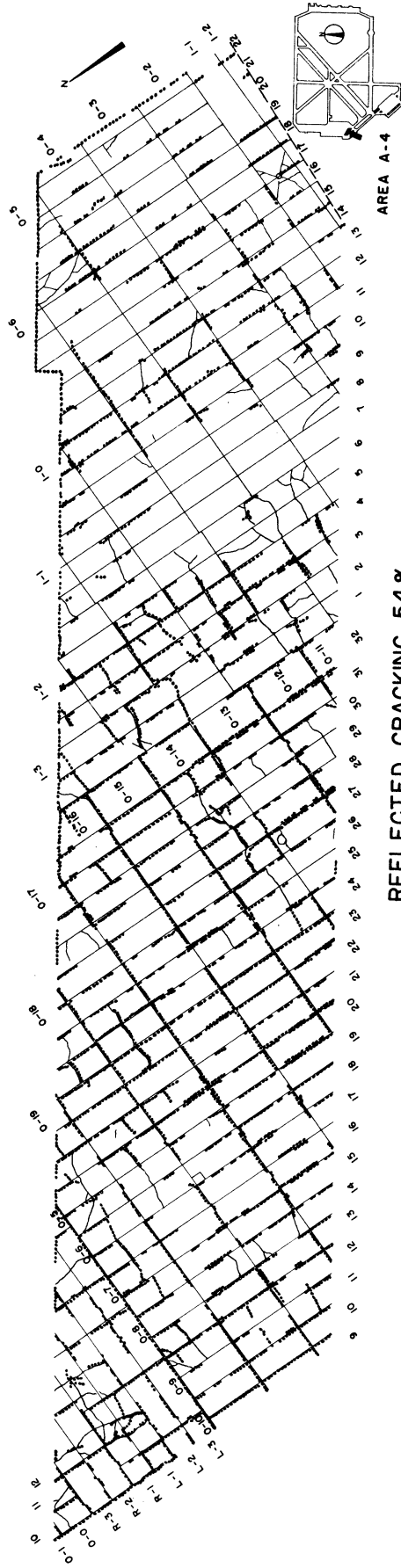
FIG. 11

SOUTH CONCOURSE



REFLECTED CRACKING, 13% (30%)

SURVEYED ON 10-12-61
EPON ASPHALTIC CONCRETE PLACED IN 1960



REFLECTED CRACKING, 54%

SURVEYED ON 10-12-61
SURFACED IN 1960

FIG. 12

DEPARTMENT OF CIVIL ENGINEERING

THE UNIVERSITY OF MICHIGAN

ANN ARBOR, MICHIGAN

THE MICHIGAN PAVEMENT PERFORMANCE STUDY
FOR DESIGN CONTROL AND SERVICEABILITY RATING

BY

WILLIAM S. HOUSEL
Professor of Civil Engineering

PROJECT OF THE
MICHIGAN HIGHWAY PLANNING SURVEY

WORK PROGRAM HPS-1 (27)

IN COOPERATION WITH
U. S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

AUGUST, 1962

ACKNOWLEDGMENT

During the past fifteen years that Willow Run Airfield has been used as a field laboratory for studying pavement design and performance, the University of Michigan has had the support and cooperation of a number of agencies who have contributed to that program. These include: the Federal Aviation Agency; Michigan Department of Aeronautics; Airlines National Terminal Service Company, Inc.; the Wire Reinforcement Institute; Michigan Trucking Association, American Trucking Associations, Inc., and Automobile Manufacturers Association; and, present sponsors of the Michigan Pavement Performance Study, now being conducted as part of the Highway Planning Survey Work Program in cooperation with the Bureau of Public Roads. The individual services of the staff of the Soil Mechanics Laboratory at the University of Michigan and of the Soil and Paving Unit at Willow Run are gratefully acknowledged. Without their enthusiastic and painstaking efforts, the project would not have been possible. Only their number prevents the individual acknowledgments which their contributions deserve.

THE MICHIGAN PAVEMENT PERFORMANCE STUDY
FOR DESIGN CONTROL AND SERVICEABILITY RATING

By

William S. Housel*

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 Highway Department in cooperation with the U. S. Bureau of Public Roads. This study had its beginning in 1946, immediately after the second World War, when the interest in highway design and performance was stimulated by the greatly increased demand on the highway transportation system and a nation-wide acceleration 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 careful 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 debate, 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 incorporated 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 answers to these questions can be found in the behavior of roads that have been subjected to these conditions over many years. Well organized pavement condition surveys in Michigan date back to the middle twenties, when the late V. R. Burton organized a group of research workers who started a series of state-wide pavement condition surveys, including comprehensive data on soil conditions and climatic environment.^{1,2,3} This work has been carried on over the years by a number of individuals, including Kellogg, Benkelman, Stokstad, and Olmstead, whose names have become well known among highway engineers and soil scientists.

These investigators early found a significant correlation between pavement performance and environment, including soil type, drainage, and climatic factors -- a viewpoint which has continued to exert a dominant influence on pavement design in Michigan. Improvements in this approach to pavement design have led to more accurate evaluation of soil conditions, drainage, and climatic environment, the utilization of local soil materials of favorable characteristics, and the selection of

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Research Consultant - Michigan State Highway Department

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

cracking, with virtually no cracks having developed 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 already 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 incipient cracking was traced to the fact that this section of the apron was poured in alternate 20 foot widths, and the concrete mixer was permitted to travel on the recently completed 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 concentrated load of the mixer at the edge of the slab without damage. As a result, these frequently accepted compromises in paving practice 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 illustrate the development and application of performance criteria, it is desired first to emphasize that the primary objectives in evaluating pavement performance are to check design procedures by determining those factors

responsible for good performance, and to isolate 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 establish 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 condition surveys and general observation of highways in service, it had been concluded that pavement performance or serviceability could be measured in its major practical aspect 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 continuity, derived from the cracking pattern, reflects 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 structural 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 pavement 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 translating 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 ($25 \div 15 = 1.67$). 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

of the traffic was equal to or less than the rated maximum loading. Since 1954, gross plane loads have increased rather than decreased; 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 construction 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 discussion. 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 conjunction with excellent subsurface drainage. However, in the north-central portion of the airfield was a lowlying area of virgin hardwood with a heavy accumulation of forest debris and organic material, and a water table close to the surface. Subdrainage was provided to lower the water table beyond the normal depth of frost penetration, but it proved difficult under the emergency construction conditions then in effect to enforce effective control of the grading operation and selective placement of subgrade soil. Consequently, 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 operations started in 1946. By 1950, it became necessary to start some slab replacement in the critical area, an operation which continued periodically for the next five years, until 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 between pavement performance and design. This is one of the weaknesses of statistics resulting from the use of averages of one type or another to obtain a broad correlation between design factors. In such averages, the extremes of behavior or abnormal results are averaged out or discarded, when actually these radical departures from the norm frequently provide the most significant and useful 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 manipulation.

Further analysis of the continuity ratios shown in Figure 3 provides an excellent example of the necessity for developing a procedure which goes beyond the statistics and can be followed, step by step, back to the behavior of individual slabs, if necessary to identify the basic design deficiency.

In Figure 4 are shown the continuity ratios 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

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

construction. Runway 9L-27R has been subjected 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, except 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 observation 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 reasonably good condition, being rated as adequate, with a continuity ratio greater than unity. However, this pavement, with no known design or construction deficiencies, reflects the influence of more severe loading conditions 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 performance 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 average 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 average 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 concentration of load application on a pavement whose lack of supporting capacity has led to

its reduction to a block pavement. The controlling factors were excessive subgrade deflection in combination with a weak unreinforced concrete pavement, which was still too rigid to mobilize the available subgrade support until both it and the subgrade had become 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 flexibility combined with adequate subgrade support. The results of a pavement condition survey made in October, 1961, have been reported 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 controlled 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 perceptible rutting or displacement of the bituminous surface course.

As previously indicated, the continuity ratio can no longer be used to evaluate performance of the resurfaced pavement; therefore, changes in the pavement profile, in terms of vertical displacement or roughness, have been adopted as a basic measure, supplemented by observations of the surface condition 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 temperature differentials in the annual cycles of freezing and thawing. Lack of adequate surface drainage at the edge of the runways was one of the deficiencies in the original construction, 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, particularly, the center taxiway are subject to heavy load repetition; while the traffic volume cannot be compared to highway traffic, the magnitude of the loads is considerably greater.

The performance of the resurfaced pavements at Willow Run provides evidence to support 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 depending 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 subgrade has proved to be sufficient to carry the heavy load concentrations applied without damaging 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 mobilize 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 designed.

WIRE REINFORCEMENT INSTITUTE PROJECT

The next opportunity to test the continuity ratio and roughness index as measures of pavement performance came on a cooperative research project sponsored by the Wire Reinforcement 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 concrete pavements under actual service conditions. Pavement performance surveys were conducted on selected projects in southern Michigan which had been in service for periods varying from 10 to 25 years. Each project included comparable reinforced and unreinforced sections which were observed over a period of approximately five years, from 1952 to 1956. Aerial photographs, supplemented by field surveys, 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 pavements in actual service could be measured in realistic terms by carefully conducted pavement 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 condition 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

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

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

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 re-tread.

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

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

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

in service and have been included in the profile 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 suffered a cumulative or continuous increase in roughness at an average rate of between 4 and 5 inches per mile per year. When it is considered that Class 1 pavements are those that have been rated as adequate or more than adequate 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 environmental factors, mainly associated with seasonal cycles and fluctuations of moisture and temperature 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 showing 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 superior 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 illuminating examples will be presented for illustration.

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 classification of Plainfield sand is correct for the project showing superior performance but incorrect for the project which has become extremely 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 pavement 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 investigation of this section revealed that a storm sewer had been laid along the edge of the pavement; backfill settlement undoubtedly produced the high roughness index.

Class 1 Recapped Concrete Pavement

The roughness index data shown in Figure 18 have been taken from 118 construction contracts 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 resurfacing 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 intersections 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 interpreted as significant would hardly be justified 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 themselves, 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 behavior and have been identified by contract number for further investigation of the abnormal behavior which they have shown. While complete investigation including field checking is still in progress, there is some significant 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 following 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 resurfaced concrete pavements, it should be pointed out that, in Figure 18, the years in service are figured from the date of the last resurfacing. 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 significant relationships have been brought out by this study, but space will not permit presentation 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 determined 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 boundaries 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 pavements in Michigan were thought of as low cost road construction. As a consequence, both design and construction requirements were much less exacting than those practiced in recent years. Consequently, a wider range of behavior 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

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

\overline{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

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 or 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 four 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 for 395 test sections in all four test loops: 96 of reinforced concrete, 240 feet in length; 96 of unreinforced concrete, 120 feet in length; and, some 203 sections of asphalt pavement, 100 feet in length. 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 (\bar{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 and 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 presents 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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APPENDIX A

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}

$$\begin{aligned}
 (\Sigma Y)^2: \quad RWP &= 678,976 \times 10^{-6} \\
 &LWP = 770,884 \times 10^{-6} \\
 \text{Average } (\Sigma Y)^2 &= 724,930 \times 10^{-6}
 \end{aligned}$$

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

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

Cracking and Patching (C + P)

$$C + P = \frac{568}{5} = 113$$

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

Rutting Depth (\overline{RD})

$$\overline{RD} = 0.261$$

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

Present Serviceability Index (PSI)

$$\begin{aligned}
 \text{PSI} &= 5.03 - 1.91 \times 0.959 - 0.01 \times 10.6 - 1.38 \times 0.068 \\
 &= 5.03 - 1.83 - 0.11 - 0.09 \\
 &= 3.00
 \end{aligned}$$

