DESIGN MAINTENANCE AND PERFORMANCE OF RESURFACED PAVEMENTS AT WILLOW RUN AIRFIELD

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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 *			
				Direct		Total		Direct		Total	
		1957	1961	1957	1961	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	-7	4
T-3	3"	31	41	19	22	34	28	12	19	-3	13
A-1	3"	44	47	33	29	38	30	11	18	6	17
Average	3"	35	41	26	26	37	30	10	16	-1	12

* T-1 omitted

TEST AREA A-2

Base Course	Thickness	With Mesh			
		Direct		Total	
		1957	1961	1957	1961
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	3"	54	54
Epon Asphalt	3"	13	30
Differential		41	24

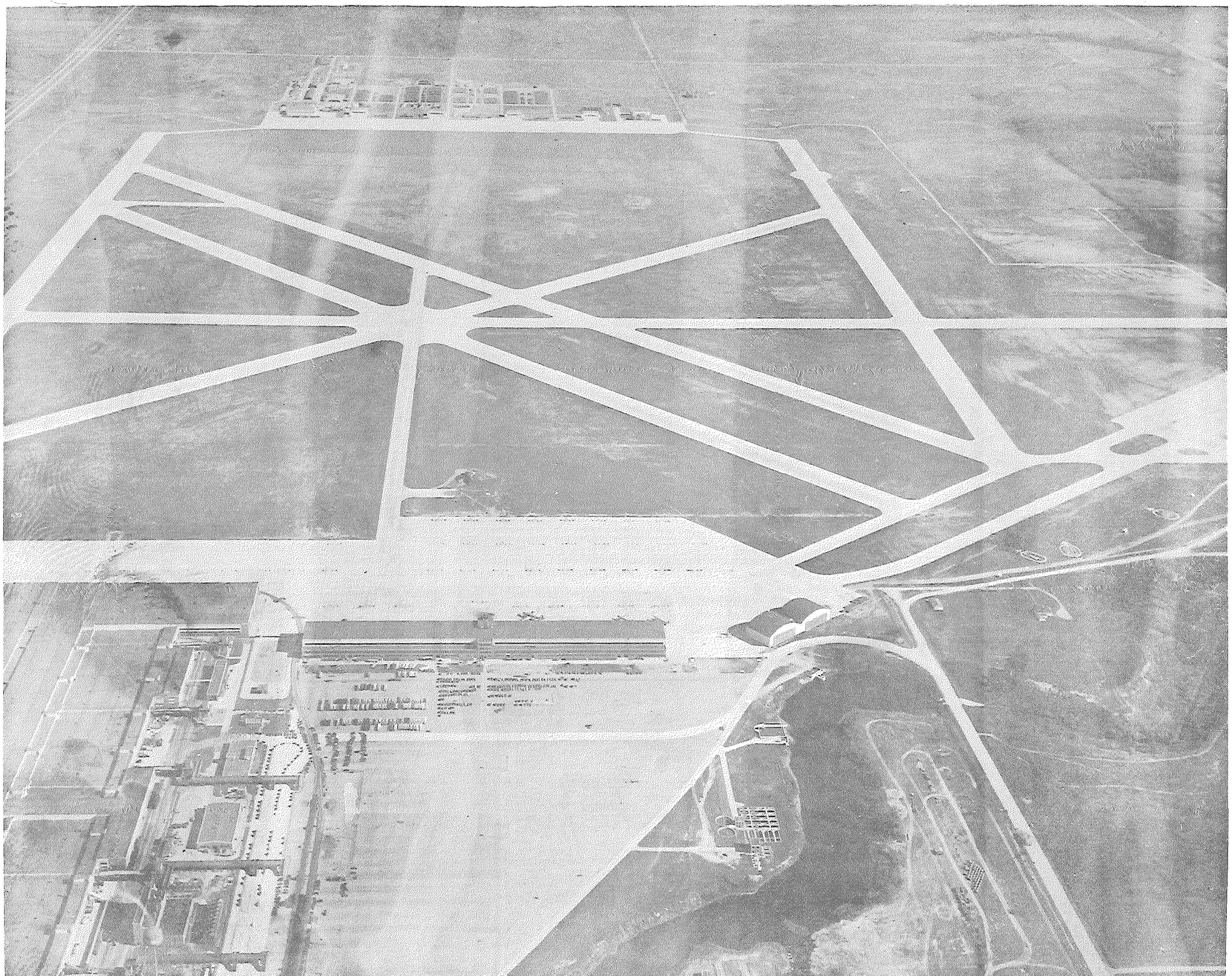


FIG. 1

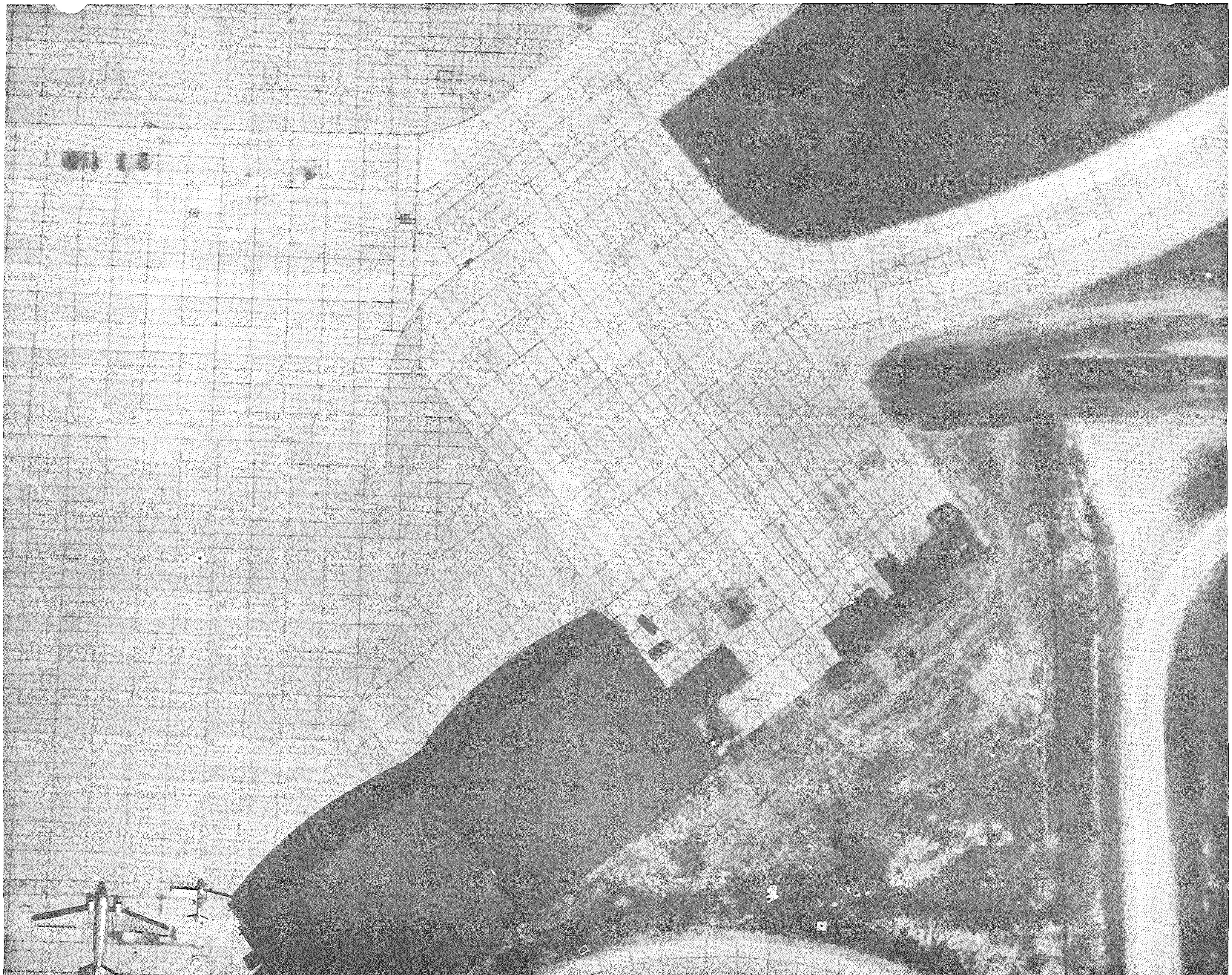


FIG. 2

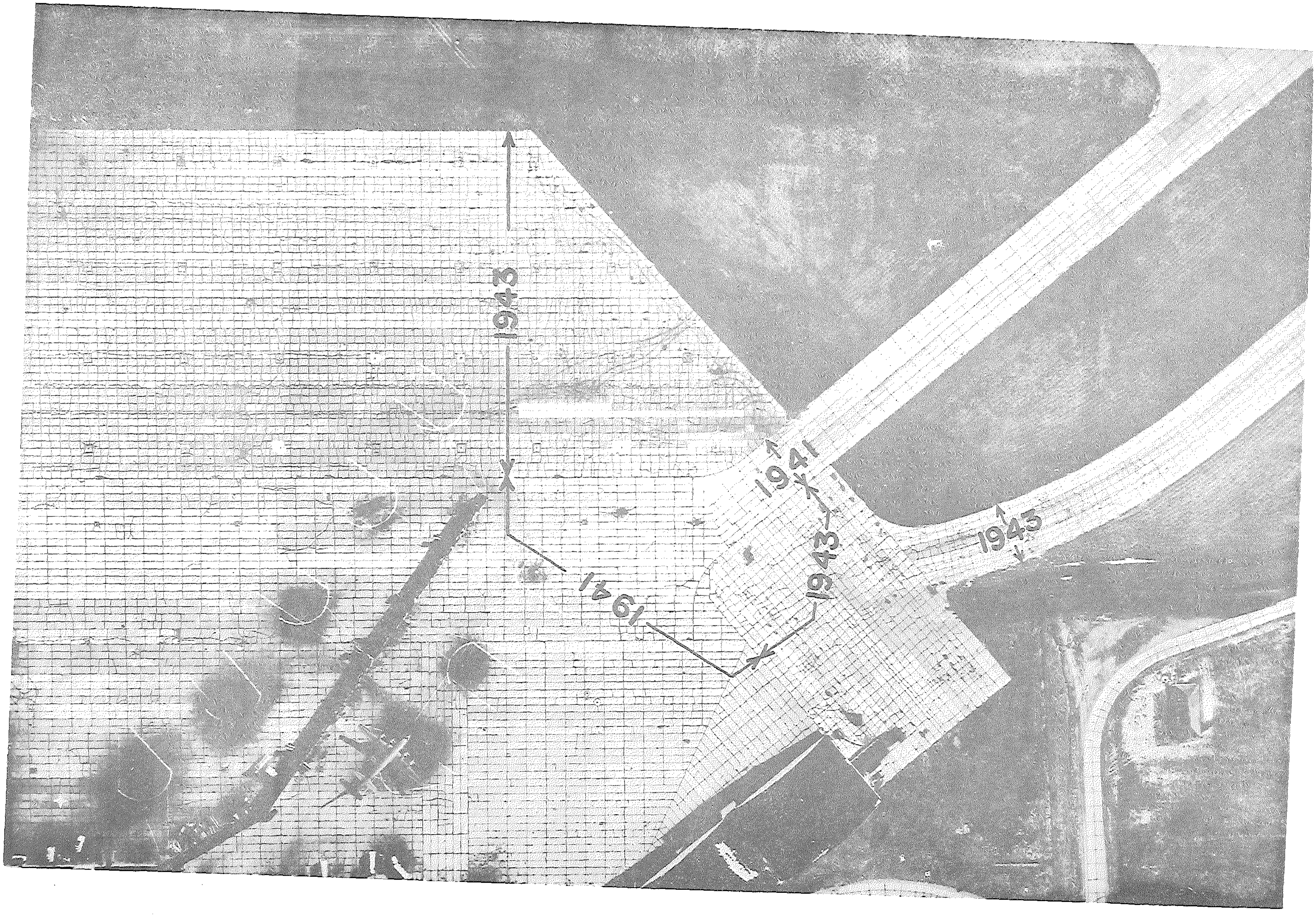


FIG 3

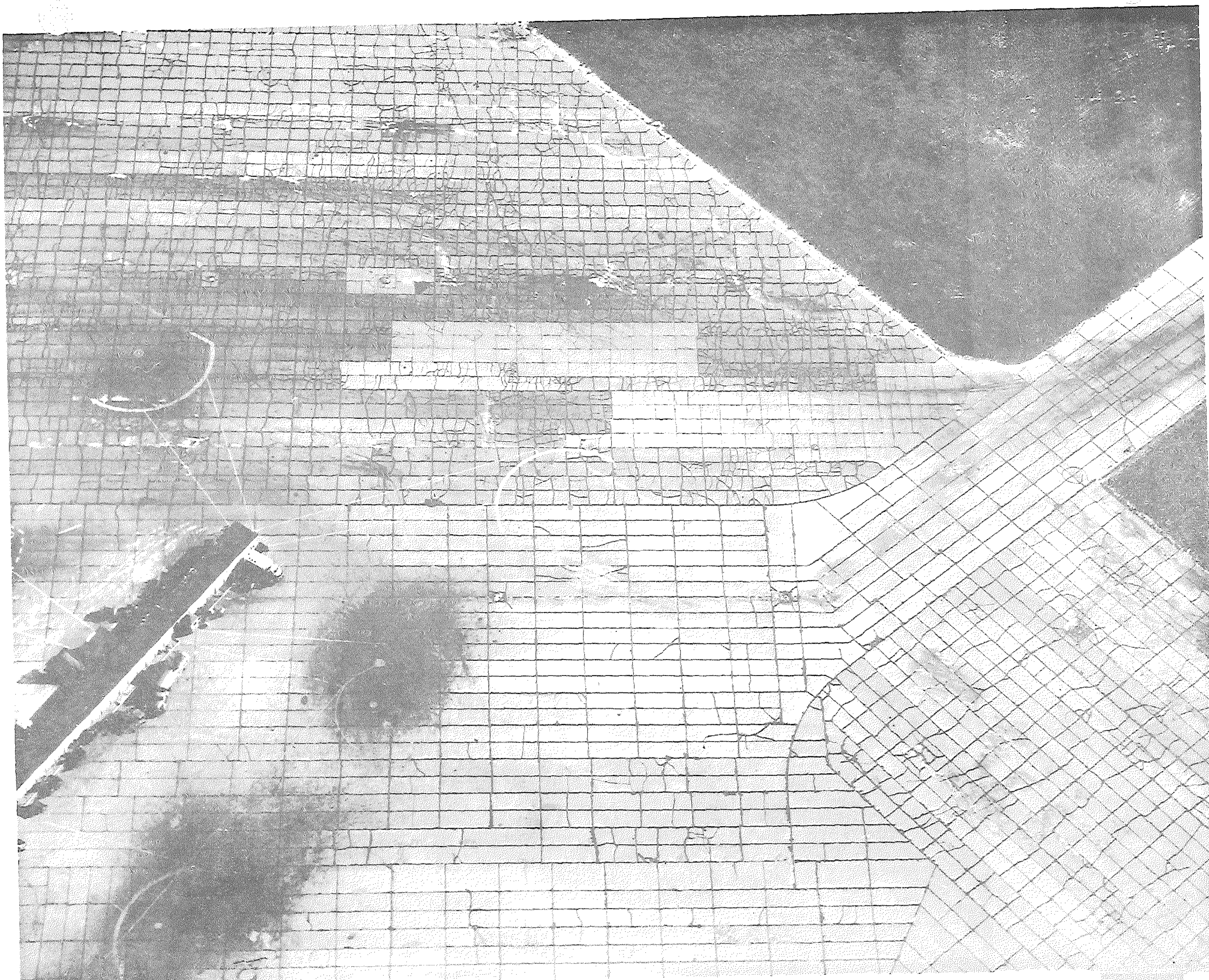
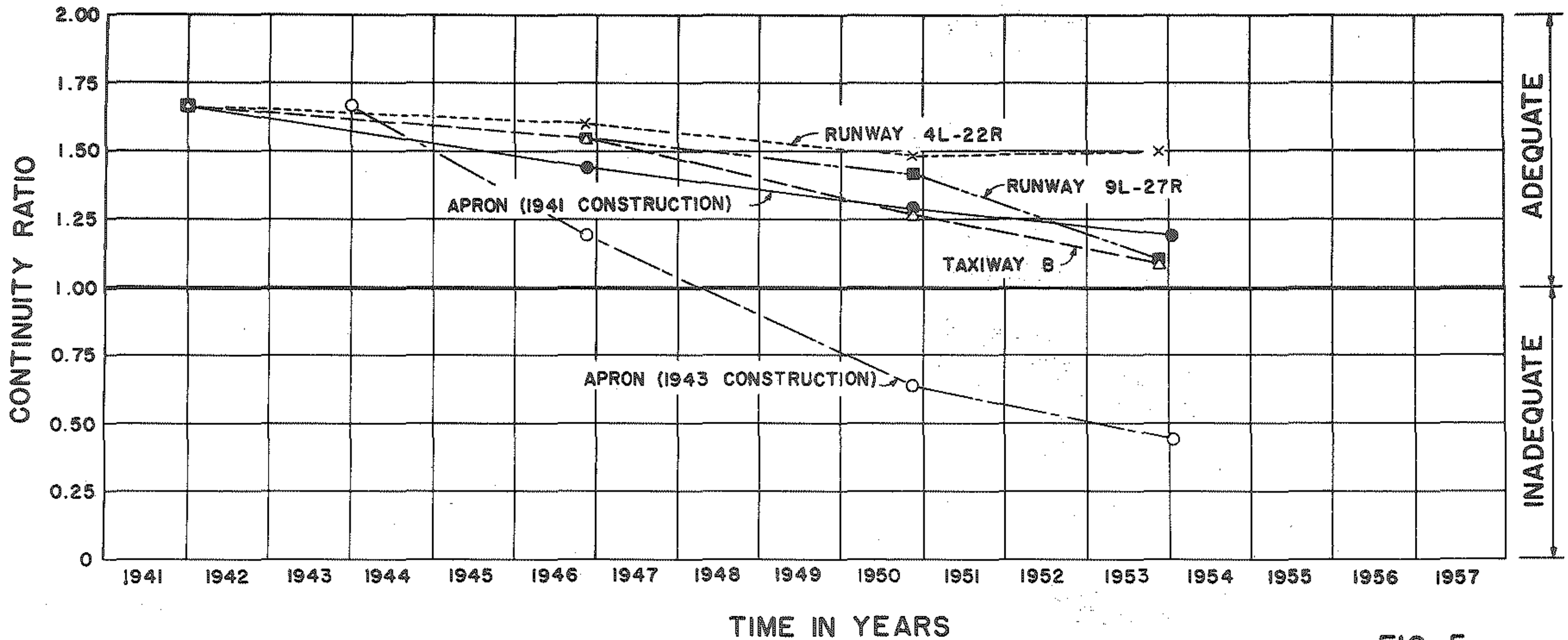


