HIGHWAY RESEARCH BOARD

Bulletin 322

Repair of Concrete Pavements

National Academy of Sciences—

National Research Council

publication 100

HIGHWAY RESEARCH BOARD

Officers and Members of the Executive Committee

1962

N.R.C. HIGHWAY RESEARCH BOARD

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Repair of Concrete Pavements

Presented at the

41st ANNUAL MEETING

January 8-12, 1962

National Academy of Sciences—
National Research Council
Washington, D.C.
1962

$1.20

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Welded Wire Fabric Reinforcement for Asphaltic Concrete

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This report evaluates the use of welded wire fabric reinforcement to alleviate rutting and/or shoving of pavement at intersections, and compares the effectiveness of extra thickness of asphaltic concrete overlays over portland cement concrete vs the use of welded wire reinforcement in the asphaltic concrete overlay to control reflection cracking in the asphaltic concrete surface. The advisability of breaking the old concrete pavement into small slabs is also analyzed. It is shown that the stability of the asphaltic concrete surface course has been increased by reinforcing with welded wire fabric reinforcement and that a considerable reduction in the amount of pavement distortion has resulted at the intersections investigated. It is also shown that, although welded wire fabric reinforcement has not demonstrated ability to prevent or reduce reflection cracking, breaking of the existing rigid pavement has prevented or at least retarded reflection cracking.

I. Welded Wire Fabric Reinforcement to Alleviate Rutting and/or Shoving of Pavements at Intersections

● ONE OF THE MOST perplexing pavement conditions that exists is the rutting and shoving of flexible pavements which often occurs at bus stops or other similar areas where buses and heavy trucks frequently brake. Such a condition is not only disturbing to the motoring public but requires costly maintenance.

In July 1956 an experimental paving section using a 6 x 3 - 10/10* welded wire fabric reinforcing in the surface course was constructed on an old viaduct in Jacksonville in an attempt to alleviate such pavement conditions. This section was in constant use for three years after its construction without apparent distortion while adjacent sections of the pavement continued to distort.

It appeared that the fabric in this experimental section had contributed to the stability of the asphaltic concrete surface course; but, inasmuch as no control sections were constructed of similar material without the fabric, there was no way to definitely state just how much effect the wire had on the stability of the pavement (1).

The results of this initial study led to a more complex experimental program on the use of welded wire fabric reinforcing in asphaltic concrete. Two city street intersections were selected on Kings Road (US 1) in Jacksonville as locations for the test sections of this program. These test sections were constructed in June 1959 during the resurfacing of Kings Road. Figure 1 shows a typical section of this roadway at these intersections. In one approach lane at each of the two intersections, 150 lineal feet of 6 x 3 - 10/10 welded wire fabric reinforcement was placed immediately below a $1\frac{1}{2}$-in. asphaltic concrete type I surface course (2). The fabric was placed on Kings Road in

*The size of welded wire fabric reinforcement is expressed by four dimensions: the spacing of the longitudinal wires (in inches), the spacing of the transverse wires (in inches), the gage (U.S. Steel Wire Gage) of the longitudinal wires, and the gage of the transverse wires (U.S. Steel Wire Gage).

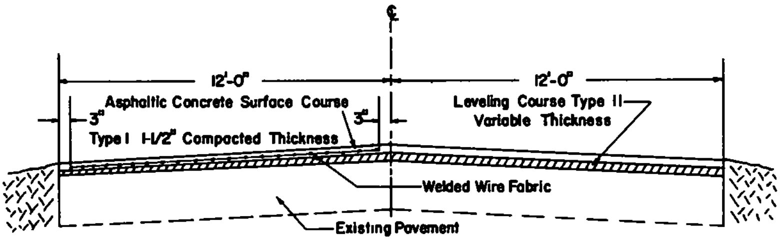

Figure 1. Typical transverse section of roadway at test sections.

the south approach lane at Tyler Street, and in the north approach lane at Pearce Street. The north approach lane at Tyler Street and the south approach lane at Pearce Street were used as control sections.

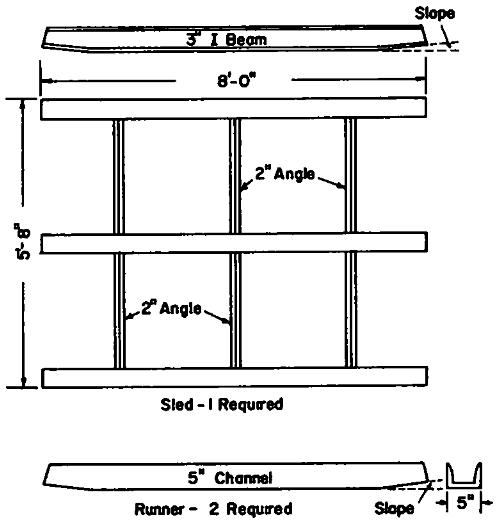

Figure 2. Hold-down device.

PROCEDURES

Construction procedures were essentially in accordance with the recommended practices offered by the Wire Reinforcement Institute (3) with the exception that the recommended 1¾-in. minimum overlay was reduced to 1½ in. because this is the maximum normal thickness used by the Florida State Road Department.

The fabric was delivered to the experimental test sites in rolls measuring 11 ft 6 in. in width and 150 ft in length. It was placed, with the transverse wires down, on the leveling course so that the 3-in. spaced wires were at right angles to the line of pavement, and the 6-in. spaced wires parallel to the centerline. A 1½-in. asphaltic concrete type I surface course was placed over the fabric with a Barber-Greene finishing machine.

A hold-down device was attached to the finishing machine to keep the fabric flat against the underlying pavement and to prevent any entanglement of the fabric with the spreader screw. The hold-down device consisted of runners on the outside of each "cat" track and a sled between the "cat" tracks (Fig. 2). The runners and sled were secured to the front end of the finisher by chains (Fig. 3).

Staples were used to secure the starting end of the fabric to the leveling course. The 1½-in. staples used were not completely satisfactory; however, the finisher was able to pave over the fabric at both intersections without incidence, and subsequent inspections of the pavement indicated that the poorly secured starting ends of the fabric were of little significance to this experiment.

Tension was applied to the trailing end of the fabric by means of J-hooks spaced 12 in. apart and a bridle fabricated from a piece of pipe and three sections of chain. This device is shown in Figure 4.

Tyler Street Intersection

Installation of the fabric was made first at the Tyler Street intersection. The fabric was delivered rolled with the transverse wires in (towards the center of roll). Because recommended procedures (3) for installation suggested that the transverse wires be placed down, the fabric could not be simply unrolled or the transverse wires would be up. Therefore, the roll of fabric was manually placed in the lane to be reinforced.

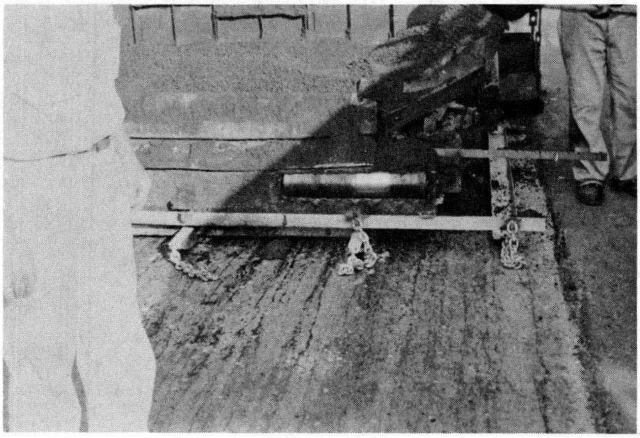

Figure 3. Hold-down device being positioned beneath finisher.

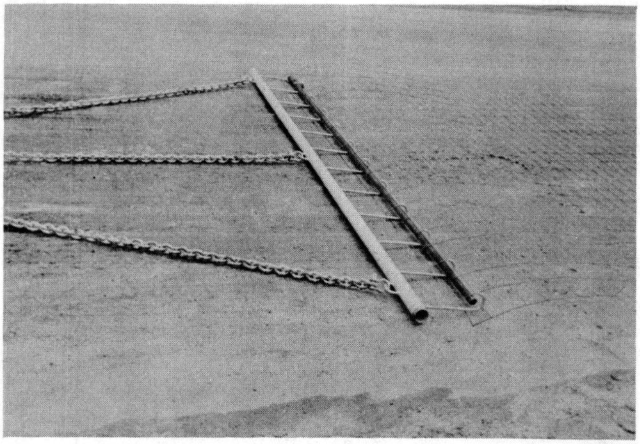

Figure 4. Bridle and J-hook device used for tensioning.

When the wire was properly positioned it was noted that the edge of fabric closest to the centerline was longer than the outside edge of fabric. This caused a series of humps or waves to develop in this longer edge. This wave action in the fabric is shown in Figure 5.

It was not possible to apply sufficient tension to make the fabric lay flat because the applied tension caused the unrolled fabric to curve due to the longer inside edge. The tension was released and paving began. While paving over the fabric, bulges that developed in front of the finisher were partially eliminated by the use of a crimping tool, as shown in Figure 6. When the wave action of the inside edge became excessive, the fabric was cut from the inside edge towards the outside edge and was cut only enough to permit it to lay flat by lapping the fabric at the cut.

During the initial rolling, wire fabric came through the surface course in several places along the longer edge of the fabric. The surface course material was removed in these areas, the fabric crimped or removed, and the surface course material replaced and rerolled.

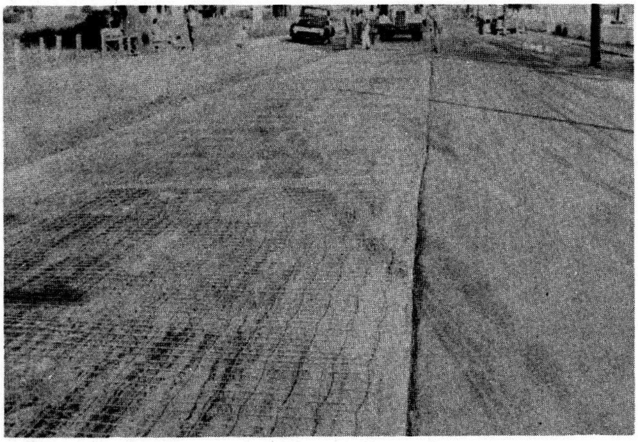

Figure 5. Fabric reinforcement in approach lane before placement of surface course. Note bulging along edge of fabric nearest centerline of pavement.

Figure 6. Using crimping tool to eliminate bulges in wire fabric.

Pearce Street Intersection

At the Pearce Street intersection the roll of fabric was placed on a spindle to facilitate its proper placement on the pavement. The fabric again had one edge longer than the other and, like the Tyler Street intersection, the longer edge was placed adjacent to the centerline of the pavement. The roll of fabric was cut into four smaller sections and the ends of these smaller sections were lapped with the trailing end of one section placed over the starting end of the next section. This permitted the fabric to lay flat against the pavement and prevented most of the bulging over the longer edge of the fabric similar to that which had occurred at the Tyler Street intersection. Only the initial starting end of the fabric was stapled to the pavement. The finishing machine was stopped about every 20 ft to allow proper crimping of the fabric that had bulged in front of it. More crimping was done in this section than at the Tyler Street intersection and no damaged pavement was found after rolling.

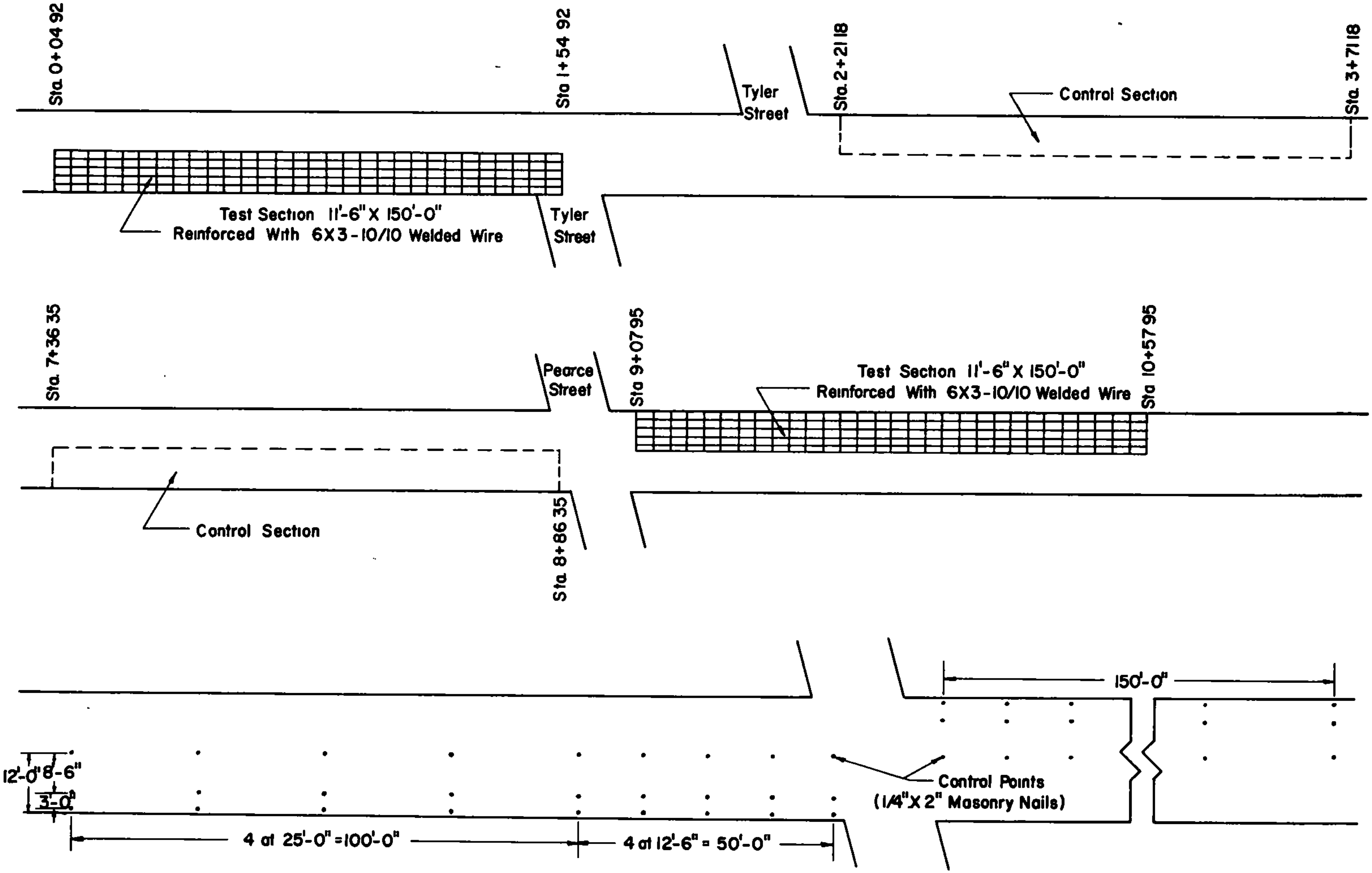

Figure 7. Plan view of test sections with typical layout of control points.