CONVERTING ROUGHNESS INDEX TO SERVICEABILITY INDEX

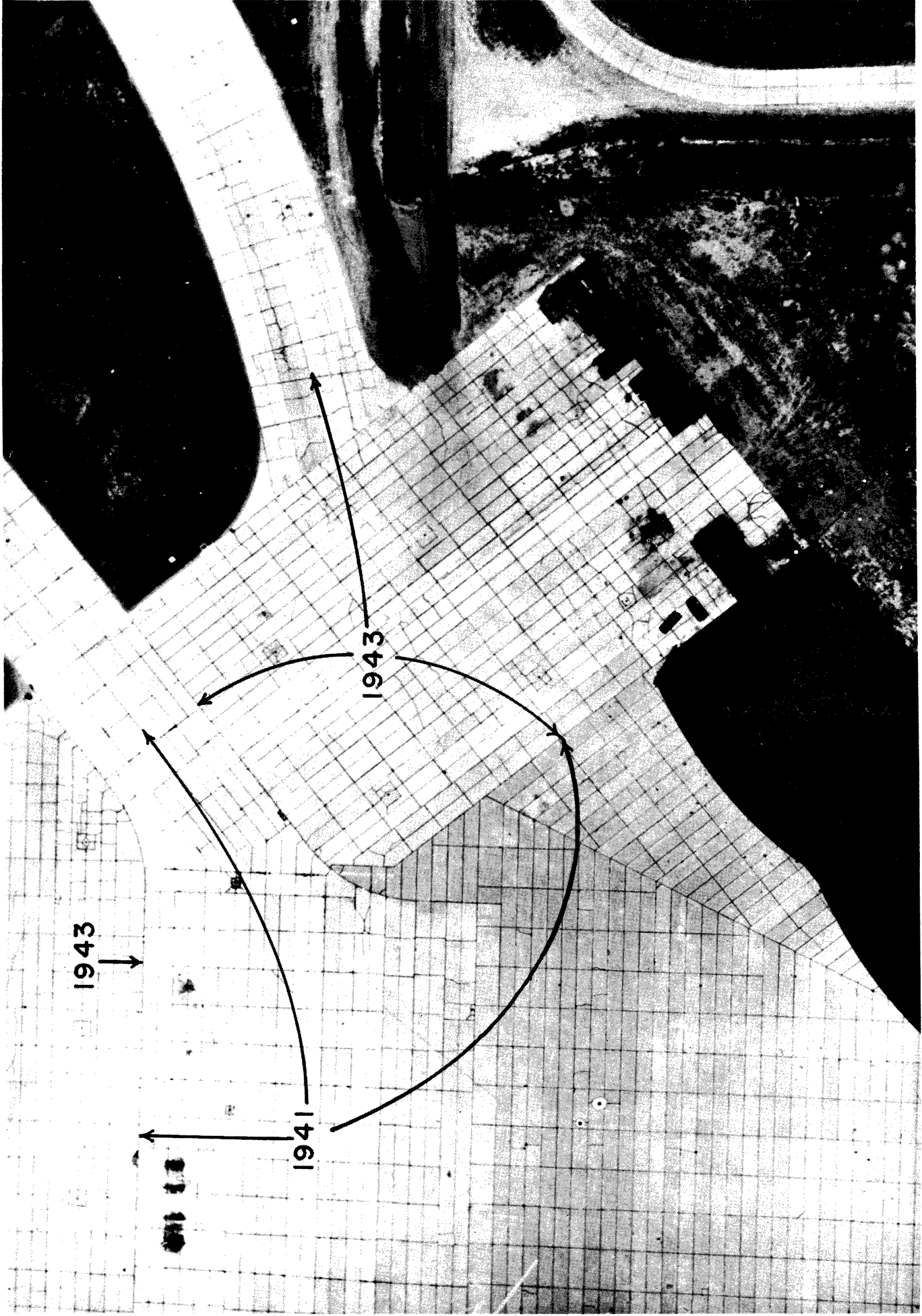
$$\text{RI} = 57 \sqrt{\overline{SV}}$$

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

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

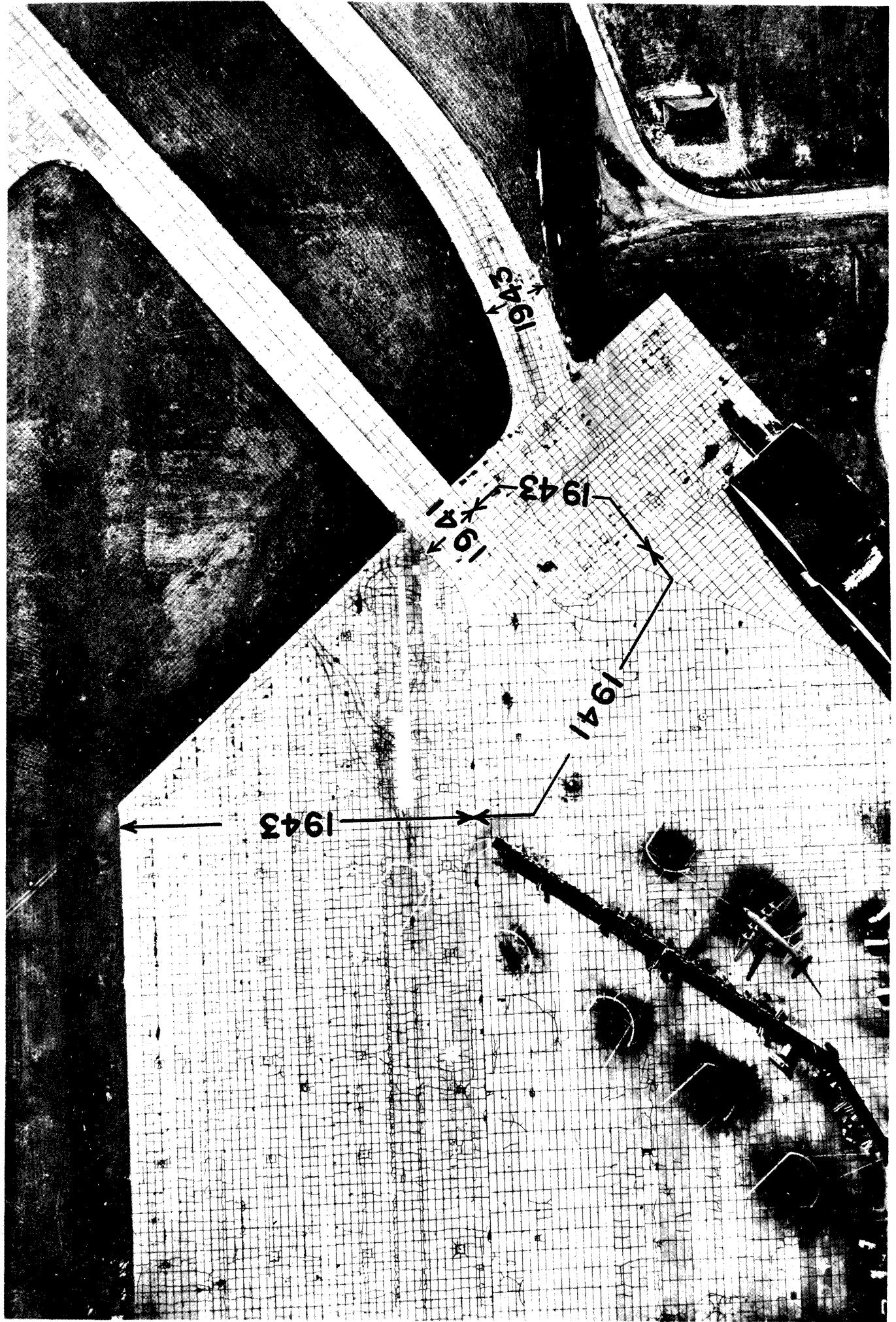
$$\text{Average } \overline{SV} = 7.44$$

$$\begin{aligned}
 \text{PSI} &= 5.03 - 1.91 \log (1 + 7.44) - 0.01 \sqrt{113} - 1.38 \times 0.261^2 \\
 &= 5.03 - 1.77 - 0.11 - 0.09 \\
 &= 3.06
 \end{aligned}$$