FIG. 4



2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0
CONTINUITY RATIO
1941
1942
1943
1944
1945
1946
1947
1948
1949
1950
1951
1952
1953
1954
1955
1956
1957
TIME IN YEARS
RUNWAY 4L-22R
RUNWAY 9L-27R
APRON (1941 CONSTRUCTION)
TAXIWAY B
APRON (1943 CONSTRUCTION)
ADEQUATE
INADEQUATE

FIG. 5

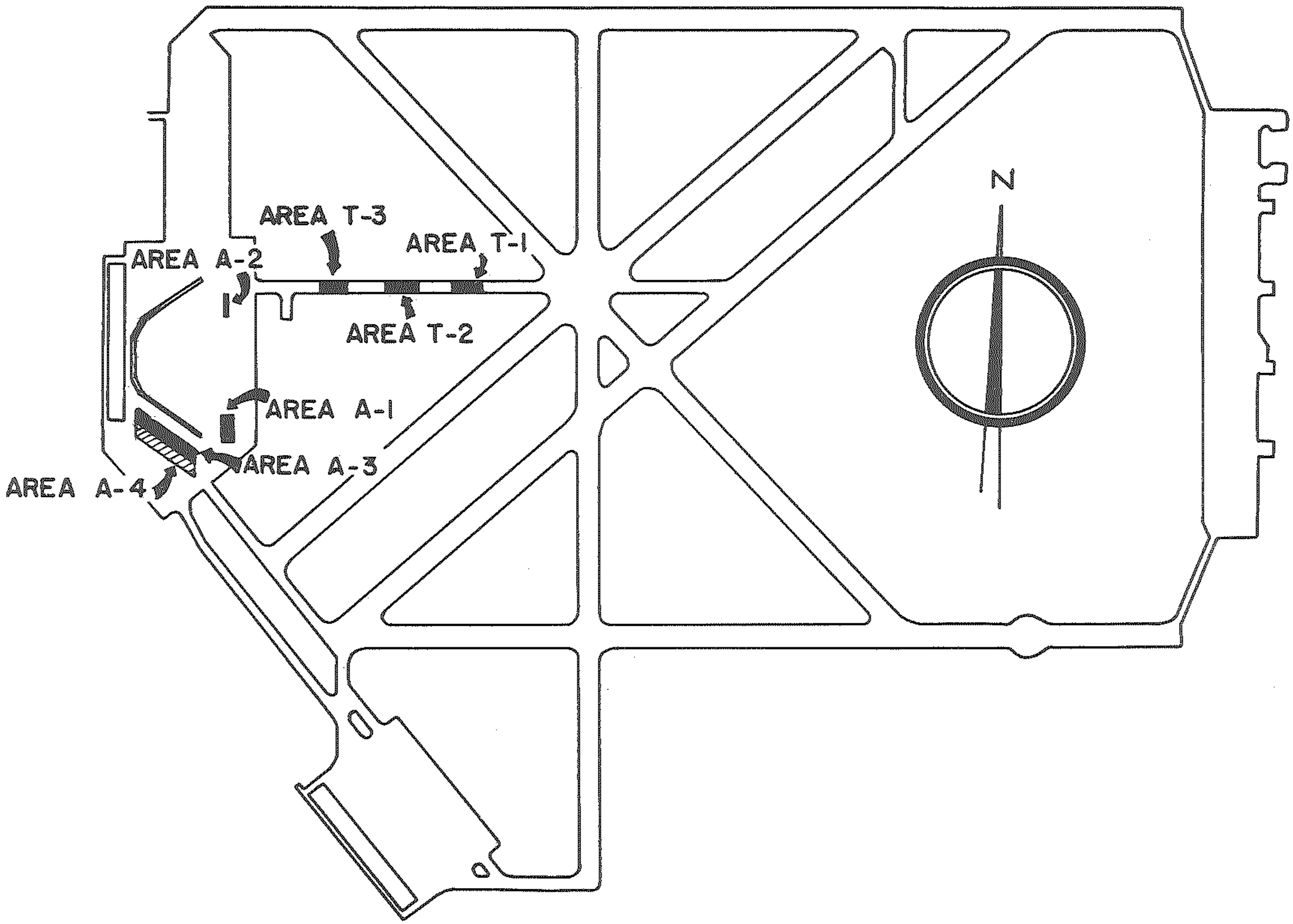
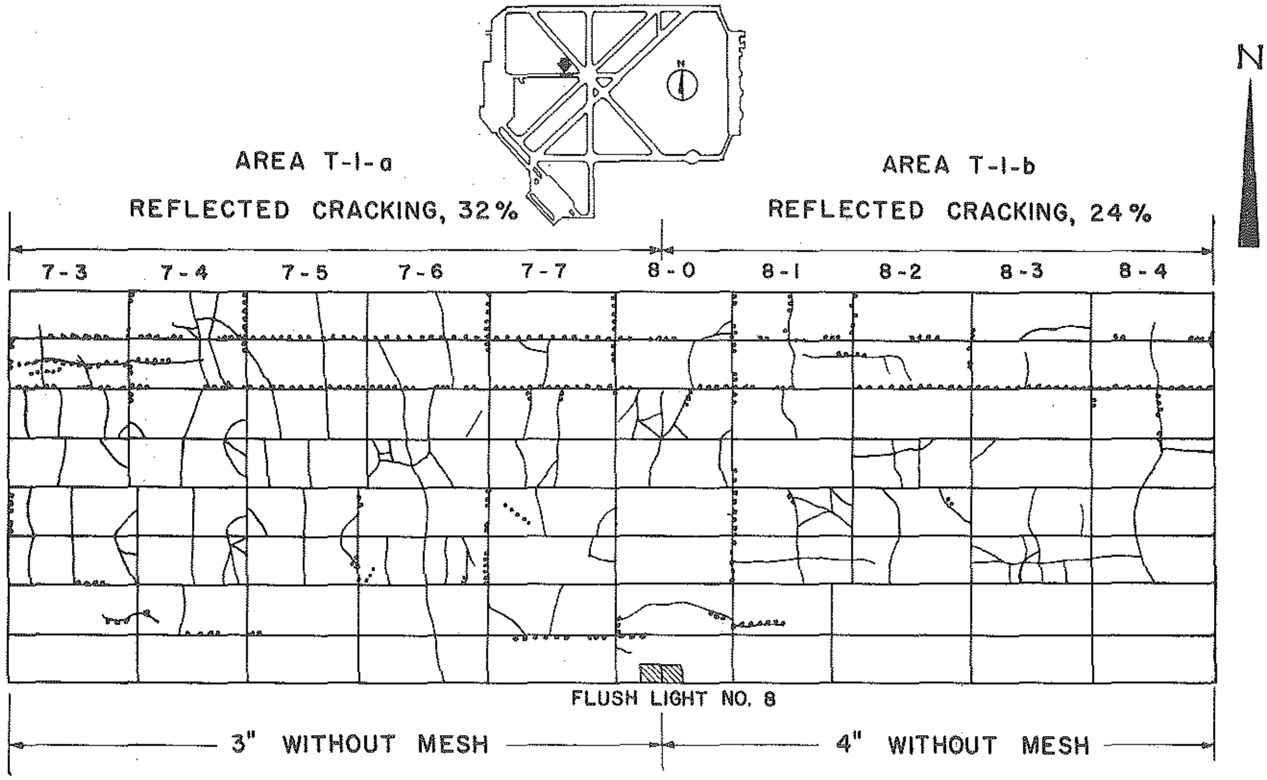
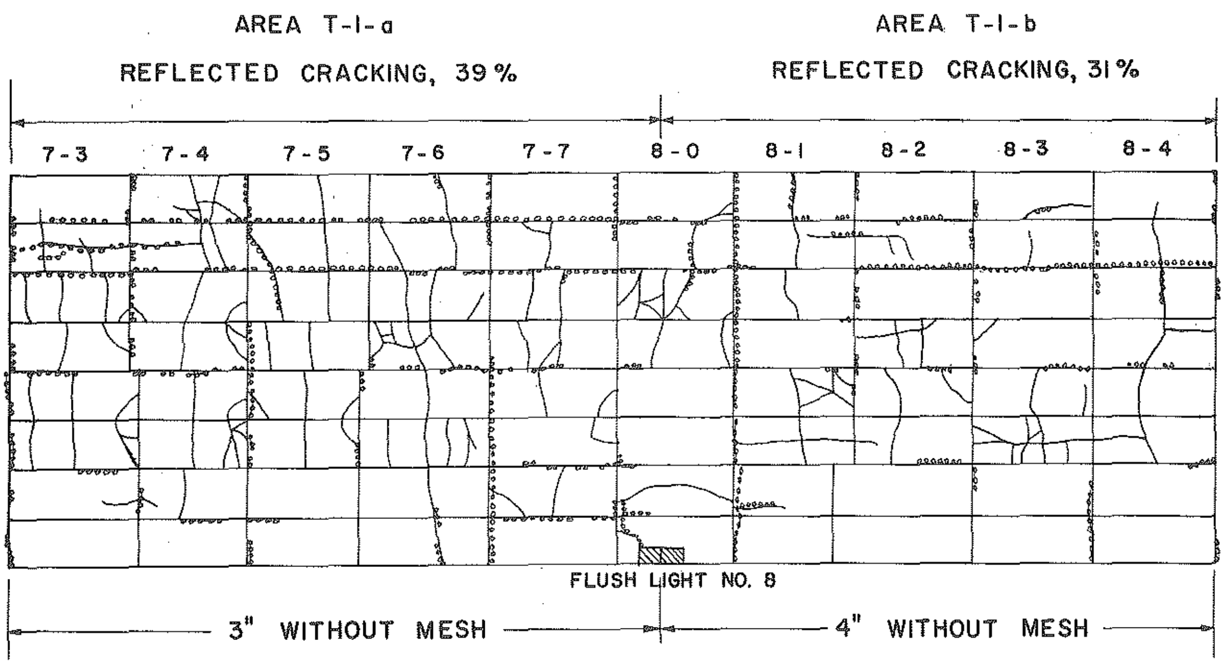