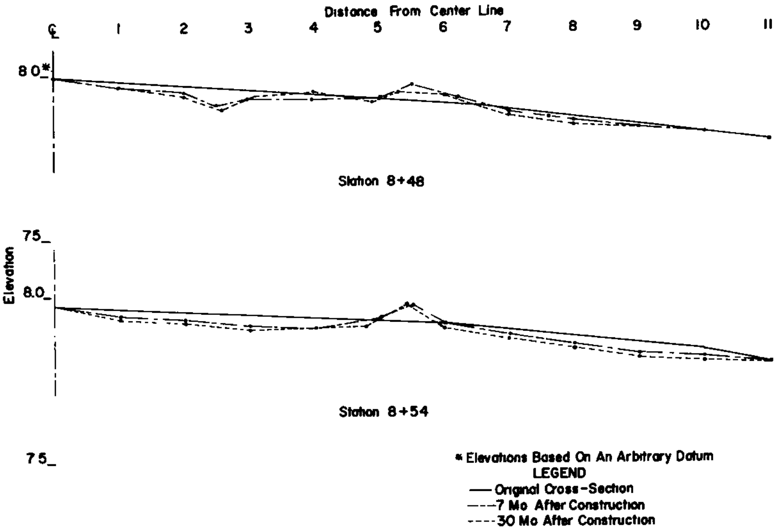

Figure 8. Distortion of Pearce Street section without reinforcement.

Controls for Evaluation Purposes

Immediately after construction special masonry nails were placed as control points in both approach lanes at each intersection in order to determine horizontal movement of the surface course. The nails were accurately spaced and aligned as shown in Figure 7.

Cross-sections were made at the stations where the nails were placed to measure the extent of any rutting that might develop. Elevations were recorded to the nearest 0.01 ft at 1-ft intervals across the width of the approach lanes.

RESULTS

Twenty-Four Hours After Construction

During the first 24 hours after construction, the pavement in the reinforced section at the Tyler Street intersection began to crack, leaving the fabric exposed along portions of the inside edge. A pattern of fine cracks, similar to the pattern of the fabric, was also noted in some areas. This was caused by the fabric having bulged under the surface course in areas, which resulted in a "springy" portion of pavement, and by the traffic creating excessive deflections. The greater part of the cracking, and all exposed wire was near the centerline where the longer edge of fabric had been placed during construction.

No indications of pavement cracking was found in the reinforced section at the Pearce Street intersection, but one small strand of wire was exposed.

One Week After Construction

Within one week after construction, cracking of the pavement was noticeable in both reinforced sections over the longer edge of the fabric. The pattern cracking in the "springy" areas at the Tyler Street intersection was more pronounced by this time and a probe into these areas revealed a void space between the surface course and the leveling course. This indicated that the fabric was not flat against the pavement and that the bridging effect of the fabric prevented the surface course from bonding to the leveling course.

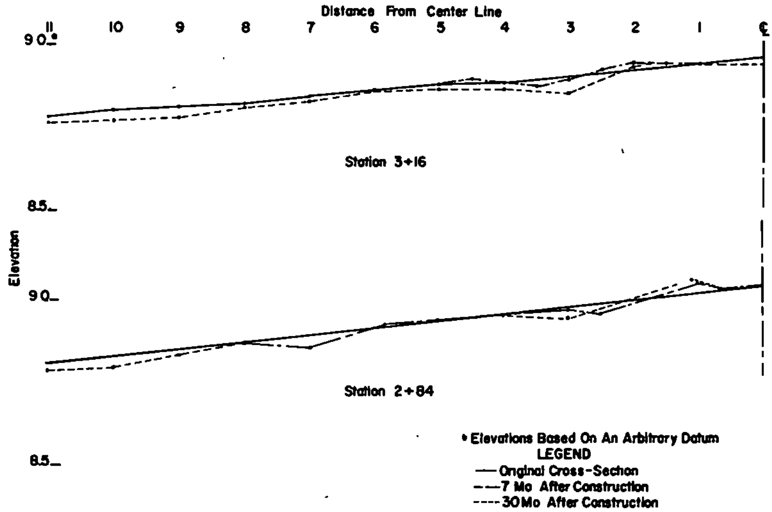

Figure 9. Distortion of Tyler Street section without reinforcement.

One Month After Construction

The longer edge of fabric caused both reinforced sections to continue to crack and repairs were necessary in these sections one month after construction. The Tyler Street section required patching of the pavement where the wire was protruding and/or the surface course had shattered. At the Pearce Street section only sand sealing of cracks in the pavement was needed.

While repairing the pavement, it was noticed that in some areas dust and water had seeped into the pavement through the cracks to the fabric, but in other areas some fine cracks which had been noted shortly after construction had disappeared, evidently closing under the kneading action of traffic.

Five and Seven Months After Construction

After five months of service no significant difference with respect to rutting or shoving could be found between the sections with the fabric and those without it. However, by the seventh month after construction severe pavement movement was noticeable in the sections without the fabric. Cross-sections were made at this time of the reinforced sections and the sections without the fabric. These cross-sections when compared with the original cross-sections indicated that excessive distortion of the pavement had occurred in some areas of the sections without the fabric, though no significant changes could be found in the sections with the fabric.

Thirty Months After Construction

Cross-sections were made again after 30 months of service. These cross-sections also indicated that no significant changes could be found in the reinforced section, but that the sections without the fabric continued to distort. Figures 8 and 9 show typical areas where excessive pavement distortion had occurred in the sections without the fabric.

Table 1 gives the total distortion for all sections after 7 and 30 months of service. The total distortion in the sections without the fabric was approximately twice that in the reinforced sections.

TABLE 1

VARIATION IN TRANSVERSE PROFILE

Street	Variation (in./in. x 10^{-3})			
	7 Months of Service		30 Months of Service	
	Reinforced	Nonreinforced	Reinforced	Nonreinforced
Tyler	5.35	7.88	5.89	9.25
Pearce	3.64	8.28	4.06	8.78

TABLE 2

HORIZONTAL DISPLACEMENT OF CONTROL POINTS IN OWP OF SURFACE COURSE

Distance from Intersection (ft)	Displacement[1] (in.)			
	Tyler Street		Pearce Street	
	Reinforced	Nonreinforced	Reinforced	Nonreinforced
0	-.156	-.156	-.188	-.250
25	0.000	-.312	-.156	-.094
50	-.125	0.000	-.250	-.250
75	-.312	0.000	-.188	-.312
100	0.000	-.375	-.250	-.063
125	-.188	0.000	-.125	-.375
150	-.250	-.312	-.063	-.063
Average	-.147	-.165	-.174	-.201

[1] A minus denotes that nails moved opposite from direction of traffic.

Movement of Control Points

The alignment of the outside wheelpath nails were measured during the fifth month following construction. These measurements, given in Table 2, indicated that the forces exerted on the pavement during the acceleration of vehicles are greater than the braking forces of the vehicles. This is explained by the fact that only the drive wheels accelerate the vehicle, while the braking force of a vehicle is distributed to the pavement by all wheels.

Subsequent measurements of these nails were made at various intervals of time during the 30 months of service following construction. These measurements showed negligible movement after 5 months of service.

SUMMARY AND CONCLUSIONS

The welded wire fabric reinforcement was furnished from the manufacturer with one edge longer than the other. This longer edge prevented the fabric from laying flat against the pavement and as a result construction was difficult and the rate of placing the surface course was reduced considerably. Cracking of the pavement usually occurred over this longer edge of the fabric. The fact that more pavement cracking occurred at the Tyler Street section than at the Pearce Street section can be attributed to the fact that more effort was made to alleviate the bulging of the longer edge of fabric within the Pearce Street section.

Although pavement cracking is undesirable, some fine cracks which were noted shortly after construction closed under the kneading action of traffic, indicating that all cracks are not necessarily damaging to the pavement. However, the seepage of water and dust into the cracked pavement prevented the possibility of other cracks closing.

Despite the undesirable pavement cracking in the sections with the fabric, these reinforced sections showed little distortion in comparison to the sections without the fabric. If, however, the welded wire fabric reinforcement is to be used successfully beneath a 1½-in. asphaltic concrete surface course, it must lay flat.

REFERENCES

1. Bransford, T.L., Research and In-Service Training Engineer, Florida State Road Department, Letter to Mr. A.C. Church (June 13, 1957).
2. "Standard Specifications for Roads and Bridge Construction, Section 233." Florida State Road Department (1959).
3. Howard, E.M., "Welded Wire Fabric Reinforcement in Asphaltic Concrete Overlays." ARBA Tech. Bull. 238 (1959).

II. Reflection Cracking in Asphaltic Concrete Overlays Placed on Old Portland Cement Concrete Pavements

● IN RECENT YEARS it has become necessary to widen and resurface many existing rigid pavements in Florida. This rehabilitation becomes necessary because of the inadequate width of pavements and/or the physical deterioration of the concrete. Widening of existing rigid pavements has been accomplished with both flexible and rigid types of widening strips. In some instances, these widening strips have been constructed entirely along one edge of the existing pavement and in other instances have been constructed adjacent to both edges of the existing pavement. After the widening strips have been constructed, a bituminous surface course is placed over the full width of the pavement to restore smoothness and improve the riding quality. This method of rehabilitating old rigid pavements has proven satisfactory and economical in most instances.

After a short period of time, however, a phenomenon known as "reflection cracking" usually occurs. These cracks develop directly over the joints and cracks in the underlying rigid pavement and also directly over the longitudinal joint between the widening strips and the old pavement.

Reflection cracking has been attributed to differential vertical or horizontal movements at joints, cracks, and pavement edges of the underlying rigid pavement (1). The horizontal movement of slabs is the result of thermal expansion and contraction of the concrete. Vertical movement may be caused by excessive deflection under loads (slab rocking), by differential settlement of adjacent slabs and widening strips, or by curling or warping of the slab due to temperature or moisture gradients within the slab.

Throughout Florida where bituminous overlays have been placed over existing rigid pavements, reflection cracking is almost always evident. These cracks produce an unsightly road, and widen and deepen with time so as to cause "thumping" for the motoring public. They also shorten the life of the overlay because water seepage through these cracks weakens the subgrade. The maintenance of pavements with these reflected cracks is a special problem.

A number of methods have been used by other states to prevent or reduce reflection cracking with varying degrees of success. One of the more recent methods reported to be most promising (2, 3, 4) involves the use of welded wire reinforcement in bituminous overlay. Such reinforcement is intended to eliminate cracking by distributing the stresses caused by the movement occurring at cracks and joints, so that the stress at any one point in the overlay is insufficient to cause the bituminous overlay material to crack.

A method that has been used in Florida for controlling deflection cracking is the breaking of the old pavement into relatively small pieces, with a maximum dimension of approximately 3 ft. The intent is to reduce the magnitude of joint movements by eliminating the cumulative effect of the larger slabs. However, no systematic follow-up of the performance of this procedure has been made.

Increasing the thickness of the bituminous overlay was also suggested as being effective in reducing the incidence of reflection cracking inasmuch as it is intuitively evident that a thicker overlay would offer more resistance to the forces introduced by any given magnitude of slab movement.

In the spring of 1959 the widening and resurfacing of a portland cement concrete pavement north of Lake City offered an opportunity to evaluate these methods of controlling reflection cracking. Advantage was taken of this opportunity to set up a research program to evaluate the effect that (a) welded wire fabric reinforcement, (b) breaking of the existing rigid pavement, and (c) increasing of the bituminous surface course thickness had on preventing or reducing the incidence of reflection cracking in asphaltic concrete overlays placed on an old portland cement concrete pavement.

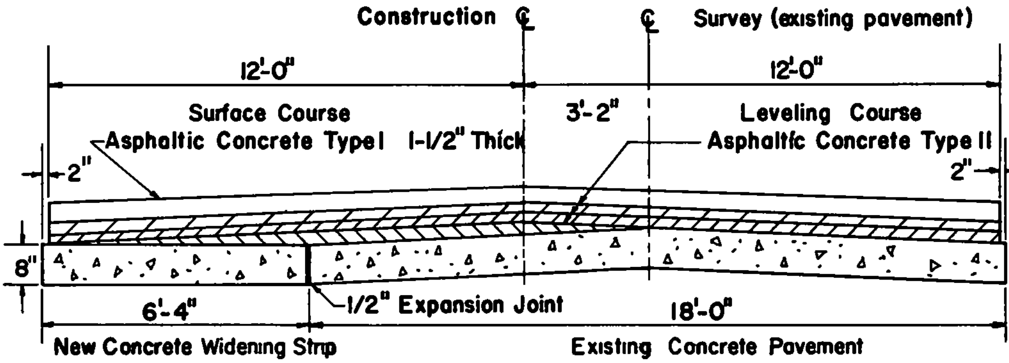

Figure 1. Typical transverse section of roadway at test section I.

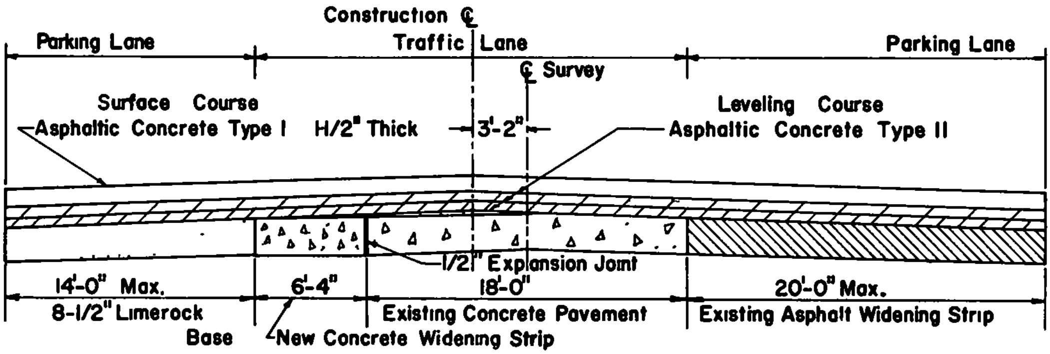

Figure 2. Typical transverse section of roadway at test section II.