SOUTH END OF MAIN WEST APRON - 1946
WILLOW RUN AIRFIELD

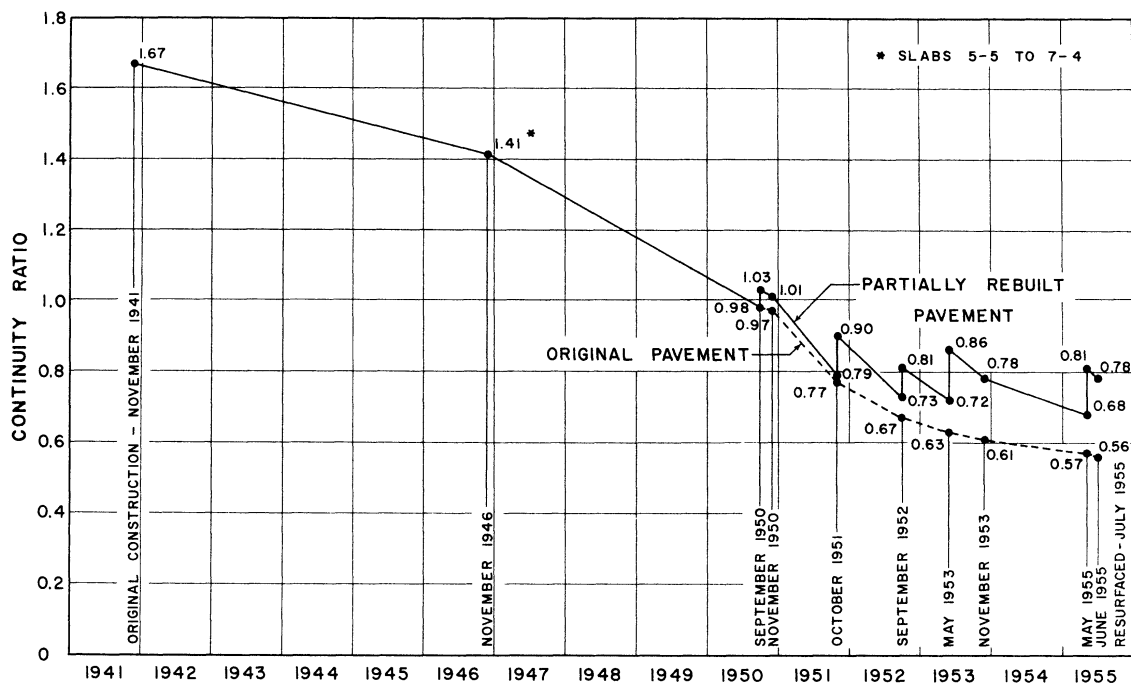
FIG. 1



SOUTH END OF MAIN WEST APRON - 1950

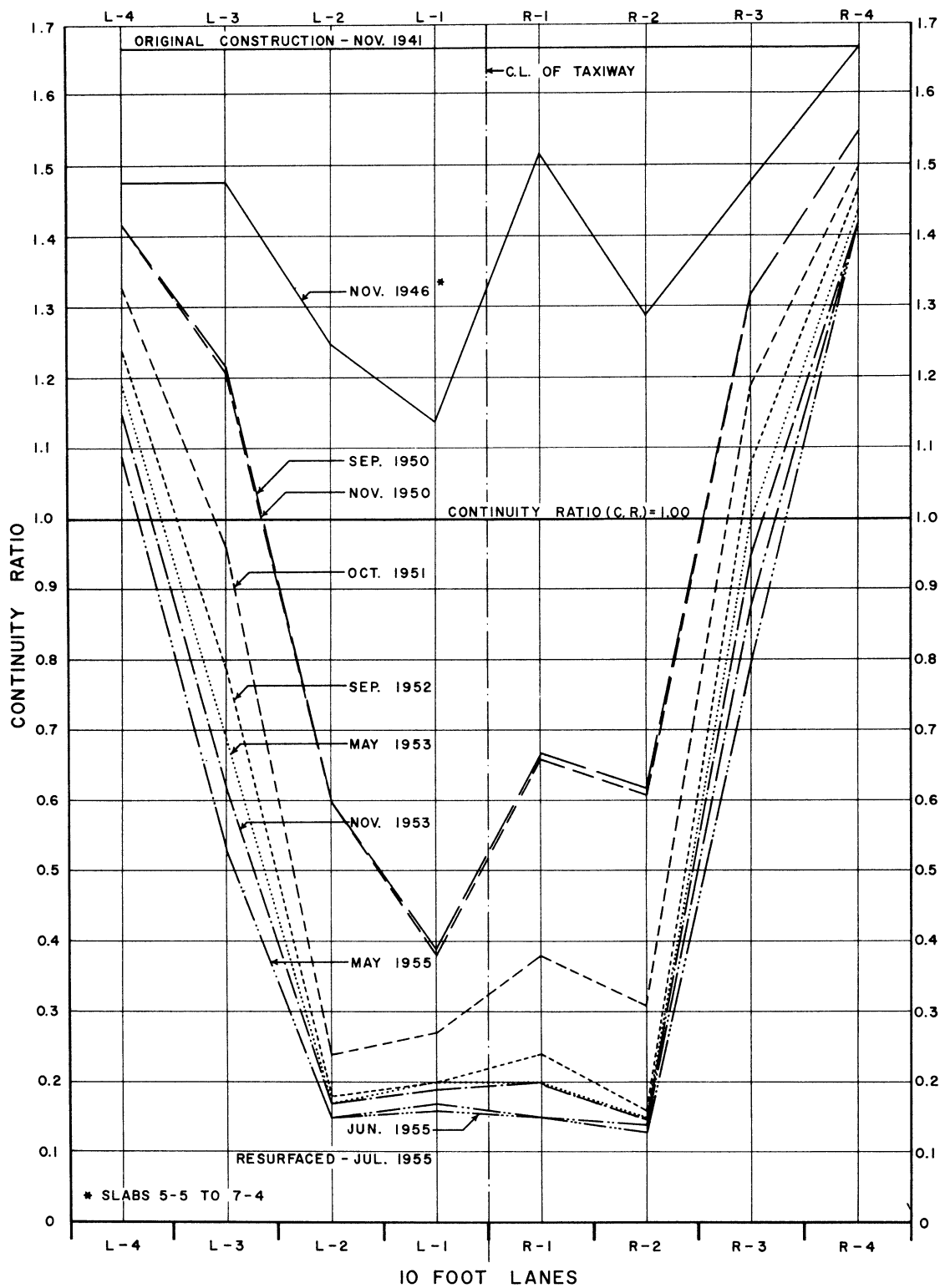
WILLOW RUN AIRFIELD

FIG. 2



CHANGE IN CONTINUITY RATIO
 CENTER TAXIWAY B (SLABS 5-5 TO 8-6)
 WILLOW RUN AIRFIELD

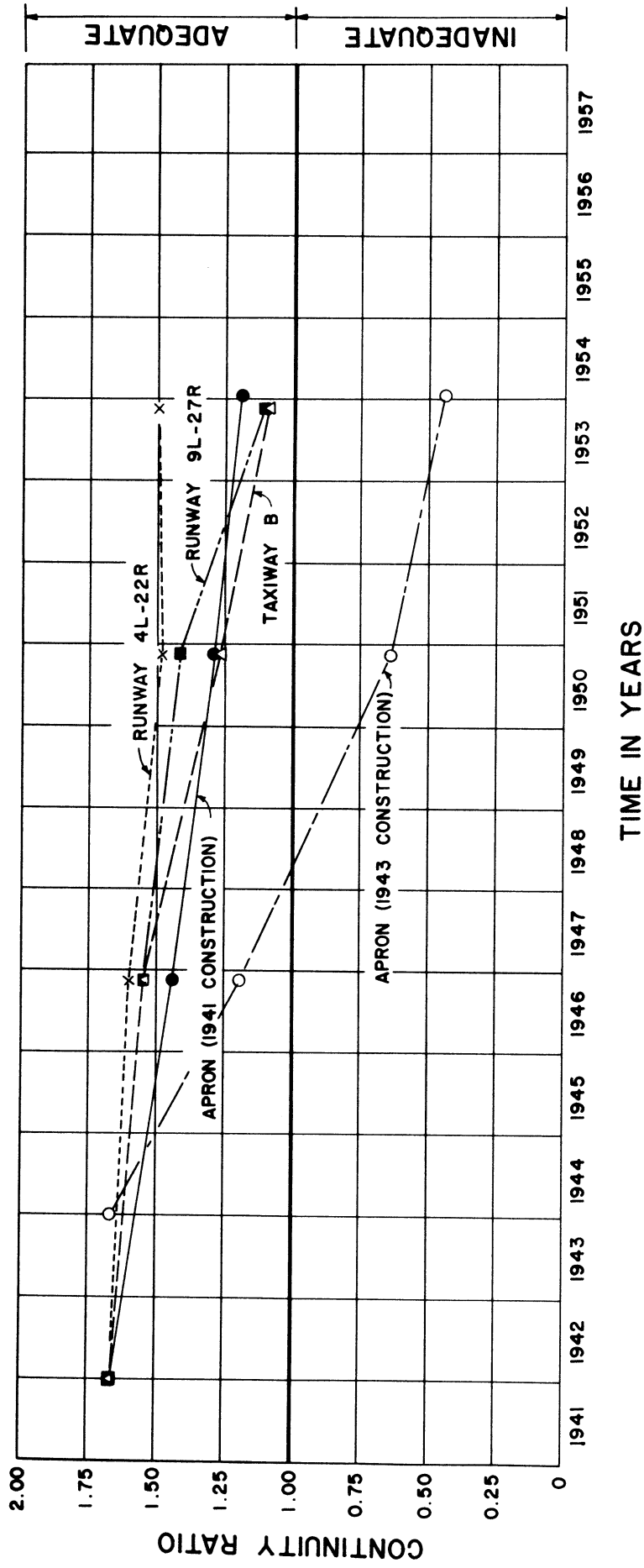
FIG. 3



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

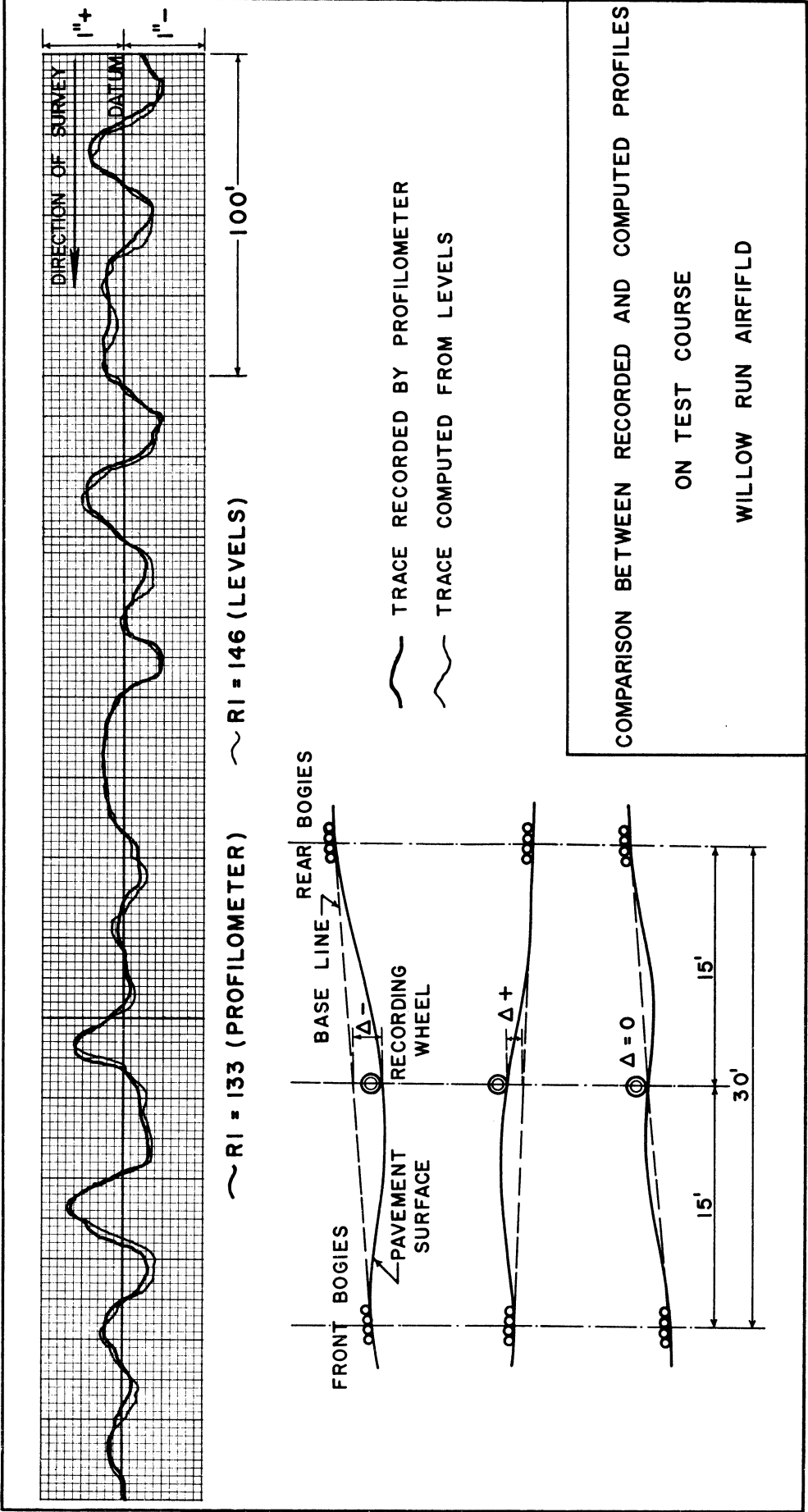


FIG. 7

TENTATIVE PAVEMENT ROUGHNESS RATING
VERTICAL DISPLACEMENT - INCHES PER MILE

U.S. BUREAU OF PUBLIC ROADS ⁽¹⁾ SINGLE WHEEL ROUGHOMETER AS MODIFIED BY MICHIGAN STATE HIGHWAY DEPARTMENT	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