FIG. 6



SURVEYED ON 5-25-57
SURFACED IN 1955



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

FIG. 7

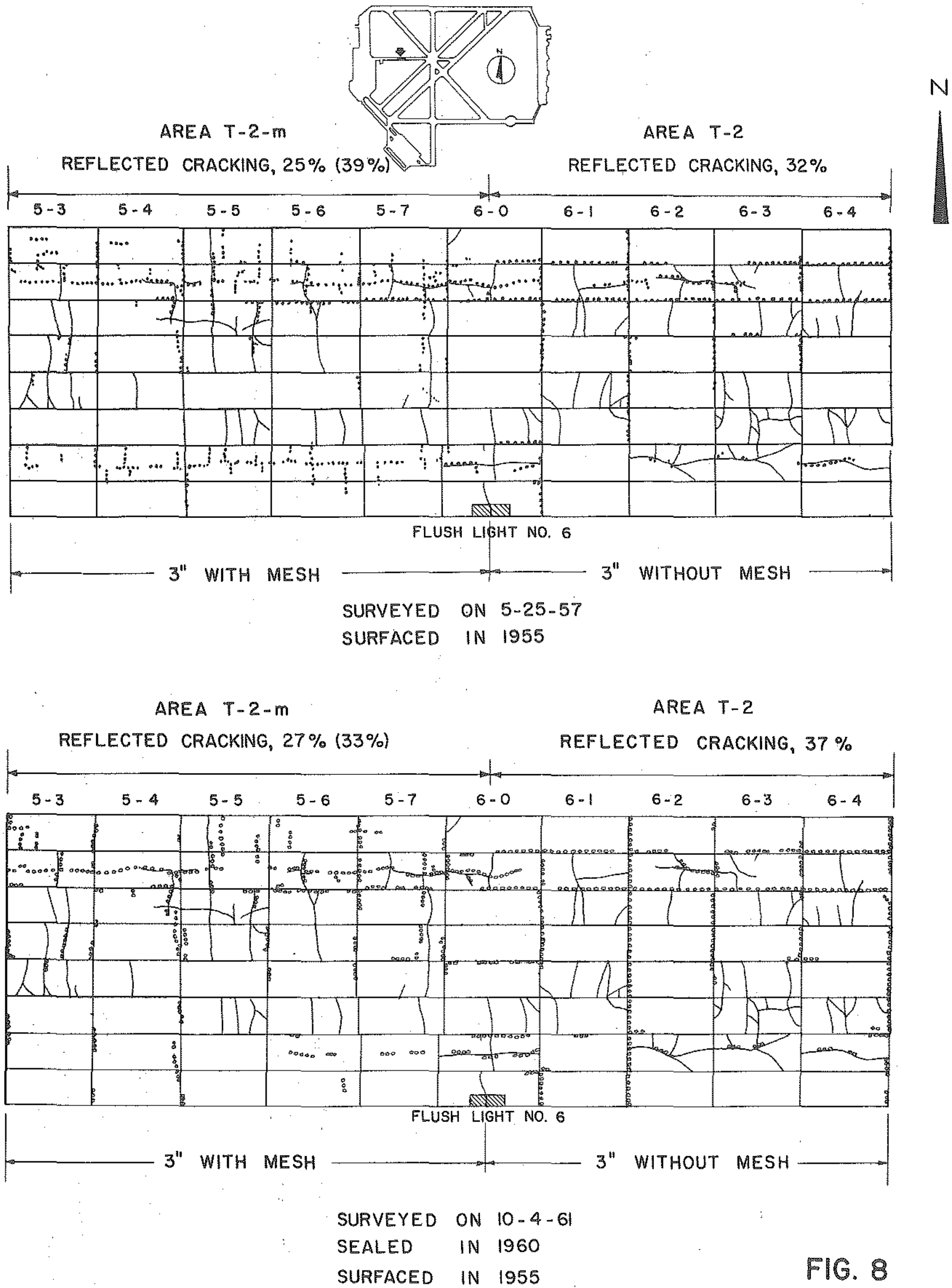
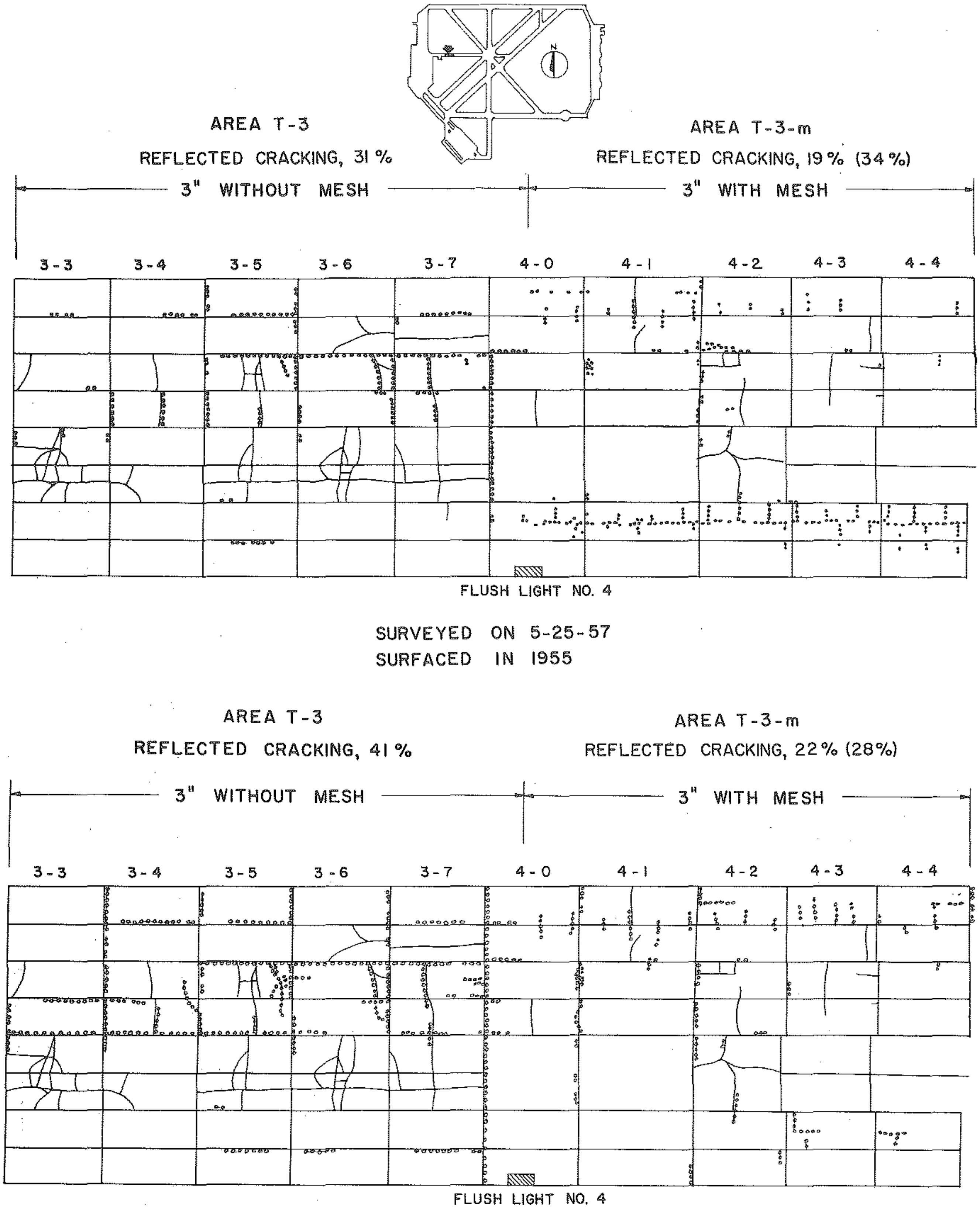

N
AREA T-2-m
REFLECTED CRACKING, 25% (39%)
AREA T-2
REFLECTED CRACKING, 32%
5-3 5-4 5-5 5-6 5-7 6-0 6-1 6-2 6-3 6-4
FLUSH LIGHT NO. 6
3" WITH MESH
3" WITHOUT MESH
SURVEYED ON 5-25-57
SURFACED IN 1955
AREA T-2-m
REFLECTED CRACKING, 27% (33%)
AREA T-2
REFLECTED CRACKING, 37 %
5-3 5-4 5-5 5-6 5-7 6-0 6-1 6-2 6-3 6-4
FLUSH LIGHT NO. 6
3" WITH MESH
3" WITHOUT MESH
SURVEYED ON 10-4-61
SEALED IN 1960
SURFACED IN 1955