SCOPE

Two test sections were selected north of Lake City on US 41 where an existing 18-ft portland cement concrete pavement was to be rehabilitated by the addition of widening strips of 8-in. plain concrete to provide a total width of 24 ft 4 in., a type II asphaltic concrete leveling course and a type I asphaltic concrete surface course.

Test Section I

The schedule for test section I included the placement of 200 ft of 3 x 6 - 10/10 welded wire fabric reinforcement placed continuously over the full width of the pavement, a 400-ft section where the existing pavement and the newly constructed widening strip were both broken, and a 200-ft section of pavement with an additional 1 in. of bituminous overlay. This increase in thickness was equivalent in cost per square yard to the use of the welded wire fabric. On the remainder of the project, the old concrete pavement was broken, except that the widening strip was left intact. Figure 1 shows a typical section of the roadway at this test section.

Test Section II

Test section II is located near a truck inspection station where, in addition to the 6-ft 4-in. rigid widening strip, two flexible widening strips (one on each side of the rigid pavement) were constructed to provide deceleration and storage lanes for trucks being inspected. Figure 2 shows a typical section of the roadway at this test section.

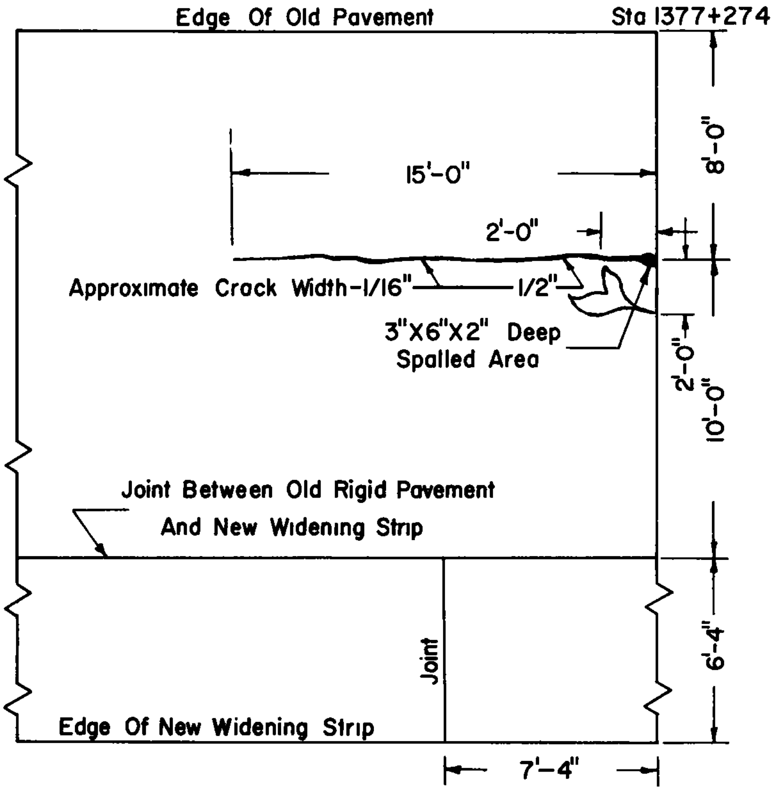

Figure 3. Plan view (upper) and photograph (lower) of typical section of pavement.

Two 250- by 5-ft sections of 3 x 6 - 10/10 welded wire fabric reinforcement were centered over the longitudinal joints of the widening strips, and 6 sheets of 6 x 3 - 10/10 welded wire fabric reinforcement were centered over selected transverse joints in the existing rigid pavement. The original pavement was not broken in this section.

Pavement Condition Survey

Before resurfacing, a detailed pavement condition survey was made of the existing concrete pavement and the new concrete widening strip in the two test sections. Included in this survey was the location and magnitude of all cracks, spalled area, joints, and patches. A photograph was also made of each slab included in this survey (Fig. 3).

PROCEDURE

All fabric was placed immediately below the 1½-in., type I asphaltic concrete surface course with the transverse wires down. The 6-in. spaced wires were placed at right angles to the centerline of the pavement and the 3-in. spaced wires were placed parallel to the centerline.

Construction procedures were in accordance with the recommended practices suggested by the Wire Reinforcement Institute (4) with the exceptions that (a) the recommended 1¾-in. minimum overlay was reduced to 1½ in. because this is the maximum normal thickness used by the Florida State Road Department; (b) the leading end of the welded wire fabric was not anchored in test section I due to construction difficulties; and (c) the leading ends of the sheets used in test section II were not all anchored.

A hold-down device was attached to the finishing machine to keep the fabric flat against the underlying pavement and to prevent any entanglement of the fabric with the spreader screw. Figure 2, Part I, of this report illustrates a hold-down device typical of the one used for this experiment. The hold-down device consisted of runners on the outside of each "cat" track and a sled between the "cat" tracks. The runner and sled were secured to the front end of the finisher by chains (see Fig. 3, Part I).

Tension was applied to the fabric by means of J-hooks spaced 12 in. apart and a bridle fabricated from a piece of pipe and three sections of chain (see Fig. 4, Part I). A crimping tool (see Fig. 6, Part I) was used to eliminate any bulging in the fabric which prevented it from laying flat against the underlying pavement.

Test Section I

The fabric used in test section I was delivered in rolls measuring 11 ft 9 in. in width and 100 ft in length. When the first roll of fabric was unrolled onto the pavement, it was noted that only the center of the fabric would lay flat and a series of humps developed along the edges. This resulted from both edges of the fabric being longer than the center portion. It was intended to place four rolls to reinforce both traffic lanes for 200 ft; but because of the difficulty in paving over the uneven fabric, this part of the experiment was discontinued after only 54 ft was placed in one lane.

An attempt was made to secure the starting end of the fabric to the leveling course with 1½-in. staples, but because the fabric had a tendency to "recoil," the staples would not hold to it. The "recoil" probably would not have occurred except for the fact that the fabric was rolled on a 10-in. mandril during the manufacturing process. Others have reported no difficulty with wire rolled in a similar manner but on a 20-in. mandril. No attempt was made to anchor the cut-off end of the fabric.

The existing pavement and new widening strip was broken between Station 1369+00 and Station 1372+89 by dropping a 2,700-lb weight approximately 8 to 10 ft. The resurfacing thickness was increased by 1 in. between Station 1372+89 and Station 1374+91 with the extra thickness being a type II asphaltic concrete placed with the leveling courses. A 100-ft transition between the standard thickness and the increased thickness was constructed at both ends of this section. The pavement was broken in the transition from Station 1371+89 to Station 1372+89 and this section is included in the evaluation of the effect that breaking the existing rigid pavement has on reflection cracking. The transition between Station 1374+89 and Station 1375+89 was not broken. A plan view of test sections is shown in Figure 4.

Test Section II

Rolls of fabric measuring 125 ft in length and 5 ft in width were used to reinforce the overlay over the longitudinal joints between the cement concrete widening strip and the

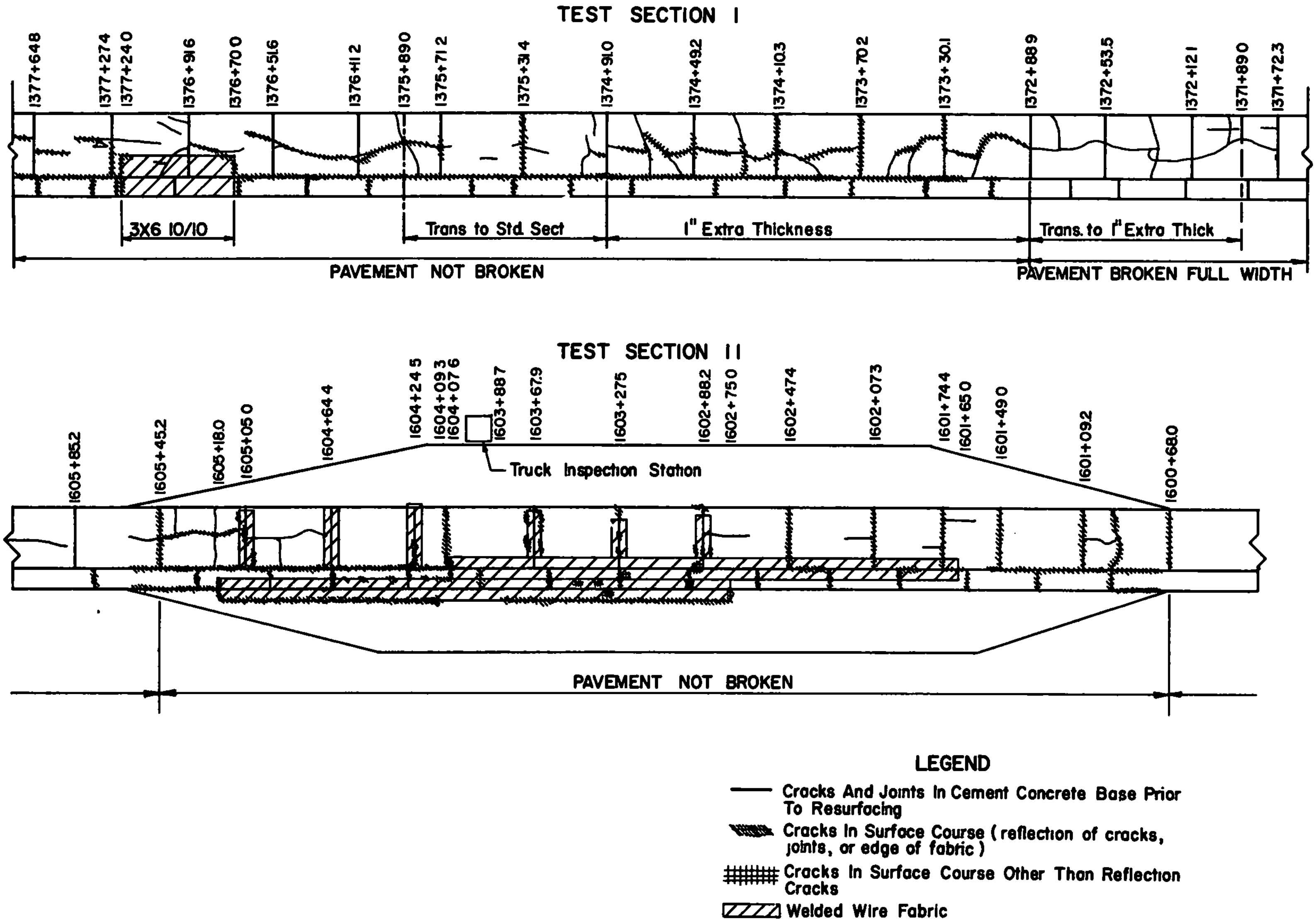

Figure 4. Plan view of test sections.

existing cement concrete pavement, and between the cement concrete widening strip and the limerock base storage lane. The fabric was rolled by the manufacturer with the transverse wires out (away from the center of roll). This allowed the transverse wires to be placed down by merely unrolling the fabric. The leading end of the fabric was secured to the pavement by setting anchors in the concrete and wiring these anchors to the fabric. Where the fabric was anchored over the limerock base, a 3- by 3-in. plate of 20-gage metal was placed over the fabric and a nail 8 in. in length and ¼ in. in diameter was driven through a hole in the plate and into the limerock base.

Two rolls were used at a time to reinforce a strip 250 ft long. As the fabric was unrolled, it would not lay flat due to bulges along one edge. Figure 5 shows the bulging of one strip of fabric and Figure 6 shows this same strip after the surface course had been placed over one-half of it and the bulges were removed by crimping. At first the adjoining ends of the 125-ft rolls were lapped with the trailing end of the first roll placed over the leading end of the second roll. The "recoil" tendency of the fabric caused the lapped end to push up through the fresh surface course as can be seen in the center of Figure 6. This was remedied by wiring the lapped ends of the rolls together.

The trailing ends of the two strips placed were not anchored during construction with the result that they pushed up through the surface course. On the day following construction they were anchored to the concrete base and the surface course repaired.

Six sheets of fabric 8 ft in width and varying in length from 13 to 17½ ft were centered over selected transverse joints and no trouble was encountered while paving over them because the sheets laid flat against the pavement. The leading edge of one sheet had to be secured to the pavement when several medium speed vehicles passed over it slightly distorting the leading edge. Staples of 1½ in. were used and worked very satisfactorily. These were the same staples used in the attempt to secure the roll of fabric placed in test section I indicating that anchoring is simplified when the fabric is in sheets, and there is no tendency to recoil. A plan view of test section II is shown in Figure 4.

RESULTS

Effect of Pavement Breaking

After 30 months of service, only a few cracks have occurred where the underlying concrete pavement was broken full width. These cracks do not appear to be related to the original cracks in the cement concrete pavement. Observations made throughout this entire project reveal that, where the cement concrete widening strip was not broken, practically all transverse joints within this widening strip have reflected through the bituminous concrete overlay, and no reflected cracking was noted where the existing pavement was broken. This indicates that reflection cracking has been controlled thus far by breaking the rigid pavement. However, it should be pointed out that, by breaking the existing rigid pavement, the structural value of the cement concrete base, as a rigid pavement, has been destroyed; unless the subbase is firm, there is the possibility of vertical movement between the fragments. Only time will tell if any such adverse effect will occur from breaking the concrete.

Effect of Increasing the Pavement Thickness

Figure 4 shows that the 1-in. extra pavement thickness has not retarded reflection cracking. This indicates that an extra pavement thickness of 1 in. is unwarranted for the purpose of controlling reflection cracking.

To further investigate the effect that thickness has on reflection cracking, 19 cores were taken from a bituminous concrete overlay in a section of roadway which had been widened and resurfaced about five years earlier. The cores were taken over the longitudinal joint between the old rigid pavement and the rigid widening strip from areas that had varying magnitudes of reflected cracks. The results of these cores showed that for an equal amount of overlay, cracks had occurred in some areas and had not occurred in other areas. This indicates that, if there is a relationship between overlay thickness and incidence of reflection cracking, it varies with the magnitude of displacement occurring at each joint or crack.