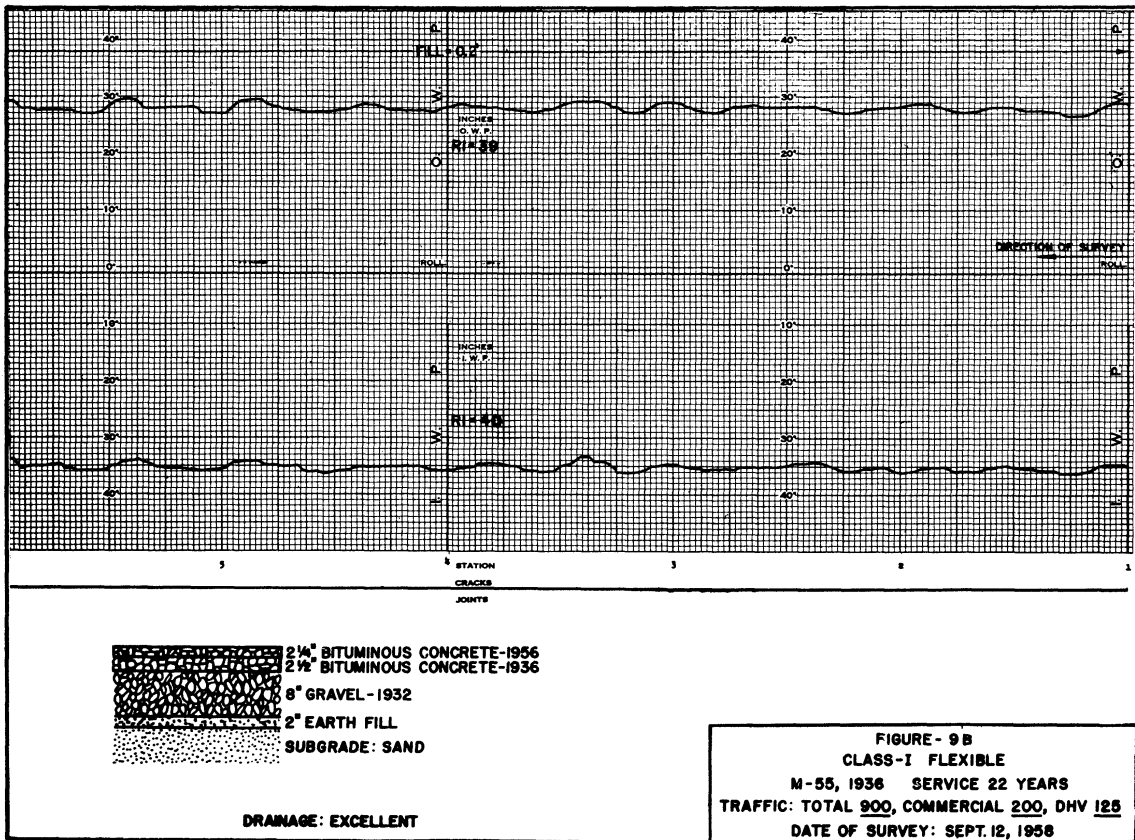
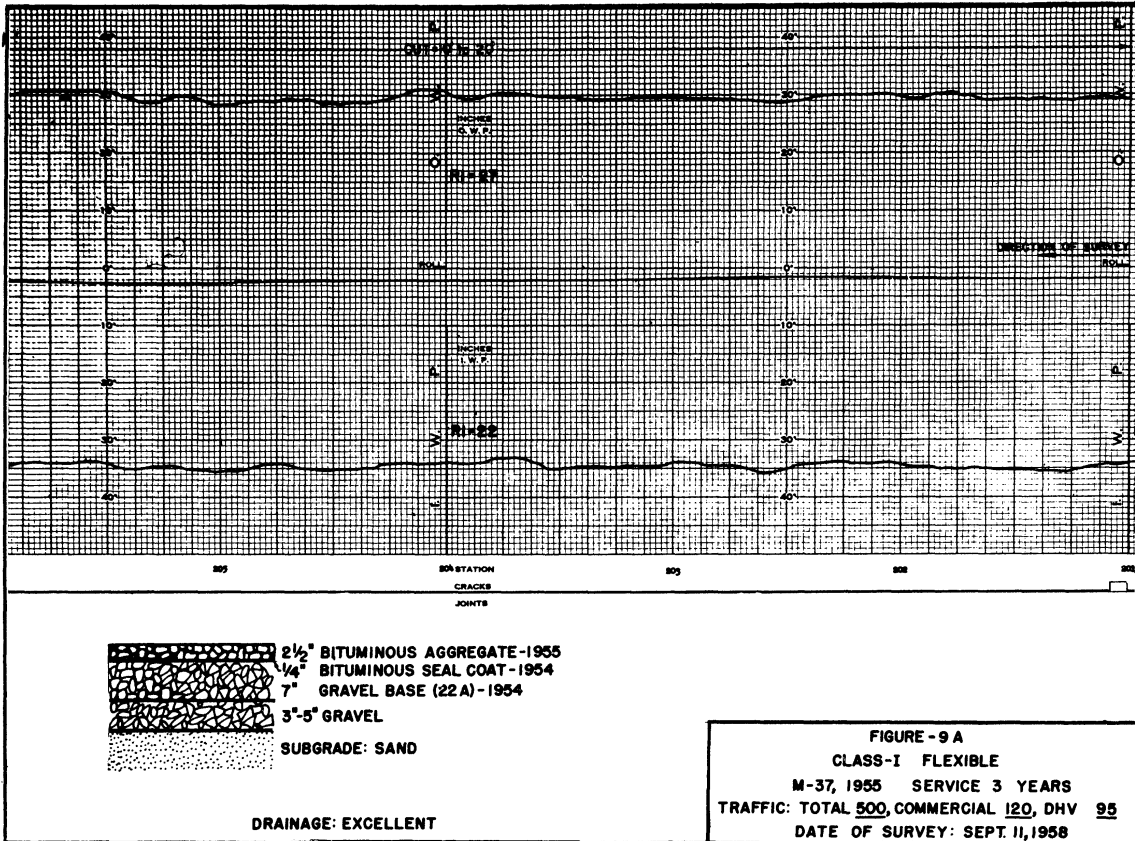


FIG. 9

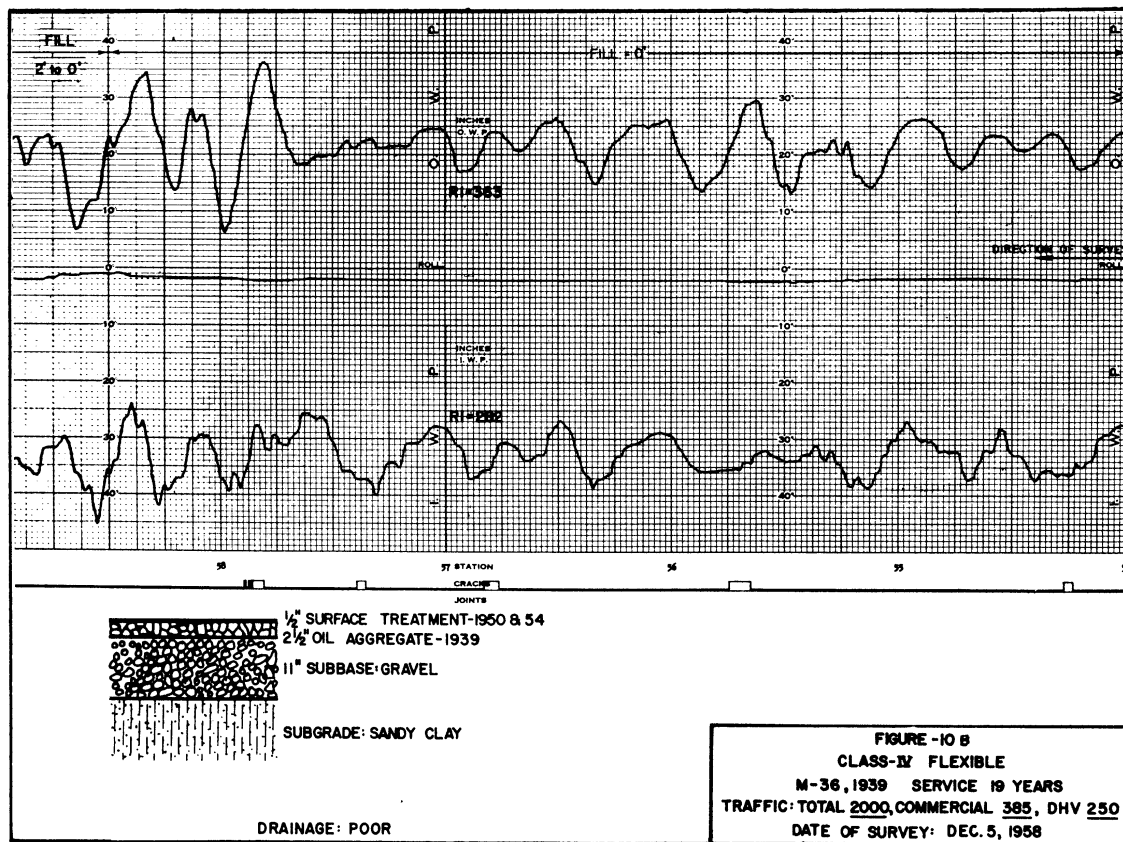
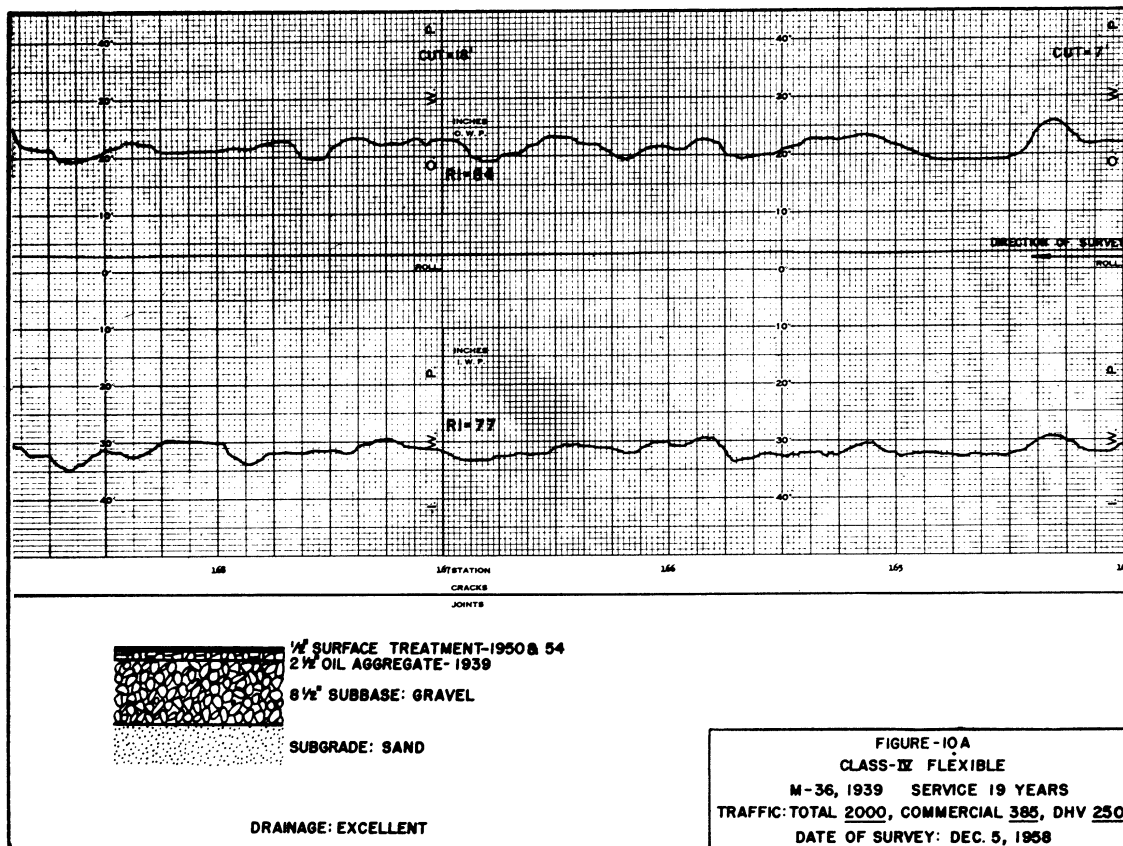