FIG. 8



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

FIG. 9

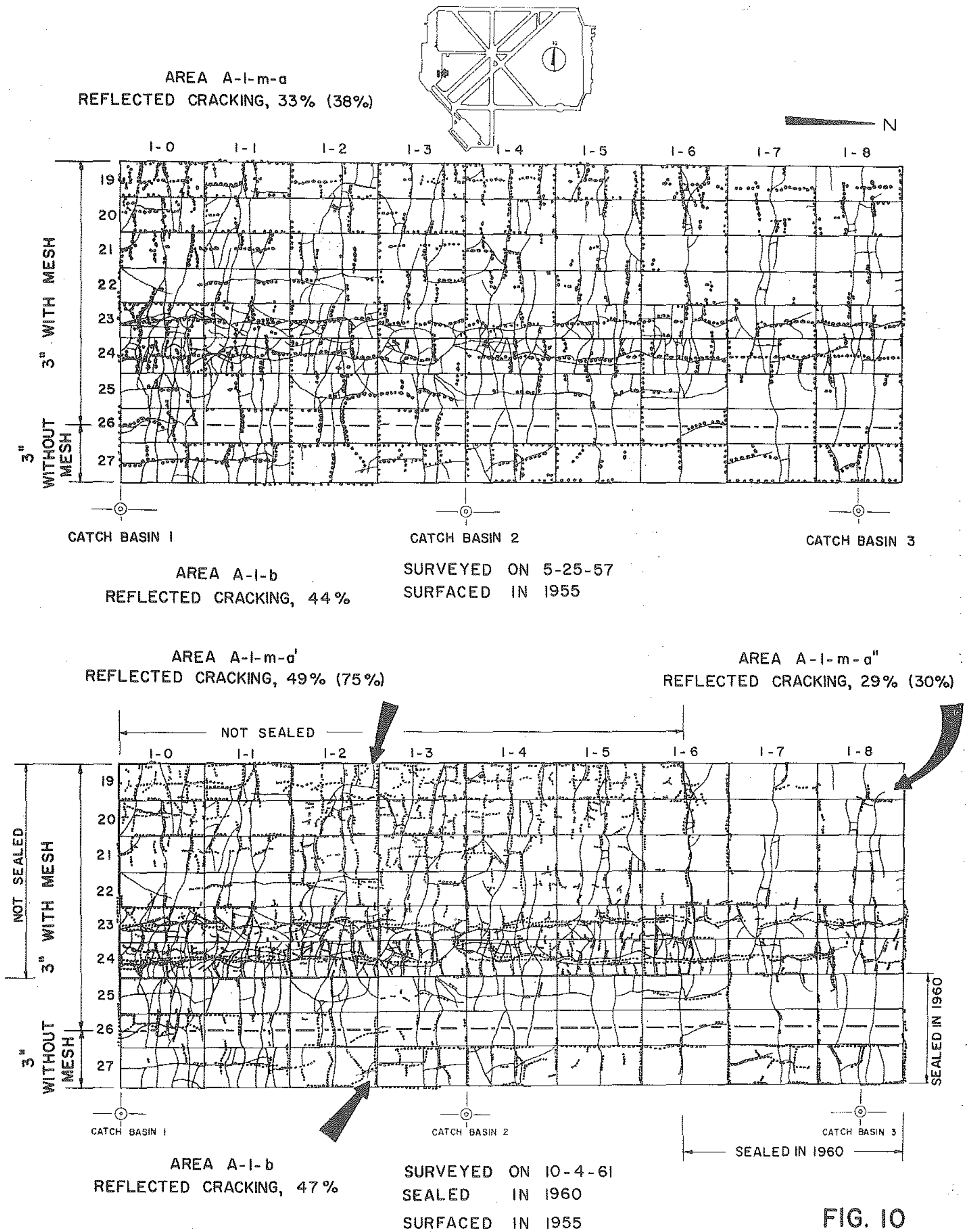

AREA A-I-m-a
REFLECTED CRACKING, 33% (38%)
N
I-0 I-1 I-2 I-3 I-4 I-5 I-6 I-7 I-8
19 20 21 22 23 24 25 26 27
3" WITH MESH
3" WITHOUT MESH
CATCH BASIN I
CATCH BASIN 2
CATCH BASIN 3
AREA A-I-b
REFLECTED CRACKING, 44%
SURVEYED ON 5-25-57
SURFACED IN 1955
AREA A-I-m-a'
REFLECTED CRACKING, 49% (75%)
AREA A-I-m-a"
REFLECTED CRACKING, 29% (30%)
NOT SEALED
I-0 I-1 I-2 I-3 I-4 I-5 I-6 I-7 I-8
19 20 21 22 23 24 25 26 27
NOT SEALED
3" WITH MESH
3" WITHOUT MESH
SEALED IN 1960
CATCH BASIN I
CATCH BASIN 2
CATCH BASIN 3
SEALED IN 1960
AREA A-I-b
REFLECTED CRACKING, 47%
SURVEYED ON 10-4-61
SEALED IN 1960
SURFACED IN 1955