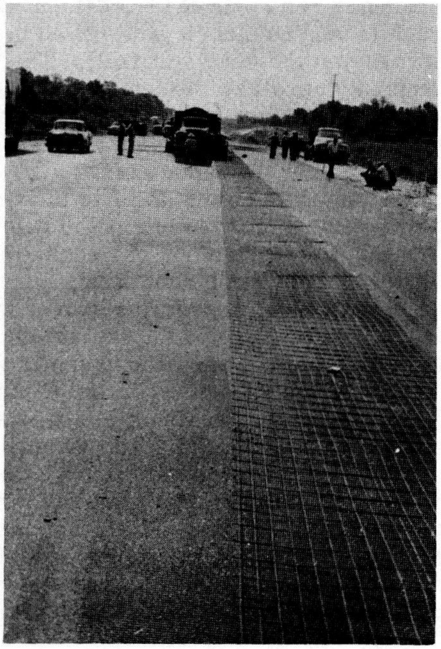

Figure 5. Strip of fabric reinforcement in proper position before placement of surface course; bulging along edge of fabric nearest edge of pavement.

Figure 6. Same strip of fabric (Fig. 5) after surface course has been placed over one-half of it. Practically all bulges have been removed by crimping; evidence of cracking in the fresh surface course seen near exposed fabric in foreground. Note end of strip protruding through the surface course in center of photograph.

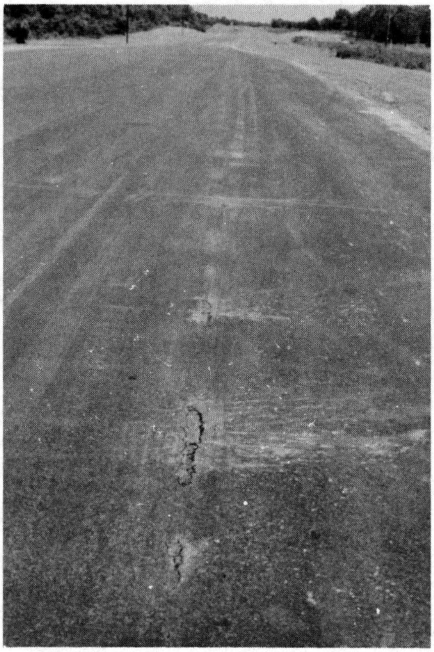

Figure 7. Same area as shown in Figures 5 and 6. A double crack over edge of fabric nearest outside edge of pavement has permitted water and dust to seep in; traffic passing over cracks has splashed water and dust onto surrounding pavement discoloring it.

Effect of Wire Reinforcing

The 54 ft of reinforced overlay in test section I has no reflected cracks within the area of reinforcing. However, Figure 4 shows that cracking has occurred over the edge of fabric in some areas. Despite the limited area that is reinforced, the fabric appears to be controlling or at least retarding the reflection cracking in this area. This is particularly true for the longitudinal joint between the widening strip and the old pavement. Also, the only transverse joint in the widening strip of test section I which has not reflected through the overlay is within this reinforced section.

The bituminous concrete overlay in test section II began cracking in the area of the strips within 24 hr after construction primarily because of the bulging of the longer edge of the fabric over the limerock base has cracked almost the full length of the strip (Fig. 7). This is the same strip of fabric shown in Figures 5 and 6, and Figure 6 shows that the exposed fabric has been crimped to lay flat. Figure 7 shows this reinforced area one week after construction and illustrates the cracking that had occurred along the outside edge of this strip of fabric at this time. Figure 7 also shows that water and dust is seeping into the cracked pavement.

In the area of the strips nearly all of the transverse joints of the new cement concrete widening strip have reflected through the overlay. The longitudinal joint between the old rigid pavement and new widening strip has also reflected through the overlay in some areas. However, the strips appear to have reduced the reflection of the longitudinal joint between the old pavement and new widening strip. The sheets have caused cracking in the overlay primarily over their edges and evidence of cracks over the old transverse joints has been noted.

TABLE 1

TOTAL REFLECTION OF CRACKS AND JOINTS AFTER 30 MONTHS OF SERVICE, TEST SECTION I (Reinforced)[1]

Width of Crack (in.)	Transverse: Transverse Joints (ft)		Transverse: Widening Strip Joints (ft)		Transverse: Original Pavement (ft)		Transverse: Total (ft)		Transverse: Reflected Cracks (%)	Longitudinal: Joint Betw. Conc. Pvt. & Widening Strip (ft)		Longitudinal: Original Pavement (ft)		Longitudinal: Total (ft)		Longitudinal: Reflected Cracks (%)
	Orig.[2]	Ref.[3]	Orig.	Ref.	Orig.	Ref.	Orig.	Ref.		Orig.	Ref.	Orig.	Ref.	Orig.	Ref.	
<1/16	-	-	-	-	4.0	0.0	4.0	0.0	0.0	-	-	-	-	-	-	-
1/16	-	-	-	-	5.0	0.0	5.0	0.0	0.0	-	-	5.5	0.0	5.5	0.0	0.0
1/8	-	-	-	-	-	-	-	-	-	-	-	9.0	0.0	9.0	0.0	0.0
3/16	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1/4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
5/16	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
3/8	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1/2	-	-	6.3	0.0	-	-	6.3	0.0	0.0	49.0	0.0	-	-	49.0	0.0	0.0
3/4	5.2	0.0	-	-	-	-	5.2	0.0	0.0	-	-	-	-	-	-	-

[1]Twenty ft of induced cracking occurred over the edge of the wire fabric, representing 15.2 percent of the perimeter of the wire.
[2]Total number of lineal feet of joint or crack in original portland cement concrete pavement.
[3]Total number of lineal feet of cracks reflected through the bituminous overlay.

TABLE 2

TOTAL REFLECTION OF CRACKS AND JOINTS AFTER 30 MONTHS OF SERVICE, TEST SECTION I (Nonreinforced)

Width of Crack (in.)	Transverse: Transverse Joints (ft)		Transverse: Widening Strip Joints (ft)		Transverse: Original Pavement (ft)		Transverse: Total (ft)		Transverse: Reflected Cracks (%)	Longitudinal: Joint Betw. Conc. Pvt. & Widening Strip (ft)		Longitudinal: Original Pavement (ft)		Longitudinal: Total (ft)		Longitudinal: Reflected Cracks (%)
	Orig.[1]	Ref.[2]	Orig.	Ref.	Orig.	Ref.	Orig.	Ref.		Orig.	Ref.	Orig.	Ref.	Orig.	Ref.	
<1/16	-	-	-	-	4.0	0.0	4.0	0.0	0.0	-	-	81.7	11.5	81.7	11.5	14.1
1/16	-	-	-	-	35.9	0.0	35.9	0.0	0.0	-	-	186.8	143.0	186.8	143.0	76.6
1/8	-	-	-	-	72.5	6.0	72.5	6.0	8.2	-	-	156.0	129.0	156.0	129.0	82.7
3/16	-	-	-	-	13.5	2.5	13.5	2.5	18.5	-	-	95.1	69.5	95.1	69.5	73.1
1/4	-	-	-	-	27.6	6.0	27.6	6.0	21.7	-	-	132.8	51.0	132.8	51.0	38.4
5/16	-	-	-	-	-	-	-	-	-	-	-	5.5	5.5	5.5	5.5	100.0
3/8	-	-	-	-	-	-	-	-	-	-	-	27.8	22.8	27.8	22.8	82.0
1/2	-	-	139.0	135.0	-	-	139.0	135.0	97.1	687.0	487.5	21.0	15.0	708.0	502.3	70.9
3/4	342.0	102.0	-	-	-	-	342.0	102.0	29.8	-	-	7.5	7.5	7.5	7.5	100.0

[1]Total number of lineal feet of joint or crack in original portland cement concrete pavement.
[2]Total number of lineal feet of cracks reflected through the bituminous overlay.

SUMMARY AND CONCLUSIONS

Welded wire fabric reinforcement, as used in this project, has been partially successful in controlling reflection cracking. The larger reinforced area of test section I has, after 30 months, prevented longitudinal and transverse joints from reflecting through the overlay. The sheet reinforcement used in test section II has been effective in reducing the reflection of transverse joints. The strip reinforcement has been effec-

TABLE 3

TOTAL REFLECTION OF CRACKS AND JOINTS AFTER 30 MONTHS OF SERVICE, TEST SECTION II (Reinforced)[1]

Width of Crack (in.)	Transverse: Transverse Joints (ft)		Widening Strip Joints (ft)		Original Pavement (ft)		Total (ft)		Reflected Cracks (%)	Longitudinal: Joint Betw. Conc. Pvt. & Widening Strip (ft)		Original Pavement (ft)		Total (ft)		Reflected Cracks (%)
	Orig.[2]	Ref.[3]	Orig.	Ref.	Orig.	Ref.	Orig.	Ref.		Orig.	Ref.	Orig.	Ref.	Orig.	Ref.	
<1/16	-	-	-	-	-	-	-	-	-	-	-	7.5	0.0	7.5	0.0	0.0
1/16	-	-	-	-	-	-	-	-	-	-	-	12.0	0.0	12.0	0.0	0.0
1/8	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1/4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1/2	-	-	40.2	27.6	-	-	40.2	27.6	68.7	484.0	22.0	-	-	484.0	22.0	4.5[4]
3/4	107.3	18.0	-	-	-	-	107.3	18.0	16.8	-	-	-	-	-	-	-

[1] 291.0 feet of induced cracking occurred over the edges of the wire fabric, representing 22.3 percent of the perimeter of the wire.
[2] Total number of lineal feet of joints or cracks in original portland cement concrete pavement.
[3] Total number of lineal feet of cracks reflected through the bituminous overlay.
[4] 9.1 percent of reflection of longitudinal joint between old rigid pavement and new portland cement concrete widening strip.

TABLE 4

TOTAL REFLECTION OF CRACKS AND JOINTS AFTER 30 MONTHS OF SERVICE, TEST SECTION II (Nonreinforced)

Width of Crack (in.)	Transverse: Transverse Joints (ft)		Widening Strip Joints (ft)		Original Pavement (ft)		Total (ft)		Reflected Cracks (%)	Longitudinal: Joint Betw. Conc. Pvt. & Widening Strip (ft)		Original Pavement (ft)		Total (ft)		Reflected Cracks (%)
	Orig.[1]	Ref.[2]	Orig.	Ref.	Orig.	Ref.	Orig.	Ref		Orig.	Ref.	Orig.	Ref.	Orig.	Ref.	
<1/16	-	-	-	-	31.5	0.0	31.5	0.0	0.0	-	-	100.8	21.0	100.8	21.0	20.8
1/16	-	-	-	-	14 0	8.0	14.0	8.0	57.1	-	-	30.5	0.0	30.5	0.0	0.0
1/8	-	-	-	-	21.0	10.0	21.0	10.0	47.6	-	-	13.5	0.0	13.5	0.0	0.0
1/4	-	-	-	-	-	-	-	-	-	-	-	13.0	13.0	13.0	13.0	100.0
1/2	-	-	60.6	57 9	-	-	60.6	57.9	95.5	-	-	-	-	-	-	-
3/4	172.5	136.5	-	-	-	-	172.5	136 5	79.1	555.9	260.0	-	-	555.9	260.0	46.7

[1] Total number of lineal feet of joints or cracks in original portland cement concrete pavement.
[2] Total number of lineal feet of cracks reflected through the bituminous overlay.

tive in reducing the longitudinal joints from reflecting through the overlay, but has not prevented the transverse joints in the new widening strip from reflecting through the overlay.

Placement of the sheets was less difficult than the placement of the rolls of fabric. Also, the use of rolls cannot be considered practical unless they lay relatively flat against the pavement when first unrolled.

Increasing the bituminous overlay by 1 in. has not prevented or retarded reflection cracking.

Breaking of the existing rigid pavement has prevented the reflection of original cracks and joints. Only a few cracks were noted where the pavement was broken. In all cases these cracks were fine in magnitude and short in length.

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Crack Control Joints in Bituminous Overlays On Rigid Pavements

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This research study concerns a method of controlling reflection cracks in bituminous concrete overlays over the transverse joints of rigid pavements. The study was undertaken to determine whether sawing and sealing joints would extend the maintenance-free life of the overlay sufficiently to justify the additional cost.

The experimental sections have been under observation for three years. Results to date indicate a substantial extension of the maintenance-free life of the overlay and consequently a reduction in the annual cost per square yard of bituminous concrete overlays.

● CONNECTICUT greatly accelerated its road building program, as did most other states, spurred by the rapid rise in the use of automobiles following World War I. A substantial amount of the pavements constructed on main routes was of portland cement concrete. The advent of World War II brought about a forced reduction of the maintenance effort. Reduced maintenance during the war, the passage of time, and a further rapid increase in motor vehicle registration made the rehabilitation of these pre-war roads mandatory. The use of bituminous concrete in resurfacing and widening held best promise for large-scale salvaging of the old pavements at the least immediate cost.

It was soon learned that transverse and longitudinal joints in the concrete pavement reflected through the overlay, thus presenting a new maintenance problem. Unfortunately, repairs were not undertaken until spalling and raveling of the cracks had become quite severe. The repairing of the spalled areas became a continuing operation, eventually reaching significant proportions.

A great deal of thought was given to finding a solution for this problem. Some experimental reinforcing of the bituminous concrete across the joints was tried, however, the results were not encouraging.

In April 1958, A. L. Donnelly, at that time Director of Research, Connecticut State Highway Department, conferred with Egons Tons, Research Engineer, Massachusetts Institute of Technology, regarding published information of work towards solution of the problem of reflection cracks (1). Test installations on Route 1A in Walpole, Mass., were inspected. As a result of this meeting the Connecticut Highway Department decided to experiment with sawed joints in bituminous concrete overlays.

EXPERIMENTAL PROJECTS

Two overlay projects already scheduled for construction during the summer 1958 were selected for the experimental work.

Project 1

Location.—This project is on US 7 in the City of Norwalk. US 7 is the main north-south highway in the western part of the State. The project starts approximately 1,300 ft north of US 15, the Merritt Parkway, and extends northerly approximately 7,400 ft. The northerly portion of the project is the crack control test section, the southerly portion the control.

TABLE 1

BITUMINOUS CONCRETE MIX USED IN EXPERIMENTAL PROJECTS

Analysis	Design Mix	Job Mix
% Passing Sieve:		
⅞-in.	100	-
¾-in.	95	100
½-in.	90	93.1
⅜-in.	75	82.3
No. 4	53	56.7
No. 10	40	43.3
No. 20	29	30.7
No. 40	19	19.3
No. 80	12	8.9
No. 200	5	4.5
Bitumen (85-100 pen.) (%)	6.0	6.35
Agg. plus bitumen (%)	100.0	99.45
Temp. of mix (°F)	290	290

Original Pavement.—The overlaid pavement consists of 20-ft wide 8-in. thick reinforced concrete pavement built in 1926. The reinforcing consisted of ½-in. deformed marginal bars placed 4 and 10 in. from the edge of the slab near the top and 4 in. from the edge near the bottom of the slab. The outside corners contained two ½-in. diagonal bars, the inside corners one bar of the same dimension. One ft of gravel subbase was placed in earth and 2 ft in rock cuts. The slabs are 40 ft long and 10 ft wide with ½-in. expansion joints. Load transfer devices and longitudinal tie bars were not used at that time.