FIG. 10

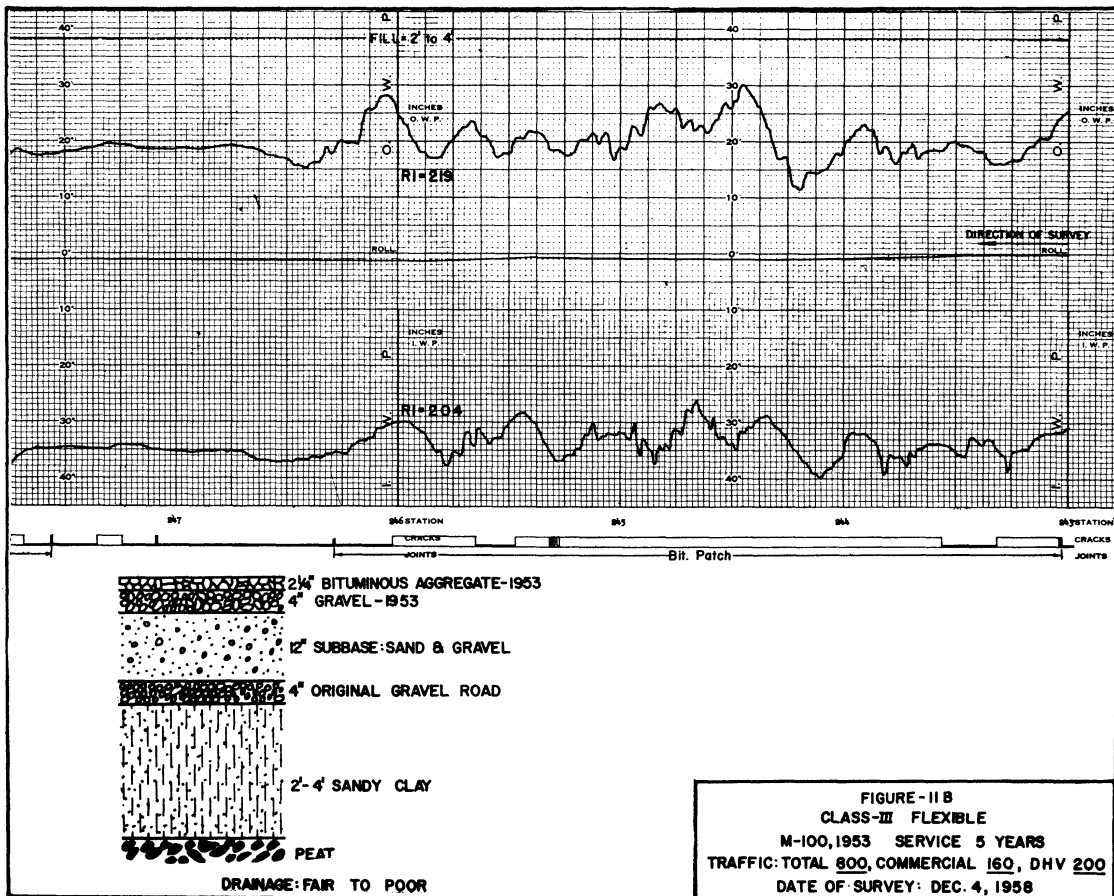
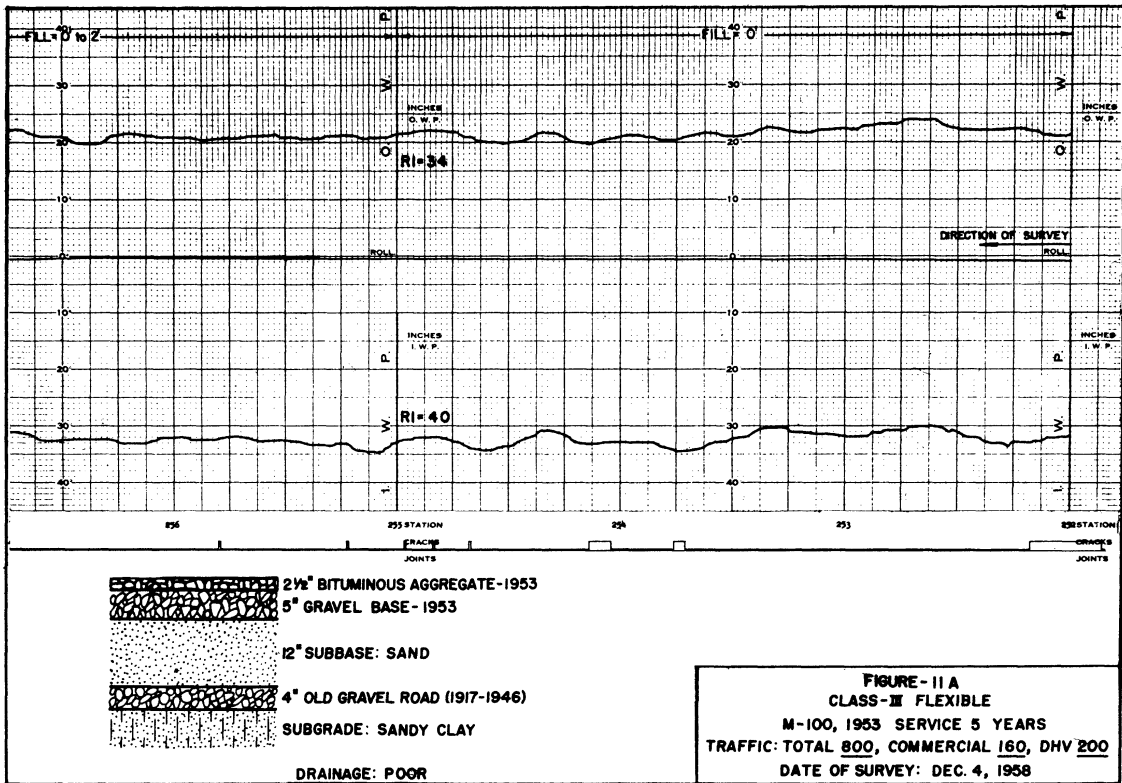


FIG. 11

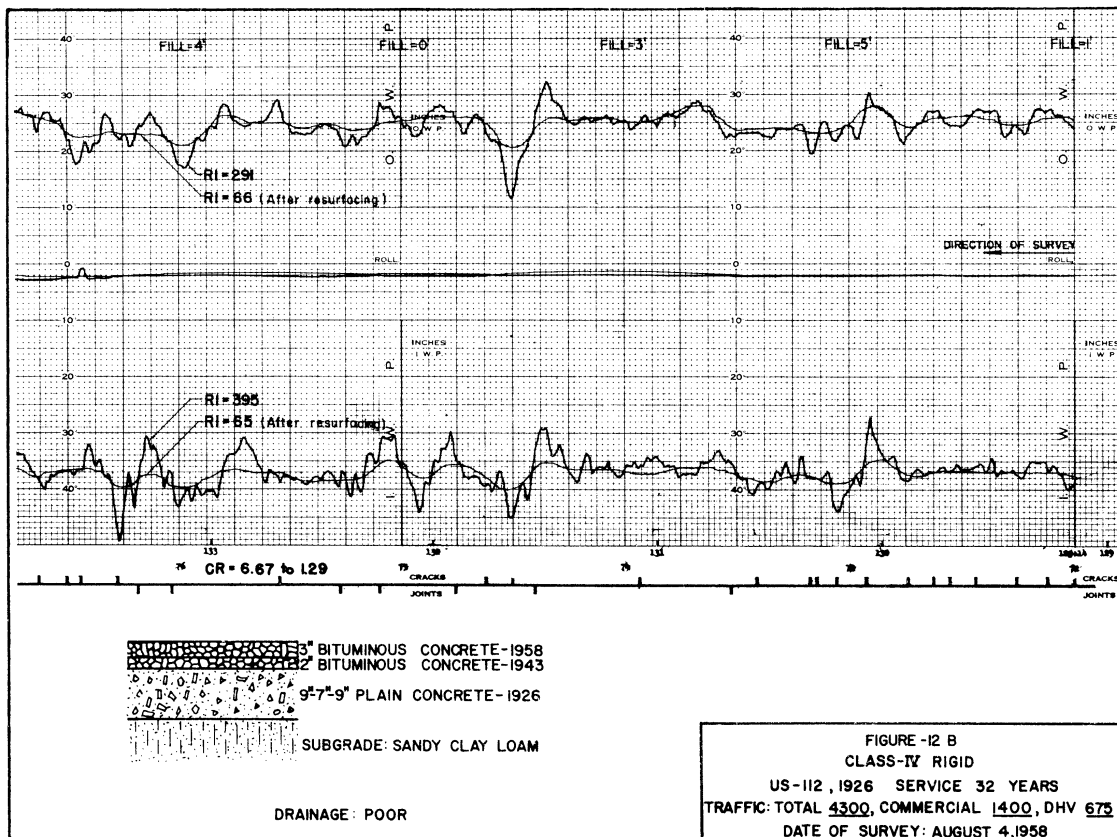
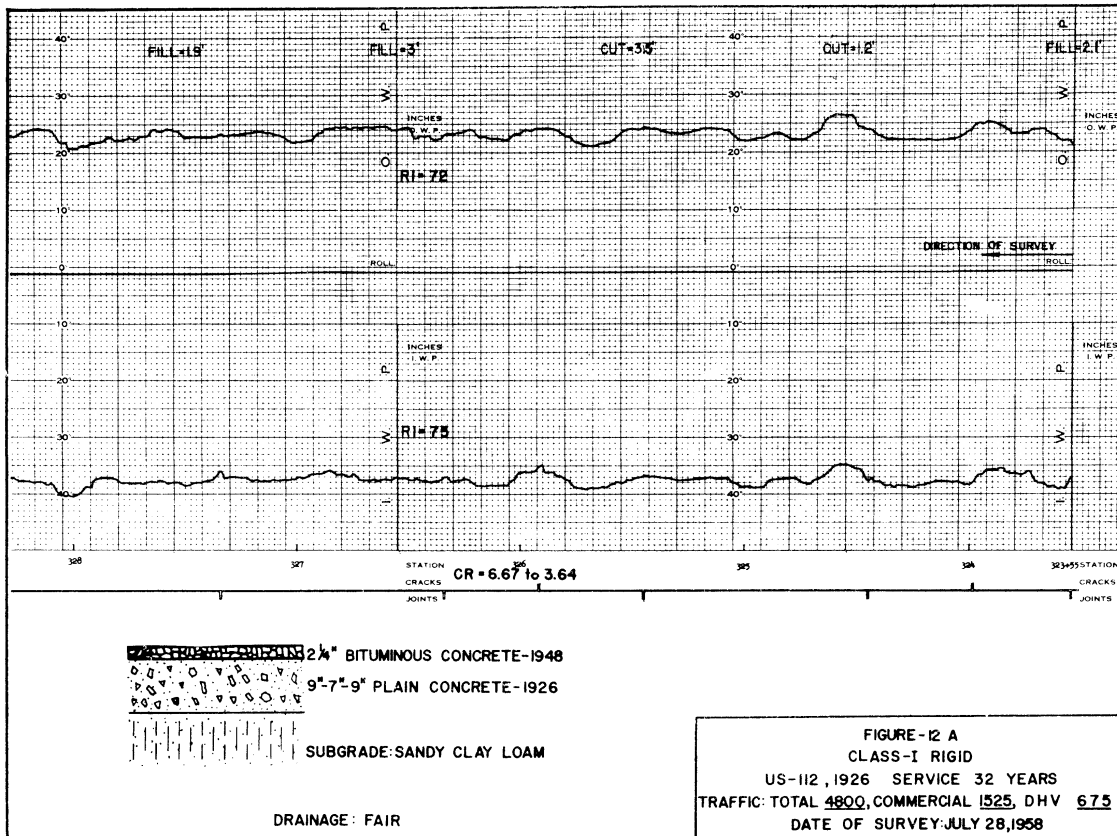