FIG. 10

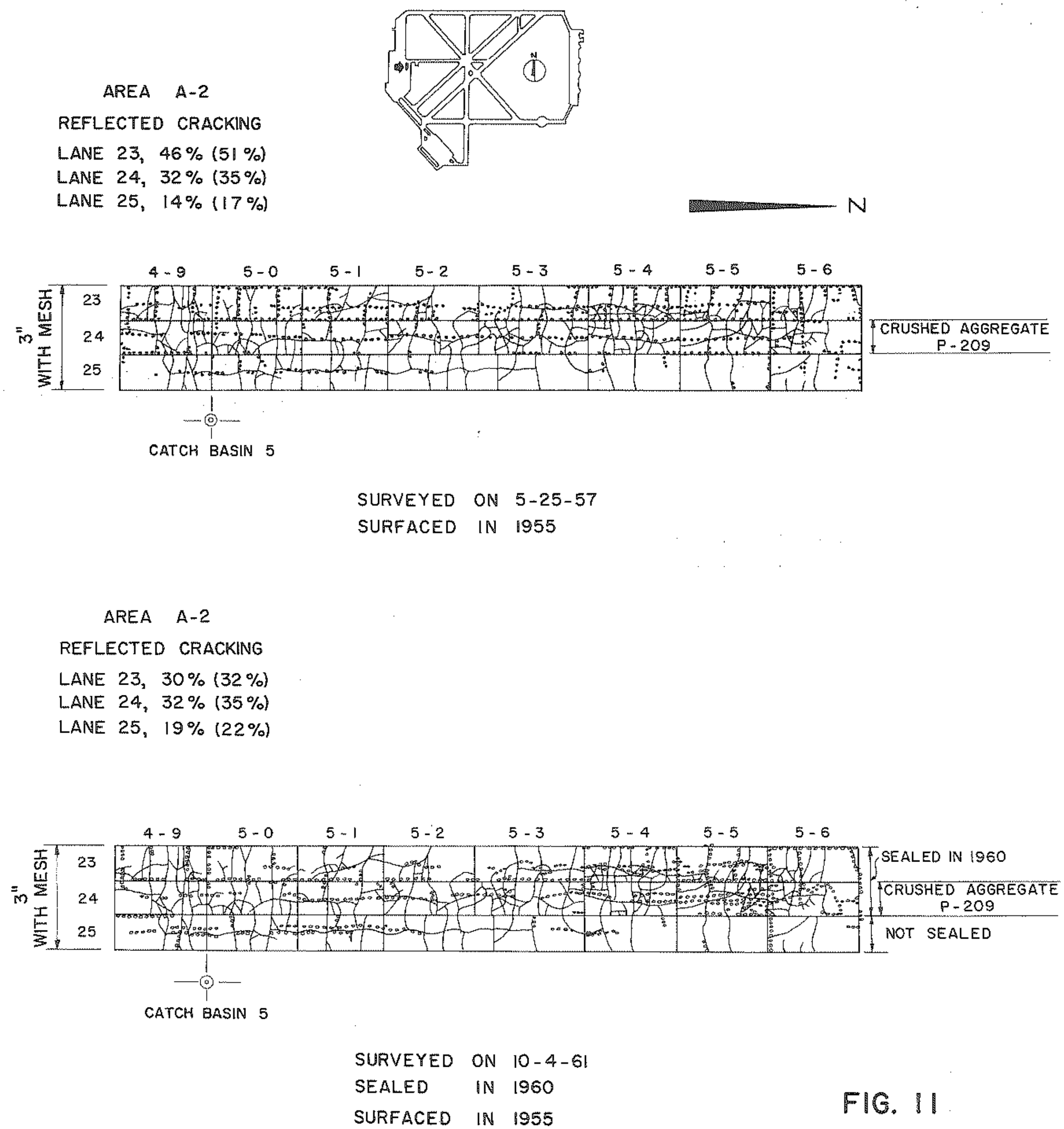
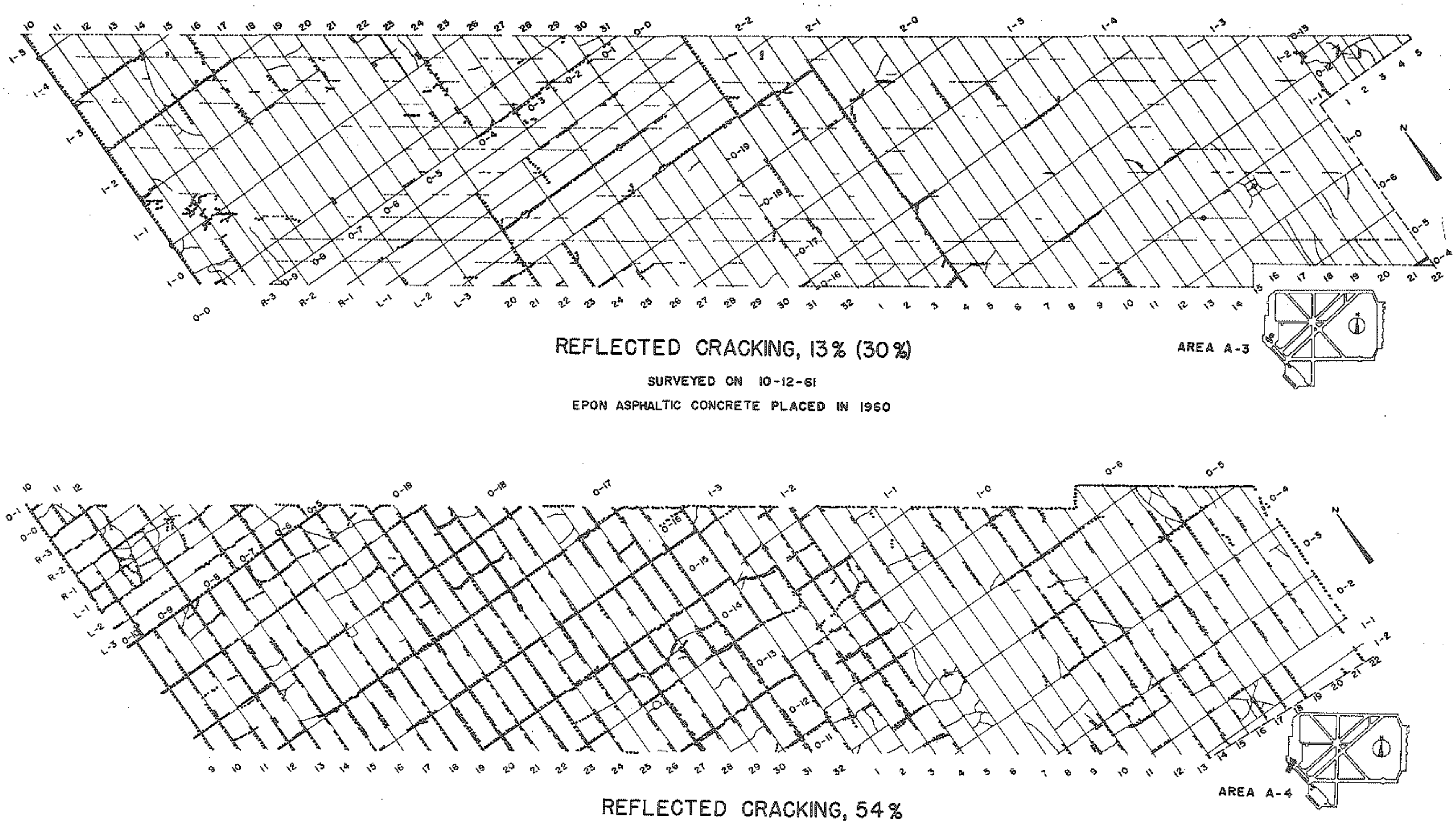

AREA A-2
REFLECTED CRACKING
LANE 23, 46% (51%)
LANE 24, 32% (35%)
LANE 25, 14% (17%)
N
N
4-9
5-0
5-1
5-2
5-3
5-4
5-5
5-6
3" WITH MESH
23
24
25
CRUSHED AGGREGATE P-209
CATCH BASIN 5
SURVEYED ON 5-25-57
SURFACED IN 1955
AREA A-2
REFLECTED CRACKING
LANE 23, 30% (32%)
LANE 24, 32% (35%)
LANE 25, 19% (22%)
4-9
5-0
5-1
5-2
5-3
5-4
5-5
5-6
3" WITH MESH
23
24
25
SEALED IN 1960
CRUSHED AGGREGATE P-209
NOT SEALED
CATCH BASIN 5
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

AREA A-3

REFLECTED CRACKING, 54%

SURVEYED ON 10-12-61

SURFACED IN 1960

AREA A-4

FIG. 12