Traffic.—In 1958 the traffic volume was 14,000 ADT. Commercial vehicles accounted for 4 percent of the traffic. The 1960 traffic count was still 14,000 ADT with approximately the same percentage of trucks.

Overlay.—The overlay, consisting of two 1¼-in. courses of hot-laid bituminous concrete, was placed with a Barber-Greene paver and compacted by a 10-ton tandem roller. The average haul from plant to job site was 43 mi. The design mix as well as the job mix are given in Table 1.

Project 2

Location.—This project is on US 1 in the Town of East Haven. The crack control joint test section consists of the two westbound lanes of a divided highway extending from the Farm River Bridge westerly approximately 0.4 mi. The two eastbound lanes in the same area are the control.

Original Pavement.—This pavement was constructed in 1942 and is of portland cement concrete reinforced with fabric or bar mat at the rate of approximately 61 lb per 100 sq ft. Slab length is 75 ft 9 in. between expansion joints. Intermediate ¼-in. dummy or warping joints are spaced at 25 ft 3 in. Load transfer units at expansion joints and ½-in. longitudinal tie bars, 2 ft 6 in. long and spaced 2 ft 6 in. on centers, were used.

Traffic.—The 1958 traffic count was 10,000 ADT. Commercial vehicles accounted for 18 percent of the traffic. The opening of the Connecticut Turnpike in January 1958 undoubtedly changed the traffic pattern on this section of US 1. Although no current traffic counts are available, the count is believed to be substantially the same as in 1958.

Overlay.—The bituminous concrete overlay on this project was essentially identical with that on Project 1. Two-course construction was used, thickness being the same, and the material was furnished from the same source and placed by the same contractor. The average haul from the plant was 5.9 mi.

PROCEDURES

Project 1

As stated previously, the northerly half of the pavement was selected for the test installation, the southerly half was the control. The joints to be sawed were carefully referenced prior to paving, and references transferred to the overlay with paint on completion of the paving operations. The overlay was placed between July 15 and 19, 1958.

Initially, 92 joints, involving approximately 0.70 mi of pavement, were referenced. In the course of paving operations and as a result of vandalism, approximately one-third of the references were destroyed. Where references were lost, no attempt was made to form crack control joints, thus reducing the total number to 60 joints. Some references have been lost in later test projects, but not to the extent encountered on this project.

Sawing operations were scheduled for mid-September for two reasons: (a) vacation travel would be somewhat reduced and consequently less interference with traffic would be experienced and (b) the bituminous concrete would be cooler and less likely to foul the concrete saw. Unfortunately the sawing operation was delayed until October 20th, resulting in some reflection cracking before sawing. Sawing was completed on October 22.

The joints were sawed $^3/_8$ in. wide and $1^3/_4$ in. deep using diamond saws. Three blades, separated by 4-in. diameter spacers $^3/_{64}$ in. thick, were mounted on the saw shaft, the two outside blades cutting to a depth of $1^3/_4$ in., the center blade, which had essentially a clean-out function, to a depth of one inch. The sawed joint was cleaned by means of compressed air. No further cleaning was undertaken.

Sealing of the joints was scheduled to follow the sawing operation. Unfortunately, inclement weather delayed sealing until October 30. The sealing was completed on November 4. This delay in sealing apparently did not seriously affect the sealing operation or the performance of the sealer.

The sealer material was a hot rubber asphalt compound, conforming to Fed. Spec. SS-S-164, applied with a combination melter and applicator.

Project 2

The bituminous concrete overlay was placed between June 9 and 12, 1958. The experiment consisted of sawing and sealing 27 joints 28 ft long, in the westbound roadway. Due to conditions beyond control, sawing was delayed until October 14. Early October temperatures were below 32 F on several occasions prior to sawing, consequently some reflection cracking had occurred.

Sawing and sealing procedures were identical with those described for Project 1.

OBSERVATIONS

A thorough inspection of the overlay on Project 1, Norwalk, was made prior to sawing, and existing reflection cracks were noted. A further detailed inspection was made on December 16, 1958. Only 7 ft of cracks in addition to those observed prior to sawing were found. It is of interest to note that some cracks observed at the time of sawing showed a tendency to knit together, despite night temperatures at or below freezing. Figures 1 and 2 show controlled and uncontrolled joint cracking on December 3, 1958.

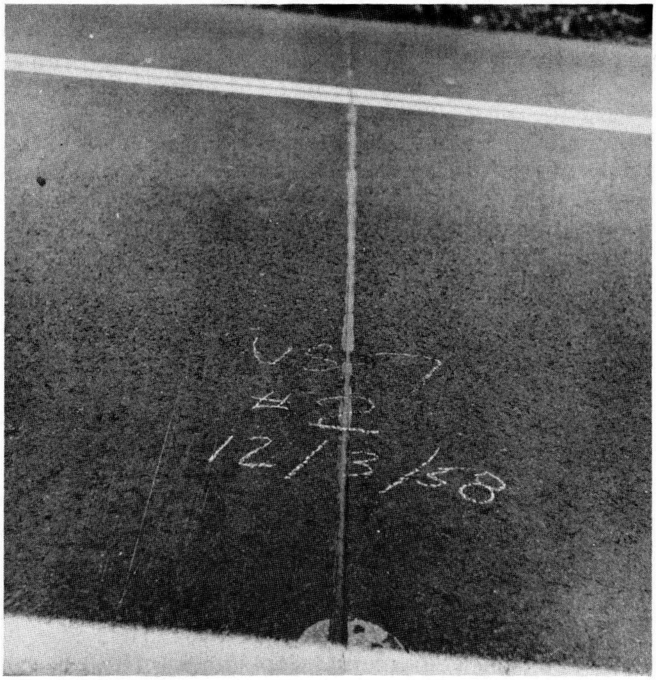

Figure 1. Controlled Joint Crack Project No. 1—Route US 7, Norwalk, December 3, 1958.

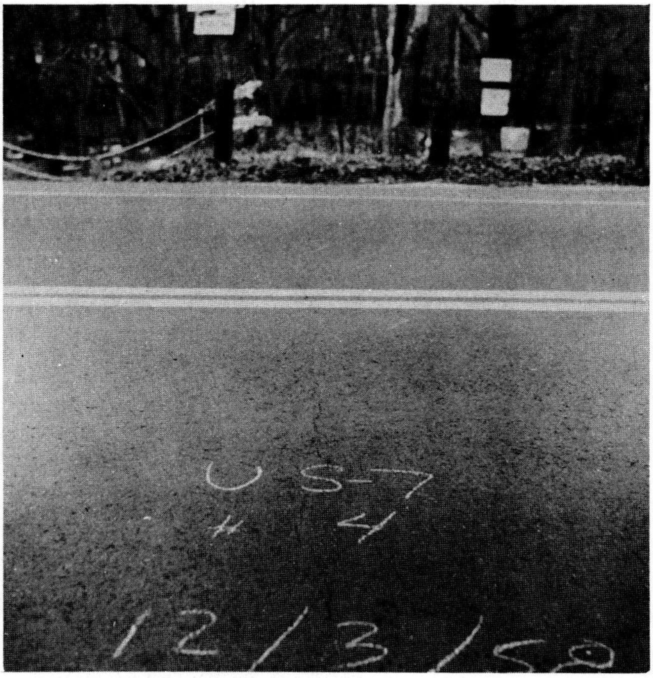

Figure 2. Uncontrolled Joint Crack Project No. 1—Route US 7, Norwalk, December 3, 1958.

TABLE 2

OBSERVED REFLECTION CRACKS AND ADHESIVE FAILURES

Date of Inspection	Project 1		Project 2	
	Reflection Crack[1] (ft)	Adhesive Failure[2] (ft)	Reflection Crack[1] (ft)	Adhesive Failure[2] (ft)
Oct. 1958	216		No obs.	No obs.
Dec. 16, 1958	223	Slight	No obs.	No obs.
Feb. 3, 5, 1960	-	-	65	359
Mar. 11, 1960	55	28	-	-
June 11, 19, 1960	41	17	-	-
July 6, 8, 1960	-	-	66	117
Mar. 13, 1961	58	1172	-	-
Mar. 16, 1961	-	-	52	743
Aug. 17, 22, 1961	4	813	-	-
Aug. 16, 1961	-	-	60	403

[1]Over transverse joint.
[2]Any failure $\frac{1}{4}$ in. or more in depth.

Project 2 in East Haven was not observed to the same extent as Project 1 during the sawing and sealing operations. Limited observations indicated a condition similar to Project 1.

Semiannual inspections have been made on Projects 1 and 2 starting during the winter of 1959-60. The joints still retain their original good appearance, and there is very little spalling or raveling except where cracking had occurred prior to sawing and sealing. As might be expected, a greater amount of adhesive failure of the sealer is detected during the winter inspections. Expansion of the underlying rigid pavement tends to close the joints in hot weather and this creates the impression that the sealer regains some adhesion. It is suspected, however, that this is at best a very temporary effect.

As expected, successive inspection reports indicate an increase in the amount of adhesive failure. For example, the February 1960 inspection stated "adhesive failure appears to be very narrow and very shallow," whereas the March 1961 inspection report stated "adhesive failure varied in depth from ¼ in. to over 1 in." The August 1961 inspection report noted a reduction in adhesive failure; the depth of the remaining adhesive failure, however, remained at ¼ in. to over 1 in. Table 2 gives the amount of reflection cracking and adhesive failure observed in each inspection.

Inasmuch as reflection cracking has occurred over 100 percent of the joints in the control sections, a tabulation thereof is omitted. Other research on the durability of bituminous concrete indicates age deterioration, or hardening, of the bituminous concrete to be much more advanced adjacent to cracks then in unbroken surfaces. For this reason the cracks in the control sections should have some attention now. Undoubtedly, routing and sealing of these cracks would cost at least as much as sawing and sealing at the initial stage.

Routing and sealing of open cracks over centerline joints has been tried on a limited experimental basis. It appears to be a satisfactory method of sealing such reflection cracks. Sawing and sealing of longitudinal cracks along the centerline and edges of the original pavement was not attempted on the two experimental projects described.

Figure 3. Controlled Joint Crack Project No. 1—Route US 7, Norwalk, May 12, 1960.

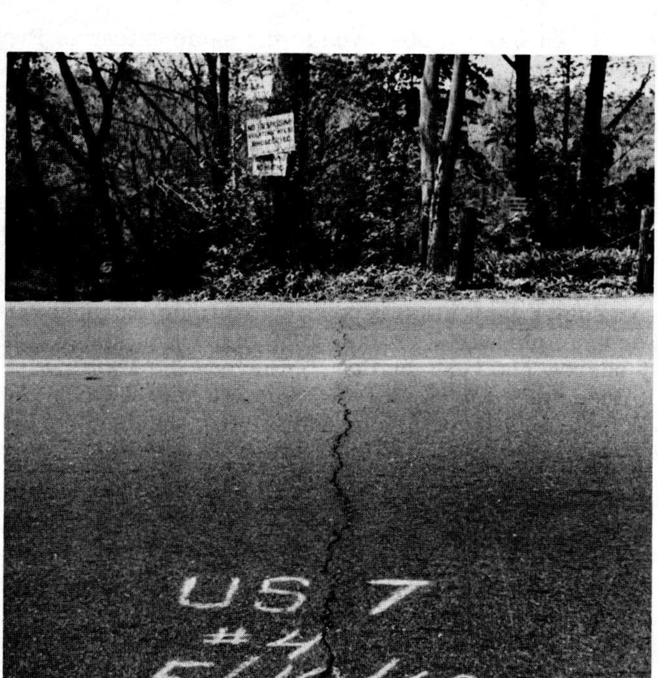

Figure 4. Uncontrolled Joint Crack Project No. 1—Route US 7, Norwalk, May 12, 1960.

Figure 5. Controlled Joint Crack Project No. 1—Route US 7, Norwalk, October 18, 1961.

ADDITIONAL EXPERIMENTAL WORK

The 1959-1960 winter inspections showed such a marked difference in the appearance of the test and control sections that three additional installations were scheduled for 1960 construction. Figures 3 and 4 show the condition in May 1960 of the joints shown earlier. Figures 5 and 6 show the joints in October 1961. The change in numbers reflects the identification now used for inspection of the joints. There is some doubt that Joint 28A is the same joint identified as No. 4 in previous figures.

The 1960 construction, however, involved some further experimentation. Reports on the work by Egons Tons (2) relating the joint shape to sealer performance raised a question as to the necessity of a $1\frac{3}{4}$-in. deep joint in a $2\frac{1}{2}$-in. overlay. Thus several different joint shapes were tried in the 1960 construction (Fig. 7). The purpose was to determine (a) the depth of cut required to insure that the controlled crack would occur over the joint in the original pavement, and (b) the effect of various joint shapes on the performance of the sealer.

These test installations were plagued by delays to a greater extent than the original test installations. Two of the three projects were paved in late September and mid-November, respectively. One project was paved in mid-July. Due to delays in sawing and sealing, all three projects developed reflection cracks prior to sawing, the most severe occurring in the late season projects.

Because it is too early to draw valid conclusions from the 1960 tests, it can only be stated that the $\frac{3}{8}$- by $\frac{1}{2}$-in. joint appears to control the location of the reflection crack

Figure 6. Uncontrolled Joint Crack Project No. 1—Route US 7, Norwalk, October 18, 1961.