FIG. 12

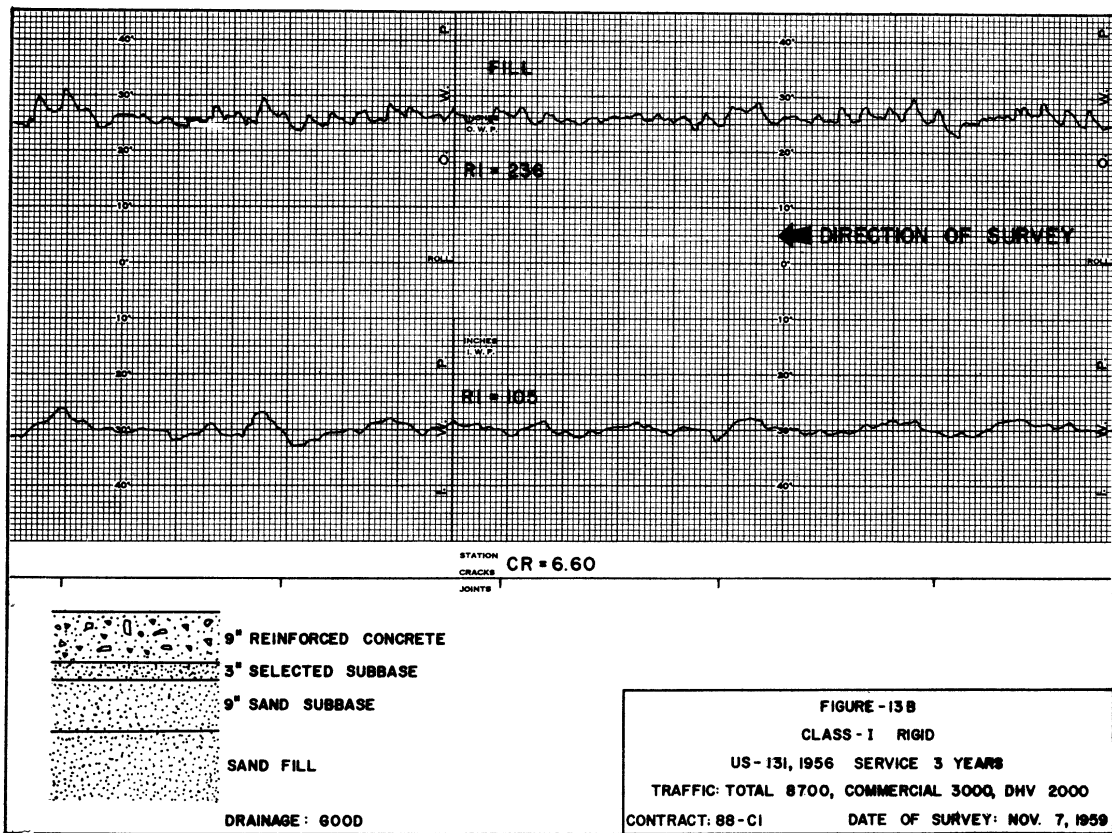
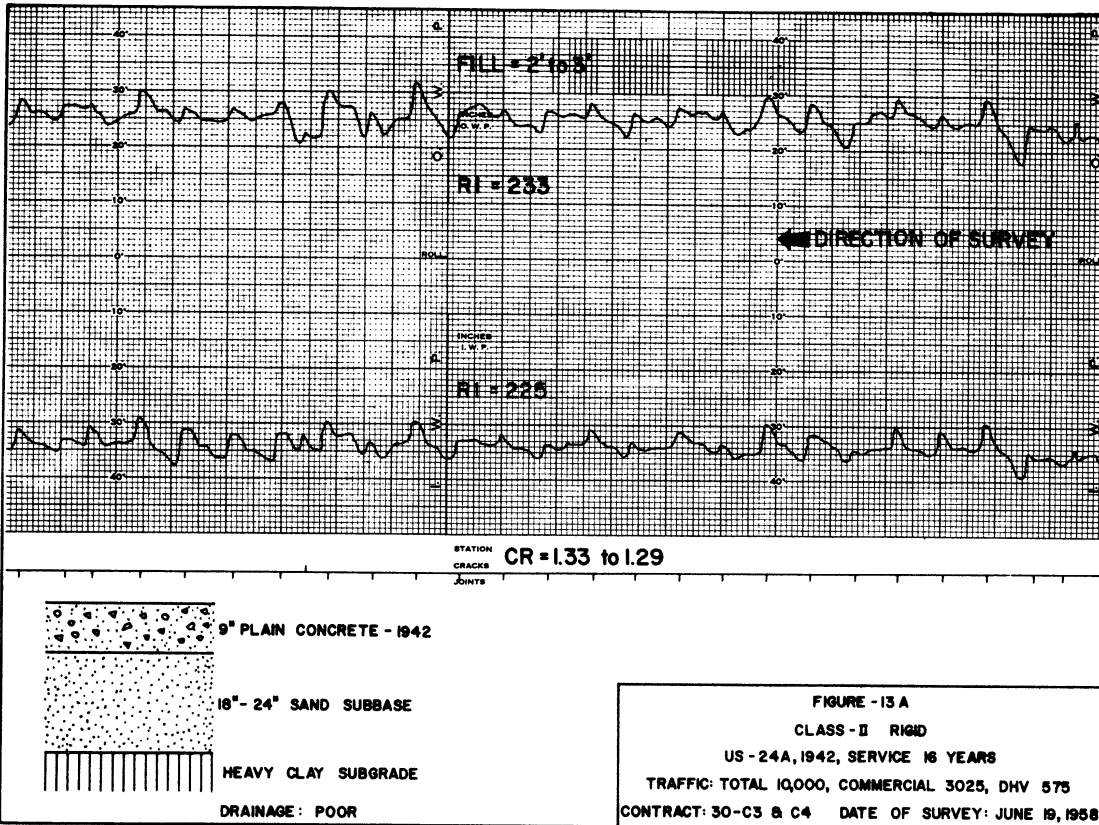


FIG. 13

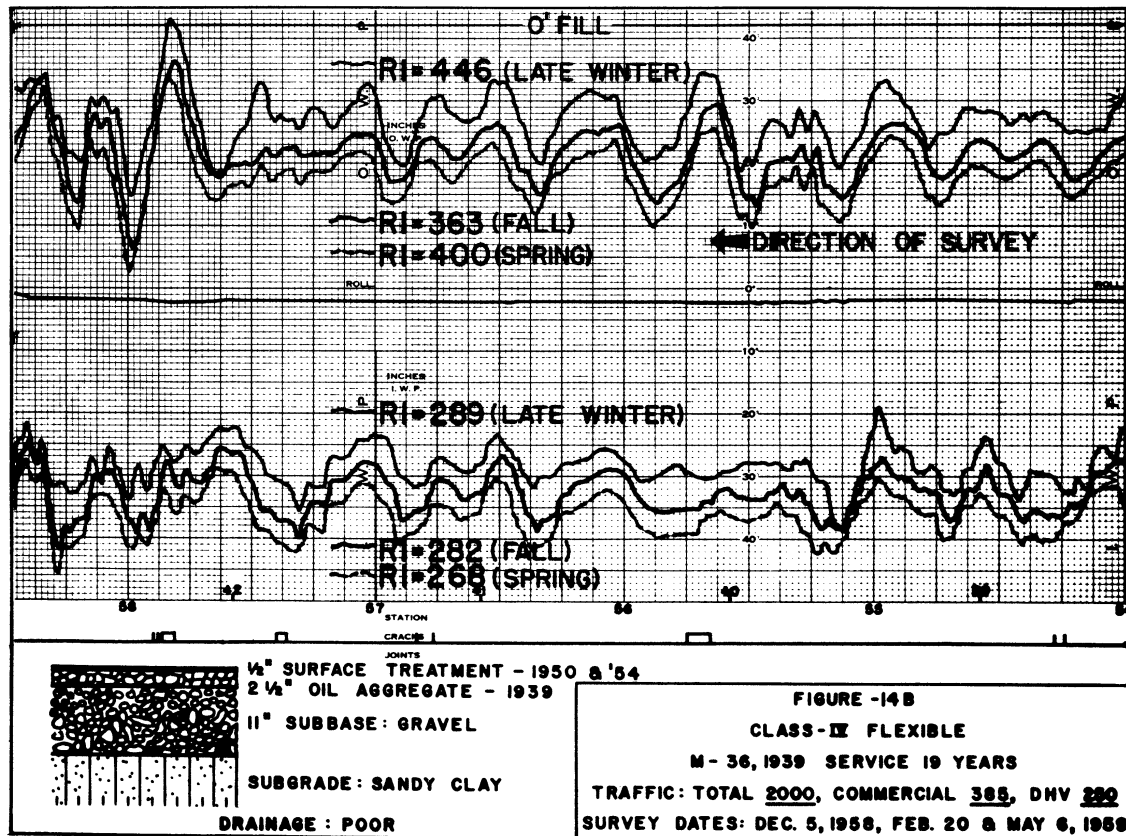
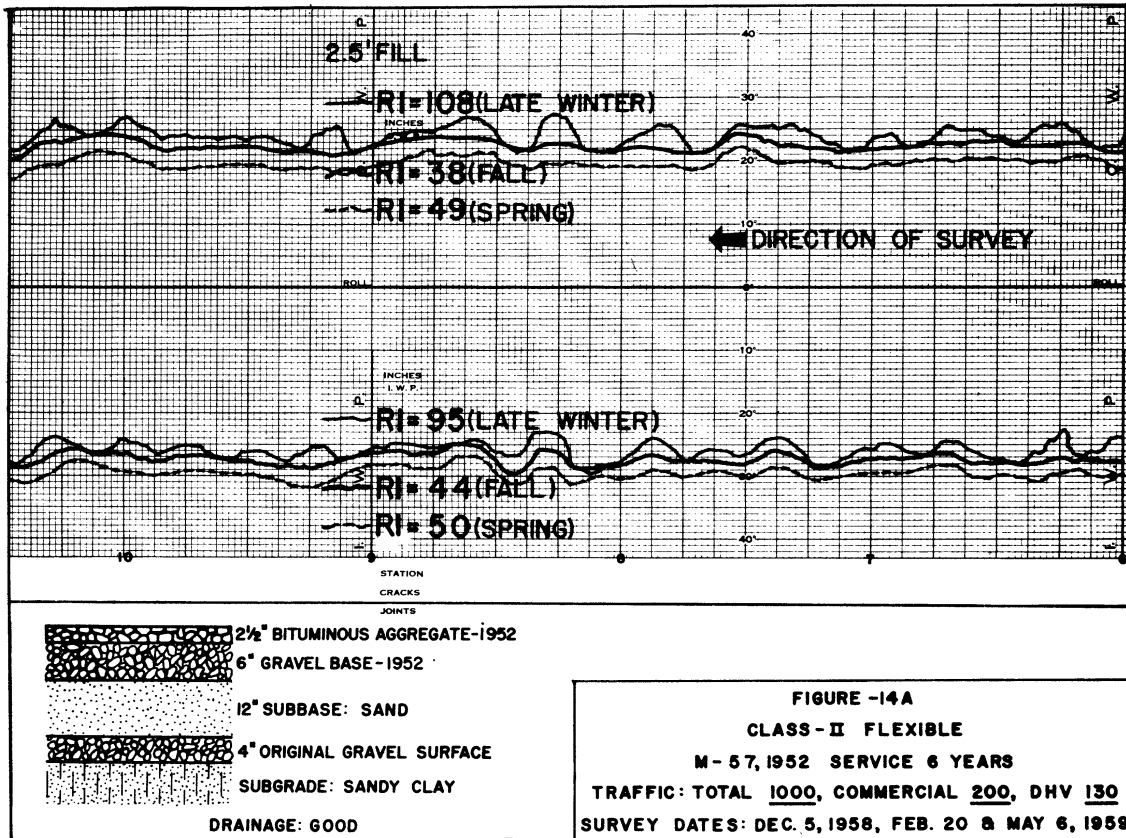
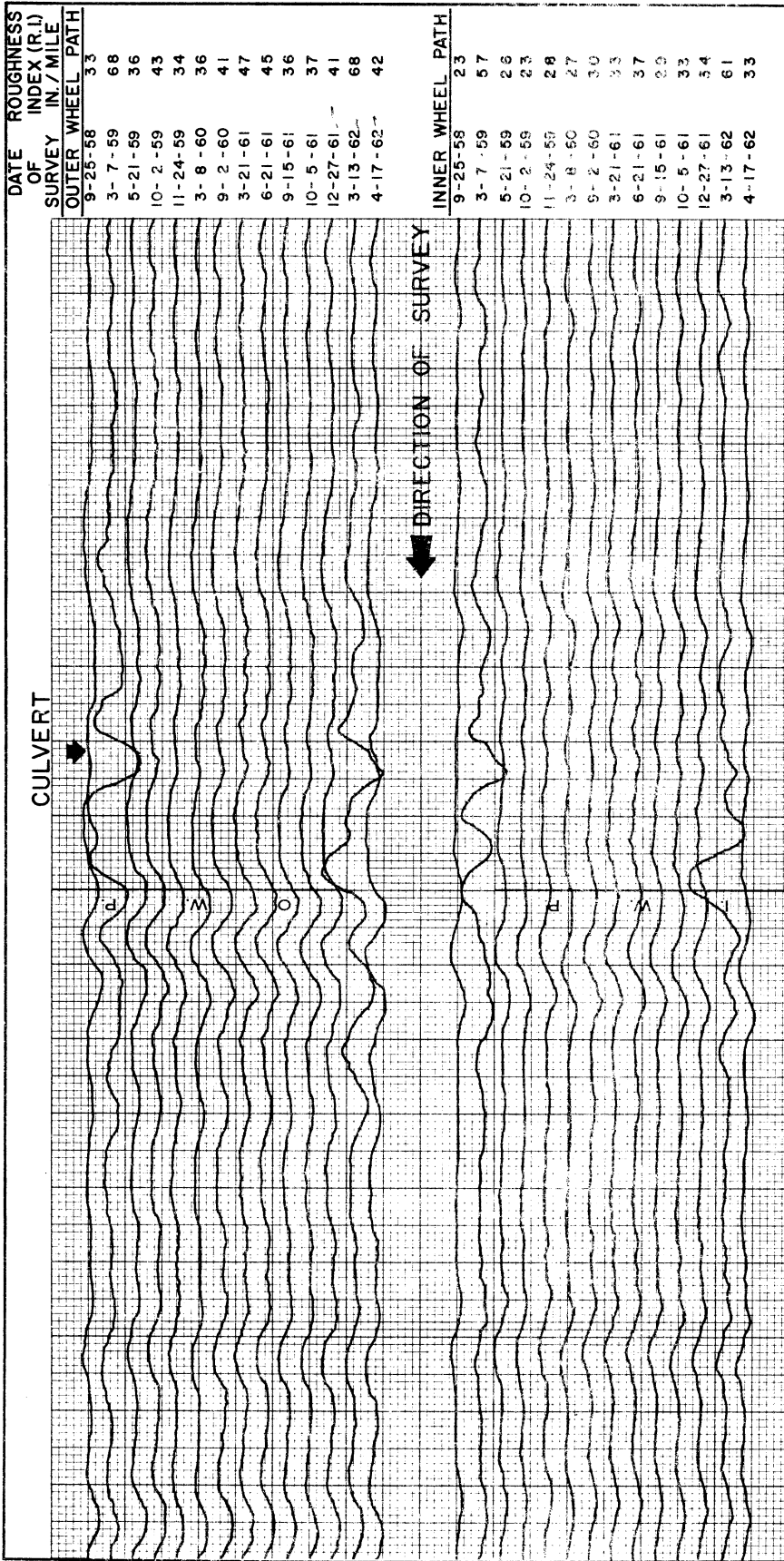


FIG. 14



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
 3" SELECTED AGGREGATE SUBBASE] OPERATION

25" (MINIMUM) SAND SUBGRADE

ORIGINAL SOIL - SAUGATUCK SAND

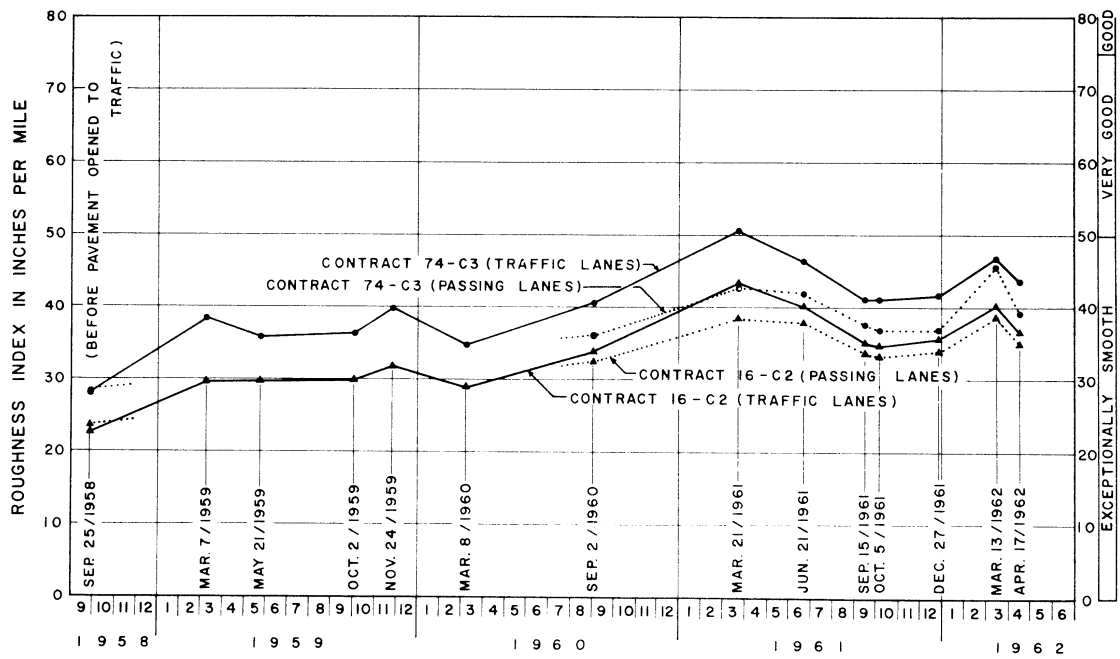
DRAINAGE: EXCELLENT

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

DATE OF SURVEY	ROUGHNESS INDEX (R.I) IN./MILE
9-25-58	33
3-7-59	68
5-21-59	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

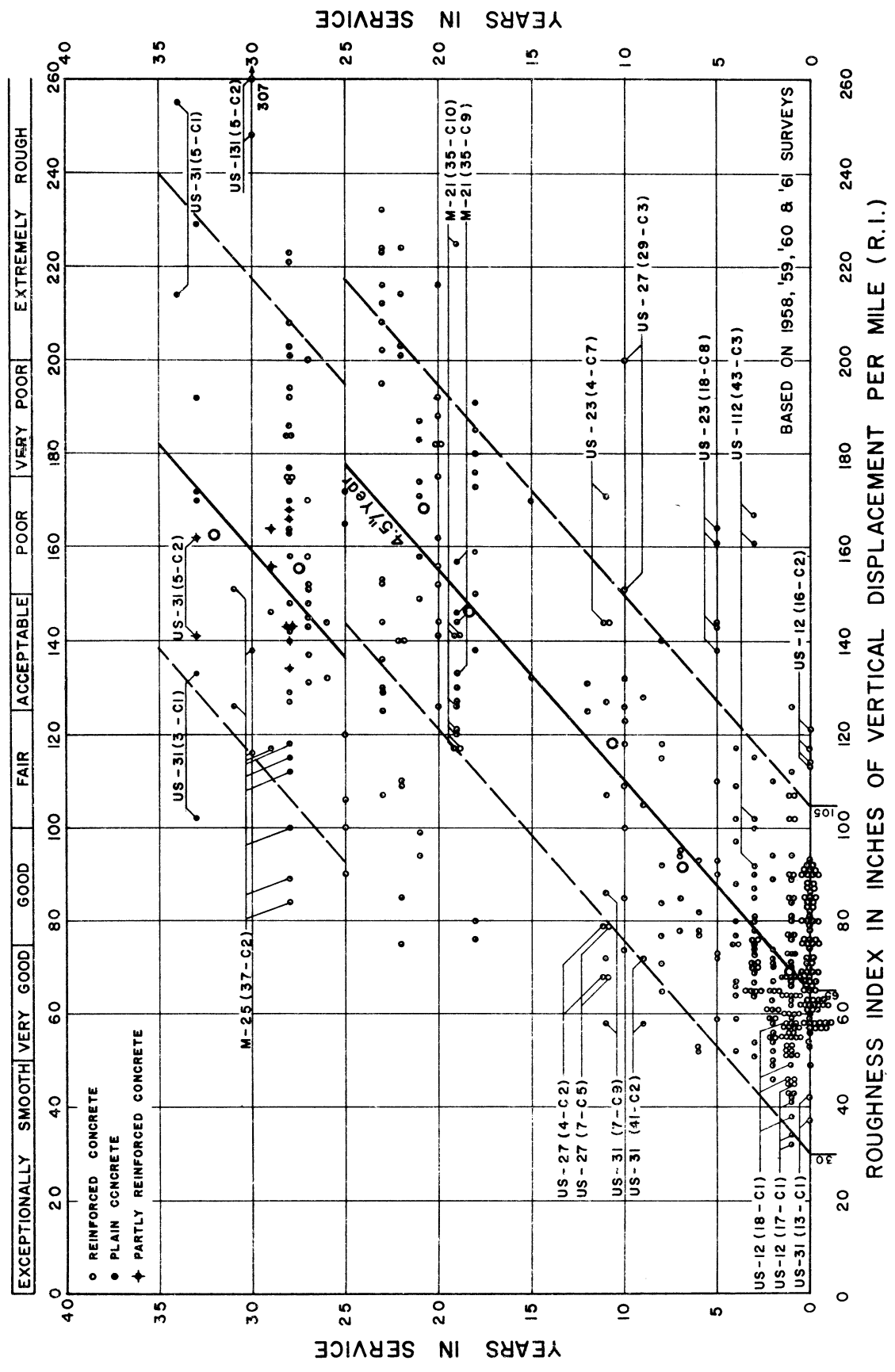
INNER WHEEL PATH	ROUGHNESS INDEX (R.I) IN./MILE
9-25-58	23
3-7-59	57
5-21-59	28
10-2-59	23
11-24-59	28
3-8-60	27
9-2-60	30
3-21-61	33
6-21-61	37
9-15-61	29
10-5-61	33
12-27-61	34
3-13-62	61
4-17-62	33