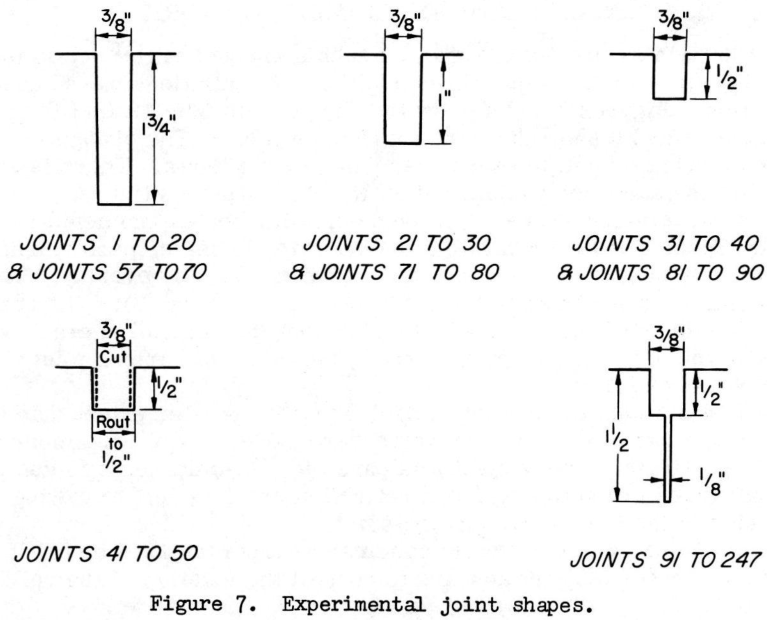

Figure 7. Experimental joint shapes.

as efficiently as the other shapes. Considerable variation in adhesive failure between replicate sections of the same joint shape raises the question of influences other than the joint shape. Figure 7 shows the system of replicate sections.

There is no information available on the experience elsewhere in controlling reflection cracks by sawing and sealing other than that reported by Egons Tons.

COST DATA

The installations to date have been experimental, consequently cost records are not conclusive. In 1961 there was one overlay project by contract in East Hartford that contained an item for sawing and sealing 10,500 ft of crack control joints. The contractor bid $0.45 per ft for sawing and sealing joints. The configuration of this joint is $^3/_8$ by $^1/_2$ in. with the center saw cutting to a depth of $1^1/_2$ in. On the basis of a 10- by 40-ft slab the additional cost is approximately $0.10 per sq yd.

CONCLUSIONS

Based on the admittedly limited experience, both as to extent and age of the experimental projects, and with the procedures used, the following conclusions are believed warranted:

1. Crack control joints are anticipated to provide from 5 to 10 years of maintenance-free service.
2. The $^3/_8$- by $^1/_2$-in. joint shape is considered adequate to control crack formation in a $2^1/_2$-in. overlay.
3. Further experimentation is needed to determine the required curing period for the overlay material to achieve most efficient sawing operation at various seasons of the year.
4. Relative efficiency of abrasive disks and diamond saws in forming crack control joints remains to be evaluated.
5. A need for experimentation with other sealers is indicated.

ACKNOWLEDGMENTS

This report would not have been possible without the invaluable assistance of Otto A. Strassenmeyer, Lawrence Miller, and Russell Dibble, who are among the author's associates in the Division of Research and Development, Connecticut State Highway Department.

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Subsealing of Concrete Pavements

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This paper concerns the experiences in District No. 4 of the New York State Department of Public Works with the salvage and restoration of old concrete pavements. These pavements were pumping badly and due to loss of subgrade support were consequently being weakened under the pounding of heavier and greater volumes of traffic. These experiences cover a period since 1948. Most of these pavements were constructed in the late 1920's and early 1930's and had been subjected to severe use. Also, they were not built to the present standards of foundation and subgrade construction, and were laid on a variety of soils prevalent in western New York. The methods used, their application, and the results obtained, are discussed.

The effect of this work, has been to restore the foundations of hundreds of miles of pavement and adapt them to present-day traffic through widening and serving as bases for asphaltic concrete overlays at an economical cost. The net result has been a consistent rehabilitation of a 1,400-mi highway system meeting present demands of traffic.

●DURING THE late 1920's and 1930's, the State of New York embarked on an ambitious program of highway construction, in order to meet the demands of traffic which had been steadily increasing since the end of World War I. Under this program a large mileage of concrete pavement was built, which has faithfully served to the present.

In most all cases, the original investment made in these pavements has paid handsome dividends to the public, by providing adequate roads and thereby being a major and important factor in the development of the economy to the highest level of all time.

However, the time has long since come and gone when these roads must be modernized to such standards that they can meet the requirements of greater volumes of traffic, heavier loads, and higher speeds. Call it the tyranny of the wheels, but it must be faced that the populace and economy have become increasingly dependent on the motor vehicle for the transportation of people and goods. As a result, the best possible means must be provided for that vehicle to use in going from one point to another. Even though these concrete pavements, built in the 1920's and 1930's, had more than paid off their original investments in the great majority of cases, it was believed possible to salvage these pavements and cause them to provide many more years of useful service. Also, where the terrain had been favorable so that major reconstruction to meet modern profile and alignment standards would not be necessary, these pavements, if salvaged by proper treatment, could be widened and resurfaced to give a modern highway. Thus, for a comparatively low cost and in a comparatively short time, a large percentage of the highway system could be rehabilitated. The major factor in effecting this rehabilitation was found to be the subsealing of old concrete pavements, and it has proved highly successful.

In 1948 N.Y. District No. 4, which has about 10 percent of the total State highway mileage (approximately 1,400 mi), had many miles of concrete pavements that were "pumping" very badly. This "pumping" was due to water penetrating the joints in the pavement and saturating the foundation soils. In five of the six counties of the District, the soils encountered in most areas are mixtures of clay, silt and loam.

When the roads were originally built, it was not the practice to place a blanket of granular material on the subgrade as is the standard today. Such a gravel blanket was thought an unwarranted expense as the concrete pavements, being rigid, would bridge over any weak areas. However, as the amount of traffic and the loads superimposed by trucks increased year after year, the poor underlying soils when saturated by water weakened the foundation, and "pumping" resulted. The process of "pumping" is merely the deflection of the pavement as loads pass over them, and because of poor subgrade support, water is forced upward through the joints and cracks and outward along the edges of the pavement. It can readily be seen that this is a process like a chain reaction which will ultimately destroy the pavement structure by breaking down the pavement slab.

This "pumping" was not just a theory but could be observed every time a loaded truck passed over a joint after a heavy rain. The water was actually forced out through the joints and cracks in a stream as thick as a lead pencil and spurting 6 to 8 ft into the air. It was obvious, where this occurred, that the slab was deflecting and broken, with the loose section jumping up and down.

Conditions became so critical that it was found necessary to do something immediately to correct these situations, otherwise substantial mileage of pavement would become a total loss. The remedy had to be quick and thorough as sufficient funds and time were not available to reconstruct these roads.

In the summer of 1948, under the general direction of the then incumbent District Engineer, A.R. Mulligan, and under the direct field supervision of the District Supervisor of Maintenance, Charles W. Donnelly, experimental work was carried on to find a remedy. A 20-ft wide concrete pavement, built in the 1920's, was selected. This road is on a heavily traveled truck route between the Cities of Rochester and Buffalo.

The pavement was in very bad condition, and after a rain the conditions of pumping described previously were prevalent. It was realized that unless something was done soon complete failure would result.

As the first step in the experimental work, it was decided to fill all joints and cracks with a light grade of asphalt emulsion using a large distributor to haul and furnish the material in large quantities at the site of work.

The material from the distributor was placed into large heating kettles from which maintenance personnel drew off the asphalt into pouring pots and filled the joints and cracks by hand. In addition other members of the crew filled the joints directly from the distributor by means of a ⅝-in. hose.

This operation, using 3 foremen and 14 men, was very crude and the method used, though necessary in order to place as much material as possible in a short time, was frankly an experiment in subsealing the pavement and restoring subgrade support.

After pouring was complete, it was found by inspection that asphalt had completely disappeared beneath the concrete slab. It was also found, during the operation of filling the joints with the hose directly from the distributor, that material would run into the joint for a period of 5 to 10 min.

A second pouring was started into the same joints, but investigation indicated that material again disappeared. It was necessary to repeat this pouring operation four times before material ceased to disappear. By this time pumping had stopped and the slabs had stopped "racking."

Between 1948 and 1958, this pavement had to be subsealed once or twice a year. In 1958 it was widened and resurfaced, and it is now one of the better pavements in the District, in excellent condition, and carrying very heavy truck traffic between two large cities.

As time went on, more efficient methods for subsealing pavements were developed as well as special equipment designed to make these new methods effective. For example, it was found that a more satisfactory subsealing could be obtained by pouring the asphalt emulsion hot and letting it flow through the cracks and joints by gravity. Also, it was found necessary to cut down the old expansion joints to about ¼ in. below surface of pavement. This depression allows the material to run along the joint and the bottom of the slab. It will eventually fill the joint and not run out and over the pavement surface.

The reason for the effectiveness of the subsealing with the asphalt emulsion is that the asphaltic material flows under the pavement slab, fills all voids, and at the same time penetrates the underlying material, thus stabilizing the subgrade. When later methods of applying the asphaltic emulsion under pressure were developed, it was found that the penetration of asphalt into the soil was from 1 to 2 in.

During the 1951 season forcing the asphalt emulsion beneath the slab by using pressure was begun. A "quick-breaking" type of asphalt emulsion that meets New York State specifications for Item 70-B, and which is substantially AASHO Specification RS-1, was used with good results. However, a heavier grade was found easier to control and also found not to break out through the pavement and shoulders as readily as the lighter material. At the same time it was discovered that if the material was heated from 130 to 140 F, it worked even better.

In the District maintenance shop a special attachment, consisting of a tapered nozzel and a shut-off valve, was developed. A hose connects this nozzle to either a large distributor or a small Tarrant machine. Through this attachment the asphalt emulsion is pumped through previously drilled holes in the concrete slab at a pressure of 20 to 50 psi. These holes are drilled adjacent to a joint or to cracks, and locations are determined by the locations of joints and cracks. Generally holes are 5 to 10 ft from the joint and 2 to 3 ft from the center line. The size of the holes drilled are approximately $1\frac{1}{2}$ in. in diameter, using standard drill steel from $1\frac{1}{8}$ to $1\frac{3}{8}$ in. in diameter. By pumping asphaltic material into these holes on different days, it was found, at some locations, that as much as 200 gal was forced under the pavement at some joints.

As yet, no effort has been made to blow the water from beneath the slab with air pressure, prior to pumping the asphalt emulsion into the holes. The water that may be present under the slab is expected to be forced out by the pressure of asphalt. Air pressure has not been used for fear of blowing the mud which may exist under the slab into the small cracks and channels that may be underneath and thus restrict the flow of bituminous material used in the subsealing.

When applying the asphalt under pressure in subsealing work, the shoulders must be carefully watched for possible blow-up material to the surface of shoulder. At the first appearance of any asphalt on the surface at the shoulder, the asphalt flow through nozzle should stop immediately.

On subsealing operations where the material is simply poured into the joint by gravity no extrusion of asphalt material has ever been observed either at time of pouring or at a later date. This is due to the fact that the time element is long enough to permit the asphalt to cure and set.

When subsealing was done using pressure and large quantities of material were used in a short time because of cavities under concrete slab, it has been found necessary to have traffic lanes closed for a few hours to permit curing and setting of asphalt. No extrusion of the material was observed at a later date.

As stated previously, the asphalt emulsion meets New York State specification for Item 70 B, Grade B, the same material used for scale patching and pavement repairs. Therefore, no special material is required for subsealing work. By heating the material to 140 F, it becomes less viscous and will flow under the slab more easily, guaranteeing more complete filling of all cavities.

The use of asphalt emulsion for this purpose is strictly for subsealing and stabilizing subgrade soil at locations made apparent by observation of pavement conditions and "pumping." No attempt is made to jack up any depressed slabs to theoretical elevations; depressed areas, after subsealing, are brought up to correct grade by spreading and rolling plant-mixed asphaltic concrete.

In 1952 a statewide investigation of concrete pavement joint supports was made. In sawing out sections of joints, the maintenance personnel deliberately selected those joints that had suffered severe faulting and had been treated the previous season by subsealing with asphalt emulsion under pressure. The subsoil under the slab consisted of a silty clay and sand mixture. A number of soil samples were removed and examined.

It was interesting to observe that the emulsion had penetrated the soil from 1 to 2 in. From this observation it is apparent that the asphalt emulsion not only fills the voids in and below the pavement slab but it also stabilizes the subgrade, which is extremely important.

If a concrete pavement is properly subsealed once, it is a fairly easy operation to go over these pavements once or twice a year to further subseal and fill any cracks. Also, if a subsealing job is going to be successful over a long period of time, it is important to do other, very necessary maintenance work to the pavement and highway section; such as,

1. Cut down all high shoulders so that pavement can drain off to ditches quickly and not allow water to stand along edges and saturate subgrades.
2. Restore all ditches to original designed grades to insure proper carrying away of water.
3. Clean all outfall ditches and make sure they operate 100 percent efficiently.

If these maintenance operations are carried on faithfully and adequate drainage of the highway section maintained at all times, the subsealing work will be 100 percent effective, inasmuch as water is the greatest enemy of the pavement section.

In District No. 4, there are no known concrete pavements that are "pumping" at the present time. From 1948 to 1953, the District concentrated on this work, from necessity. Since 1951, this subsealing operation has become just one part of the over-all program for maintaining concrete pavements which consist of subsealing, joint repairs, scale patching, shoulder maintenance, ditch maintenance, etc. Therefore, the operation of subsealing and allied maintenance by the District's forces during the past twelve years has paid handsome dividends by making it possible to utilize hundreds of miles of these old concrete pavements as foundations for a concentrated widening and resurfacing program at a low cost per mile. This is borne out by the fact that of the 1,400 mi of highways in District No. 4, approximately 400 mi have been widened and resurfaced so that they meet modern standards, utilizing salvaged concrete pavements that were saved from deterioration and destruction through subsealing.

In Monroe County alone, over 90 mi of this work was done within the last 2½ years at an average cost of $100,000 per mi. Some costs ran as low as $50,000 per mi, but average was brought up because of greater width on some jobs. Had the pavements been allowed to break up, the cost of replacement would have been many times more. This represents a tremendous saving to the taxpayer and at the same time provides him with a substantial mileage of new pavement with a minimum of inconvenience.

In conclusion, from 1948 to 1961, District No. 4 has used an average of 300,000 gal of of asphalt emulsion per year for subsealing, scale patching, and joint pouring. This work was done entirely separate from the regular surface treatment program, completely with the District's own forces, and without additional funds or equipment.

Design, Maintenance and Performance of Resurfaced Pavements at Willow Run Airfield

WILLIAM S. HOUSEL, Professor of Civil Engineering, University of Michigan, and Research Consultant, Michigan State Highway Department

● 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 Airfield. Willow Run was built in 1941 by the Ford Motor Company under a Defense Plant Corporation contract. As a consultant to the Ford Motor Company and its 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 ft, deposited on a waterworked clay till plain, within the limits of the postglacial Lake Maumee, now Lake Erie. Subgrade conditions over most of the field were almost ideal, although in the north-central portion there was a low-lying 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 that has remained in effect 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 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

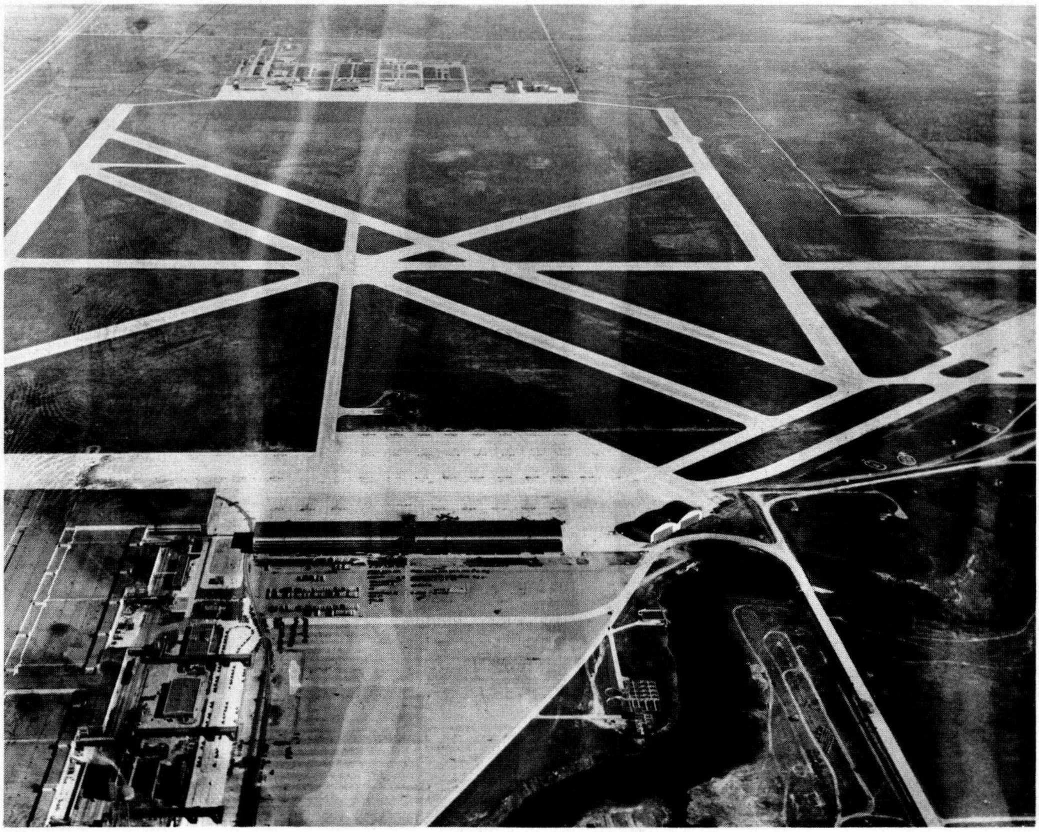


Figure 1.

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

Figure 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 sq yd of concrete pavement, roughly equivalent to 115 mi of 22-ft highway pavement. As noted previously, 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 ft with a longitudinal keyed construction joint at both sides and a dummy joint at the center of the pour, subdividing the pavement into 10-ft lanes. There were transverse expansion joints at a spacing of 125 ft, with ¾-in. premolded filler and ¾-in. round steel dowels at 12-in. centers, and dummy contraction joints at a spacing of 25 ft. Paved runways were 160 ft wide and taxiways 80 ft wide.

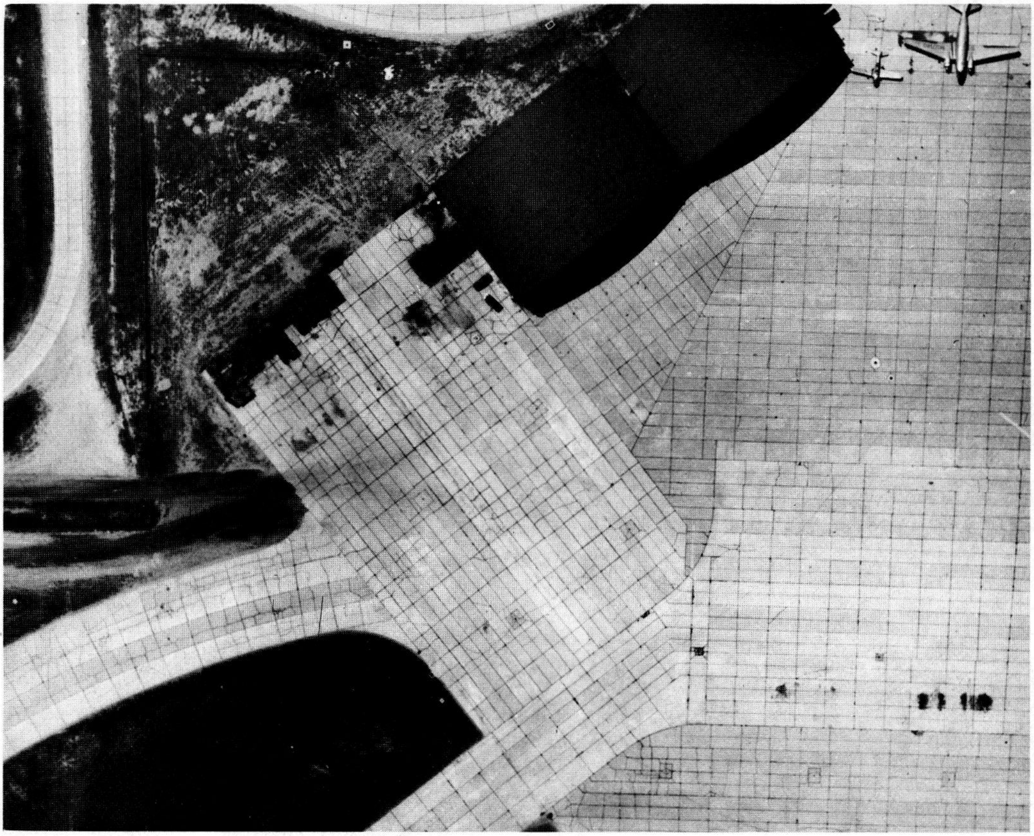


Figure 2.

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 ½-in. round steel dowels 24 in. long at 30-in. centers in the longitudinal dummy joint for 15 ft 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.

There were some significant variations in construction practice, however, between the 1941 and 1943 construction, which developed some sharp contrasts in performance, revealed by subsequent pavement condition surveys. Figure 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.

Figure 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-ft 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-ft slabs into two slab lengths. A survey

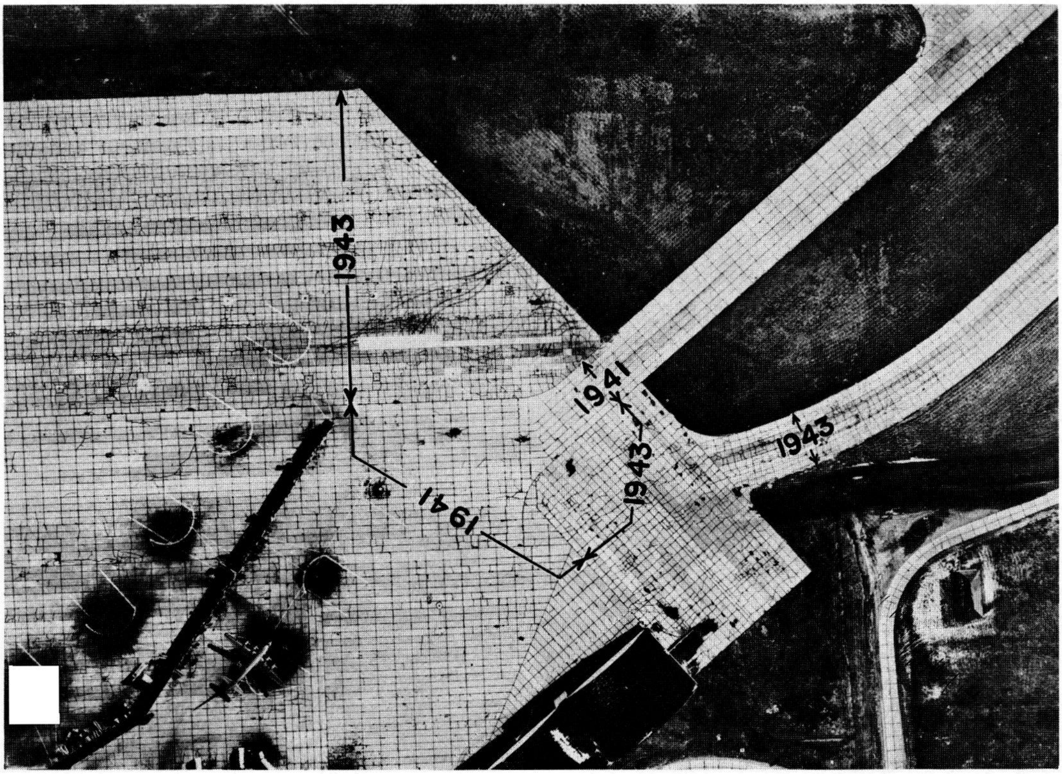


Figure 3.

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 previously.

TABLE 1

PERCENTAGE OF CRACKED SLABS, 1950 SURVEY

Type of Crack	Year of Survey	Cracked Slabs (%)	
		1941[a]	1943[a]
Transverse	1946	4	35
	1950	10	85
Longitudinal	1946	Negligible	15
	1950	Negligible	25
Diagonal	1946	Negligible	2
	1950	Negligible	4

[a]Date of construction.

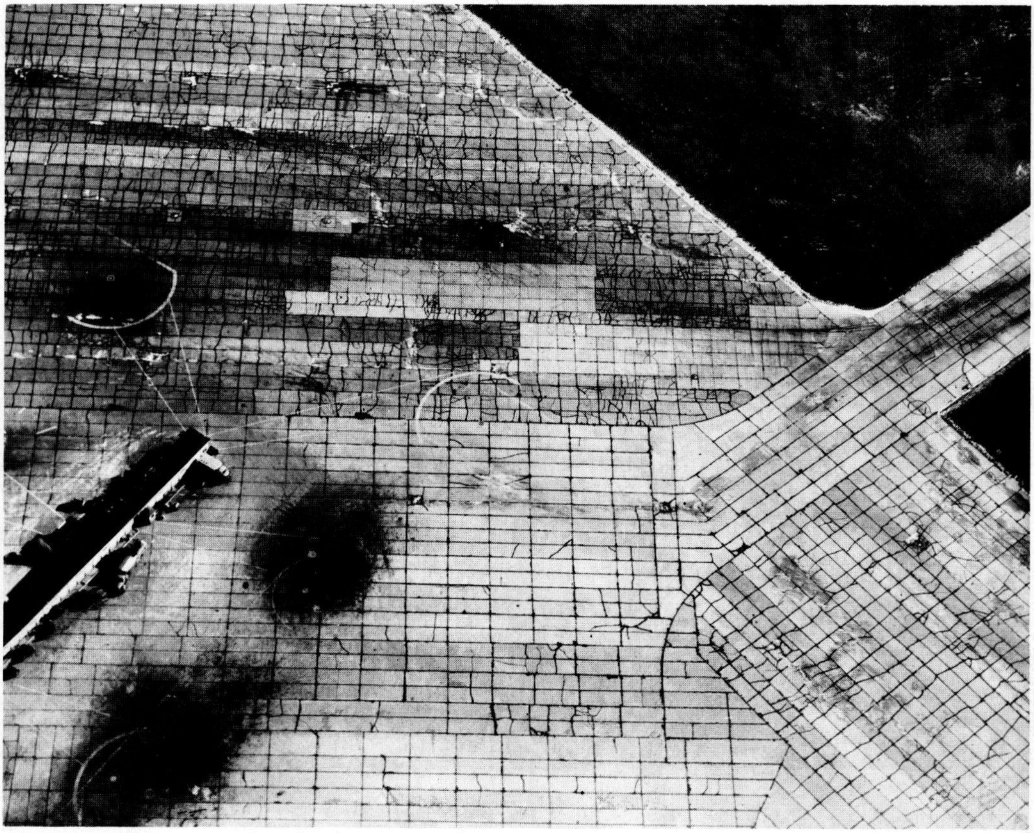


Figure 4.

One of the most interesting developments shown in Figure 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-ft 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.

Figure 4 is another photograph of the same area taken in 1954, just before 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

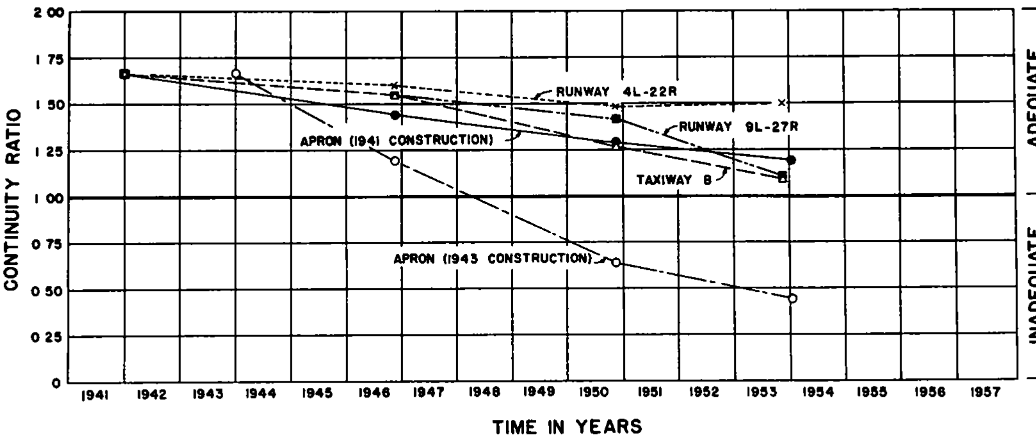


Figure 5.

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 where 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 ft. Thus, the initial continuity ratio for a 25-ft slab length would be 1.67, while a continuity ratio of unity, or an average slab length of 15 ft, would be considered satisfactory from the standpoint of structural adequacy. Continuity ratios less than unity, or slab lengths less than 15 ft, 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, Figure 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.