FIG. 15



CUMULATIVE CHANGES IN ROUGHNESS
 US-31 MUSKEGON - GRAND HAVEN EXPRESSWAY
 CLASS-1 FLEXIBLE PAVEMENT

FIG. 16

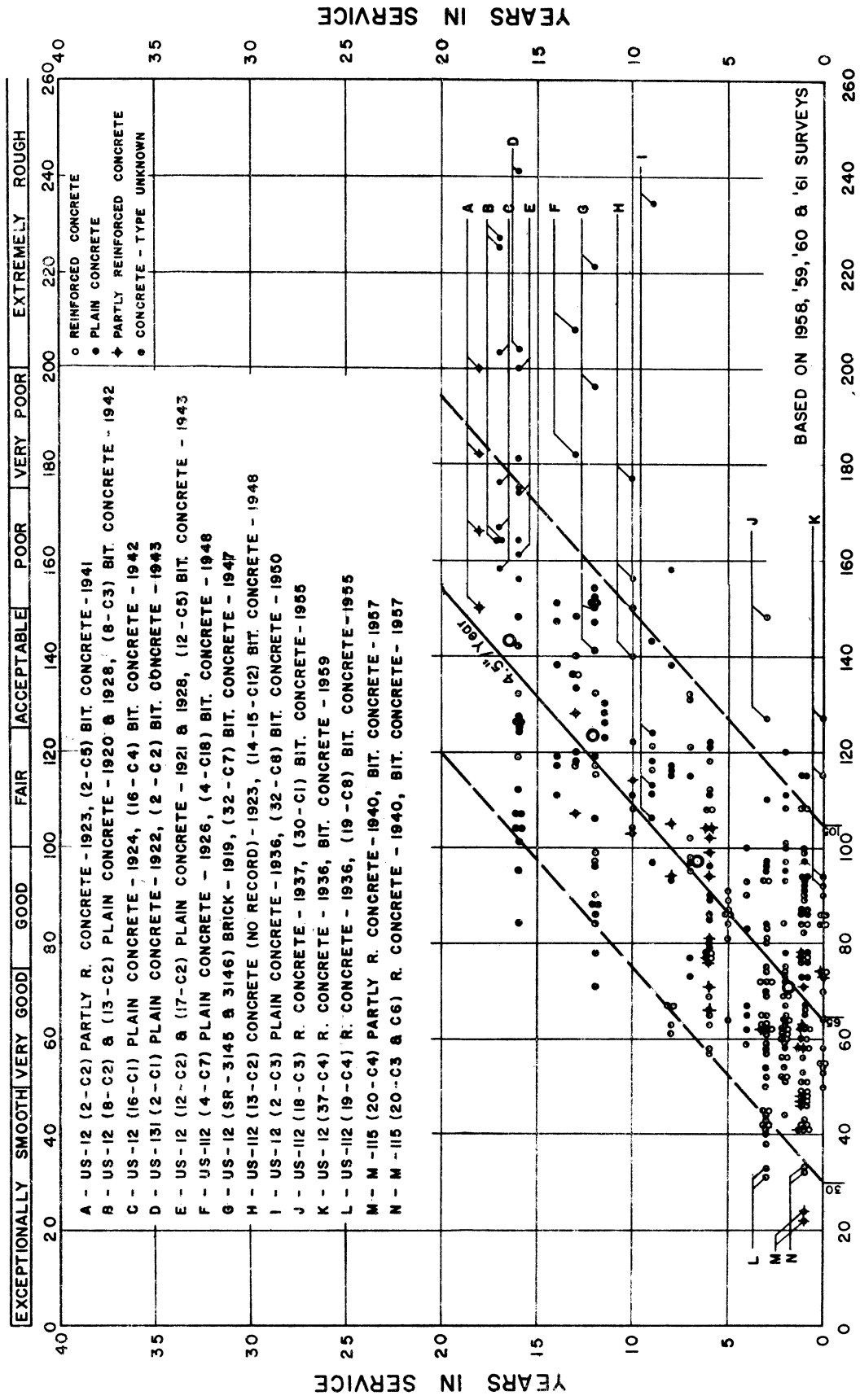


CUMULATIVE CHANGES WITH AGE IN SERVICE

RIGID PAVEMENT - CLASS - I

TRAFFIC LANE

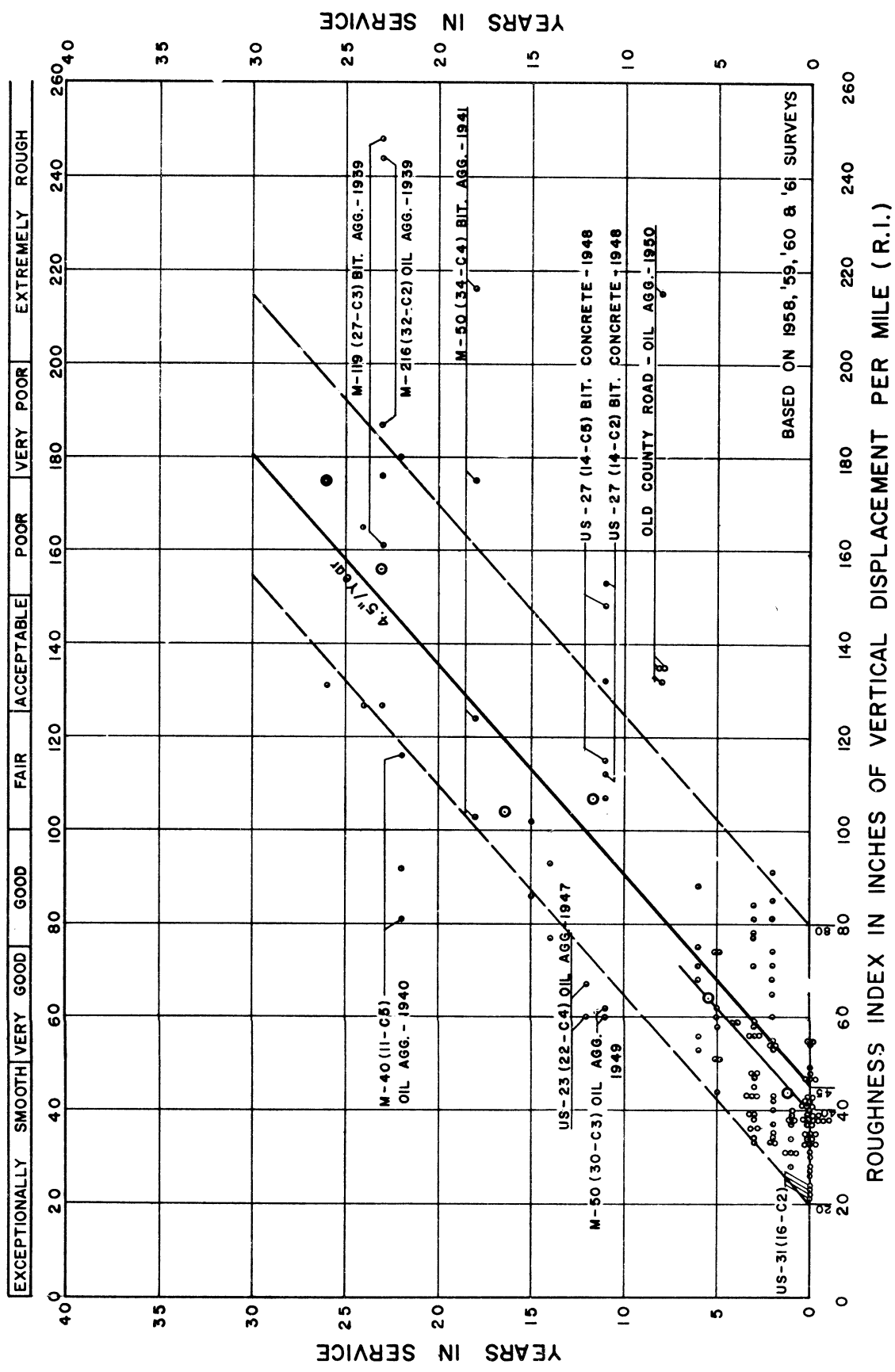
FIG. 17



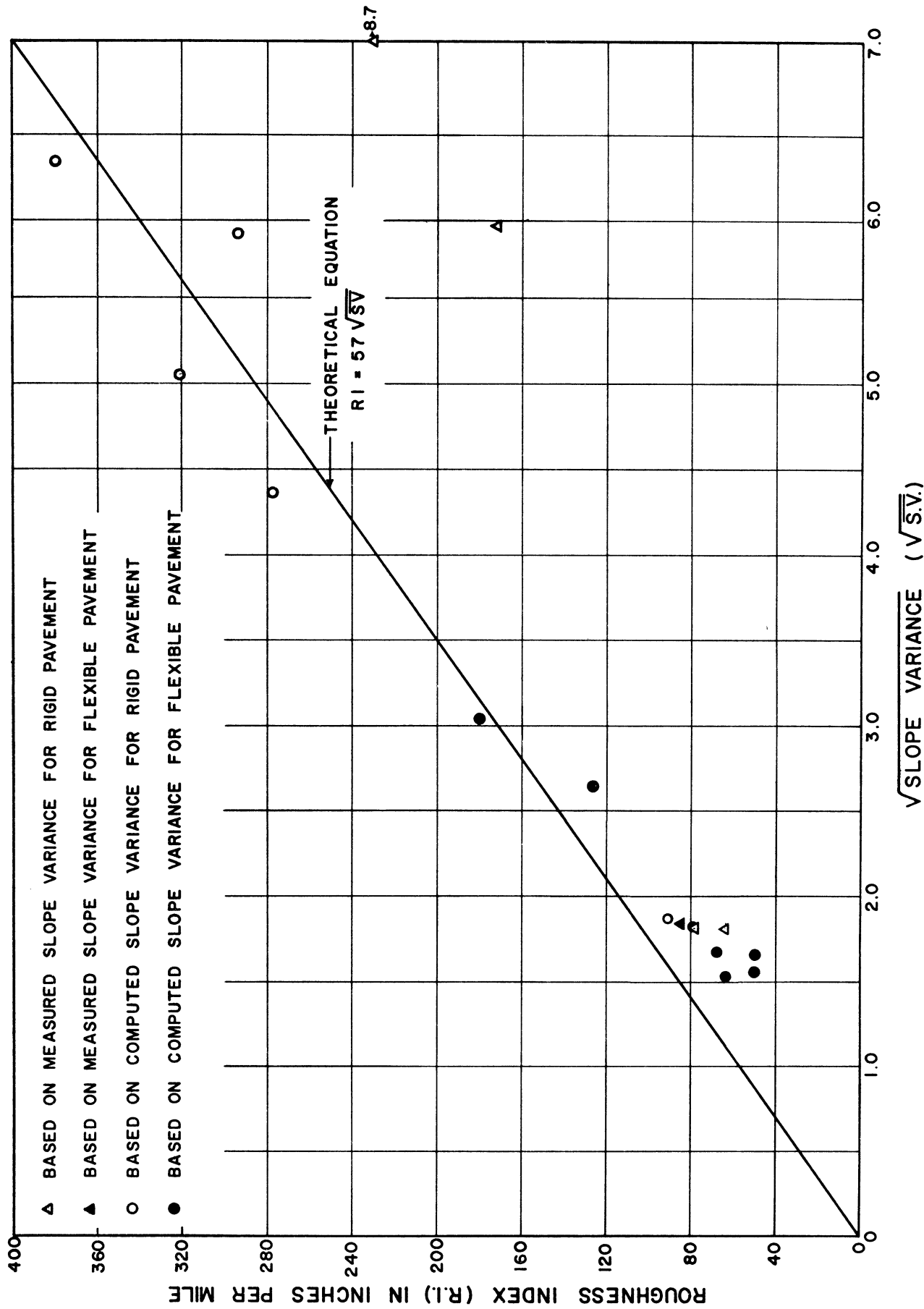
CUMULATIVE CHANGES WITH AGE IN SERVICE
 RECAPPED CONCRETE PAVEMENT - CLASS - I

TRAFFIC LANE

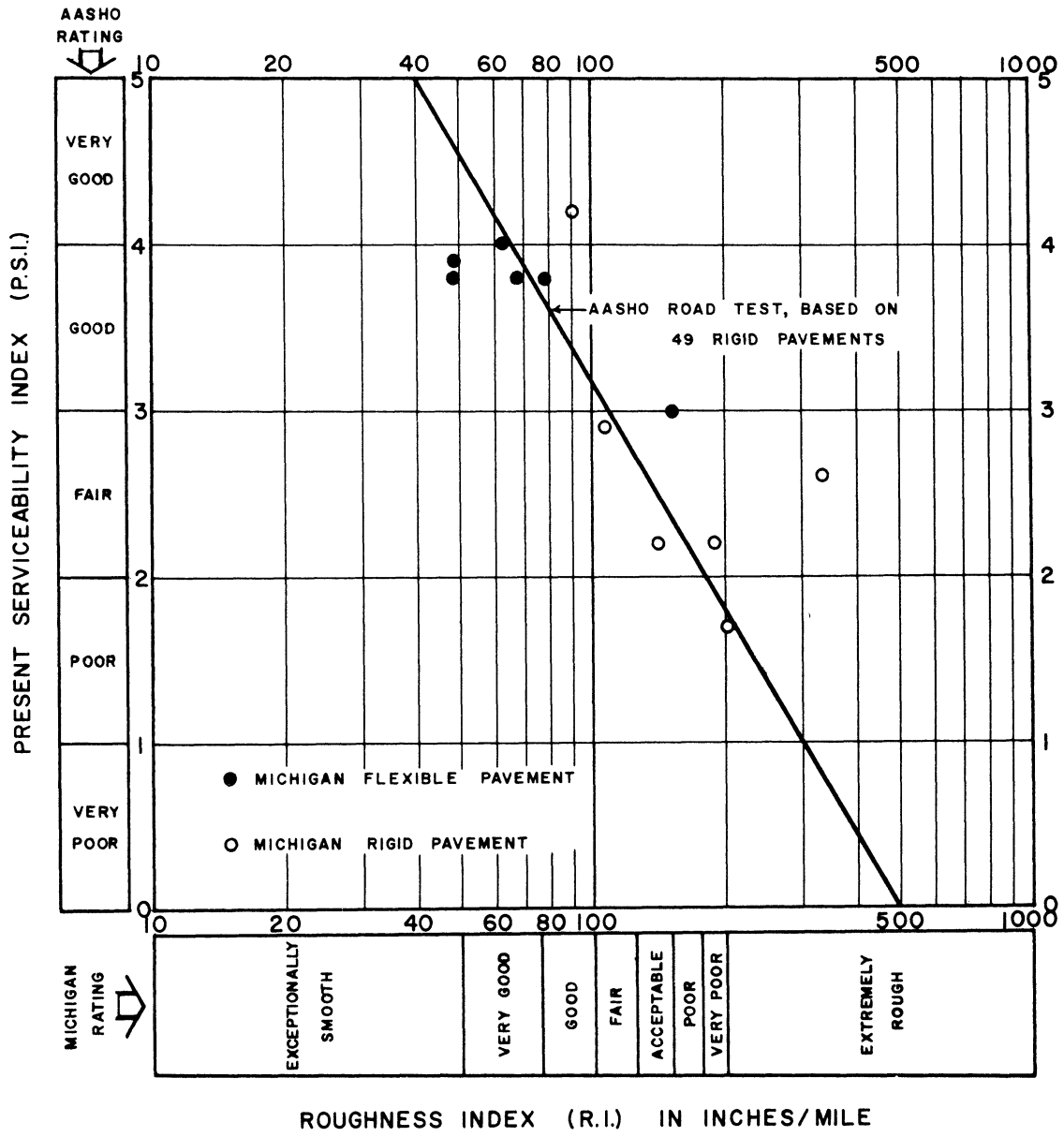
FIG. 18



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



MICHIGAN ROUGHNESS INDEX
VS.
AASHO SLOPE VARIANCE



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