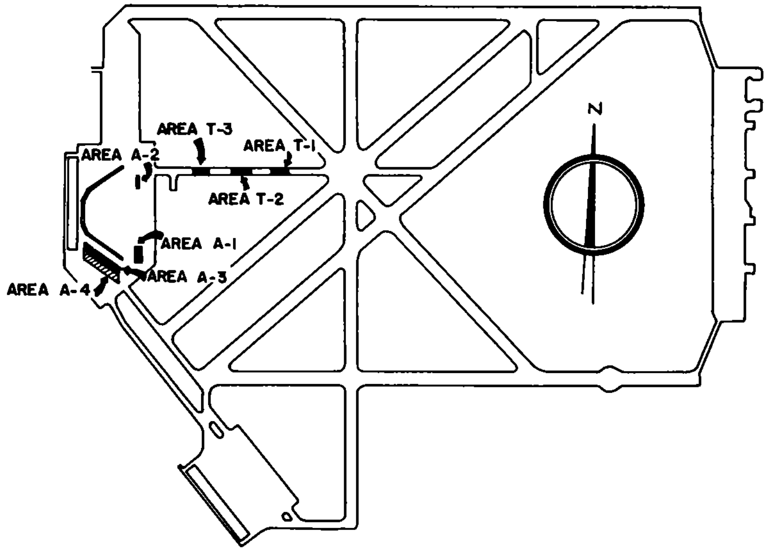


Figure 6.

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. Although 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 Figure 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. Although 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 1¾-in. bituminous concrete binder course (CAA Specification P401-A) and a 1¼-in. 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 gal per sq yd. 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-in. 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 (1) so it will not be given in detail in this report. The welded wire fabric was placed directly on the concrete surface, with the transverse wires at 3-in. 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 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 1¼-in. 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 that have been reflected through the bituminous surface. Variation in the construction details of the bituminous surface that are being compared is indicated in connection with that portion of each test area to which it refers.

Test Area 1, shown in Figure 7, is on the center taxiway and was intended to show the effect of varying the thickness of the bituminous surface from 3 in. in Area T-1-a to 4 in. 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 percent in partial Area a and 24 percent in partial Area b, with an 8 percent differential in favor of the greater thickness of bituminous surfacing. Also, 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 percent 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 Figure 8, is on the central taxiway and gives a comparison of reflected cracking in the 3-in. 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 the figure, 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 ft 6 in. wide, laid over the longitudinal joint with a 6-in. clearance between sheets at the center of the 10-ft lanes. This might be regarded as a construction defect that could possibly be corrected by providing continuous reinforcement in some manner between the sheets of wire mesh.

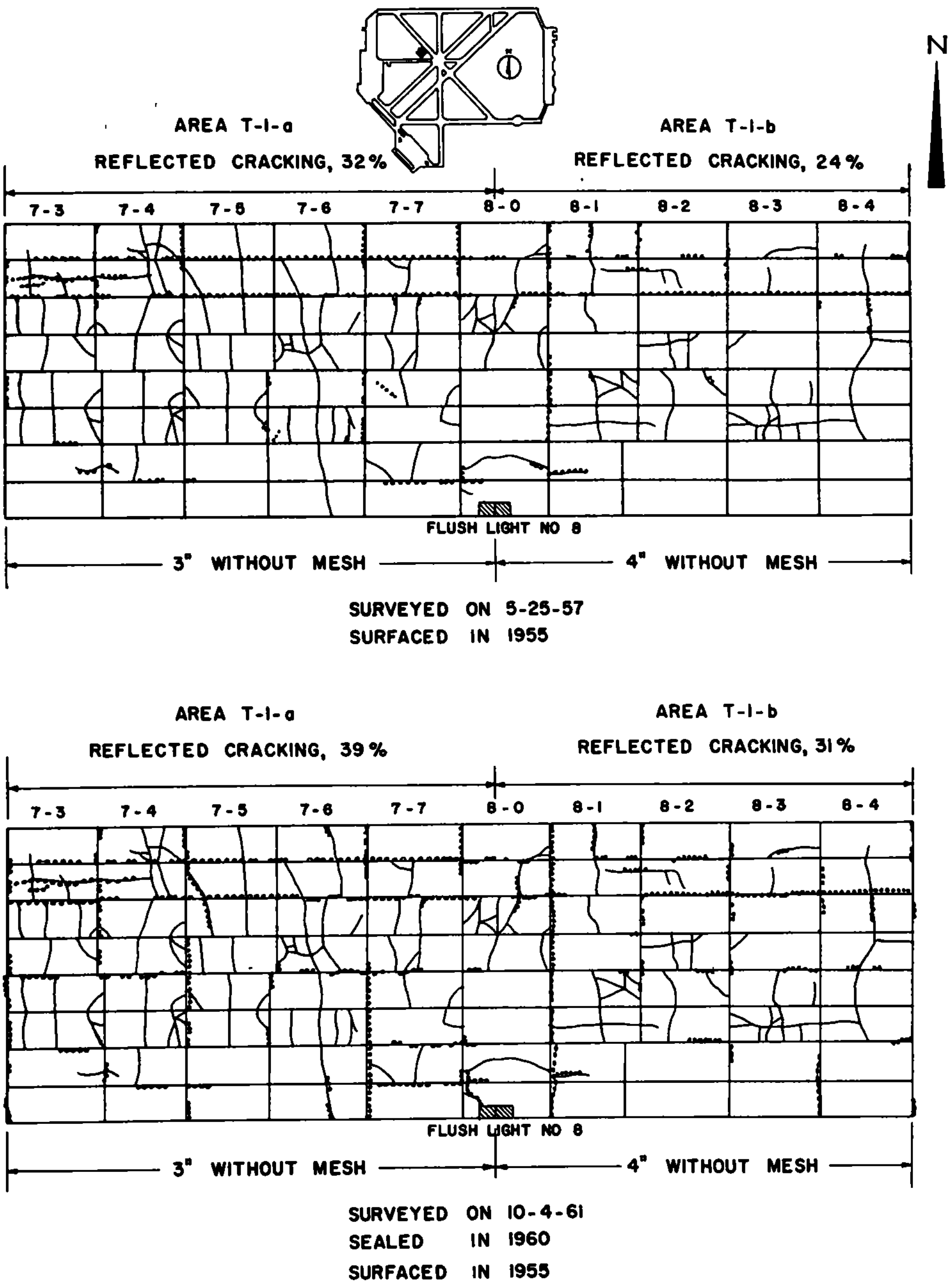


Figure 7.

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 pre-

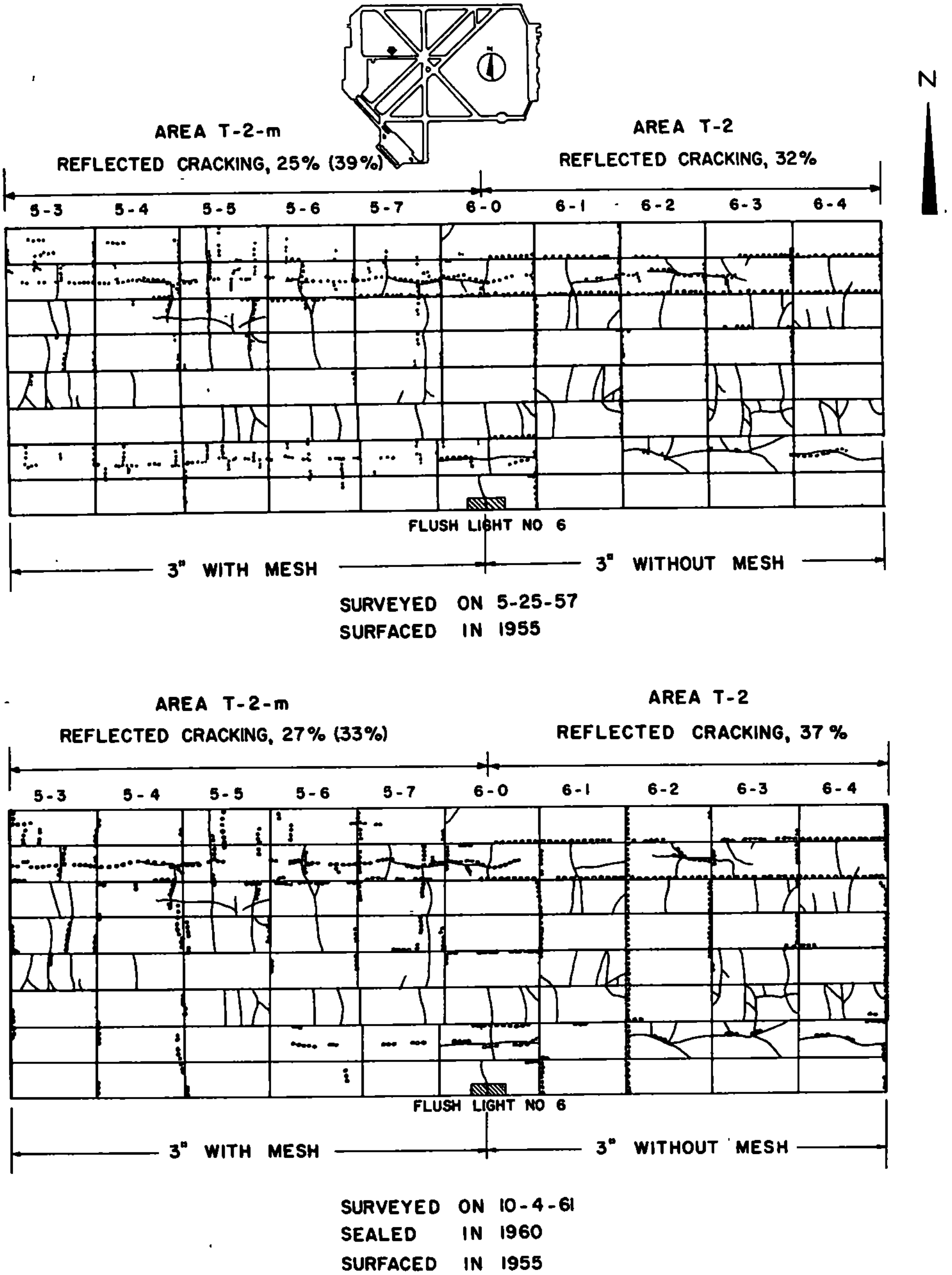


Figure 8.

viously 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) percent with mesh and 32 percent without mesh, must be viewed with the previous 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 percent 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 percent from 1957 to 1961, even with the benefit of sealing in 1960. In the reinforced area, the direct reflection increased from 25 to 27 percent, a negligible figure in practical terms, 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) percent, 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 Figure 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 Figure 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-in. 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-in. bituminous resurfacing, with and without welded wire fabric. Also, 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 Figure 10, the unreinforced area showed a reflected cracking of 44 percent; in comparison, the reinforced area had 33 percent direct crack reflection, or (38) percent including the translated cracking. In both cases, there was a measurable differential in favor of the reinforced area. At the bottom of Figure 10, the results of the 1961 survey of the areas sealed in 1960 show the unreinforced area having a reflected cracking of 47 percent, as compared to a direct reflection in the reinforced area of 29 percent, or a total of (30) percent including translated cracking.

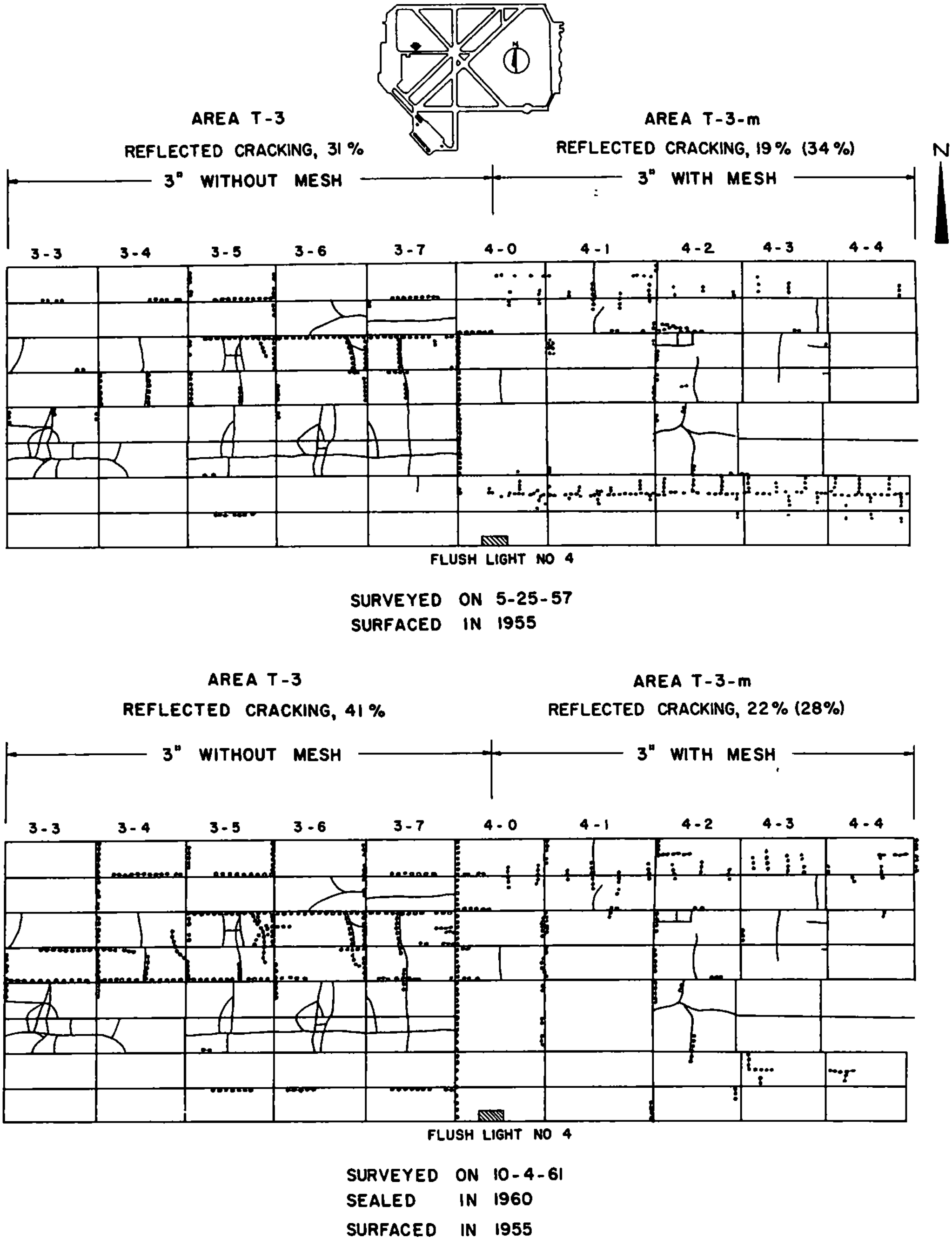
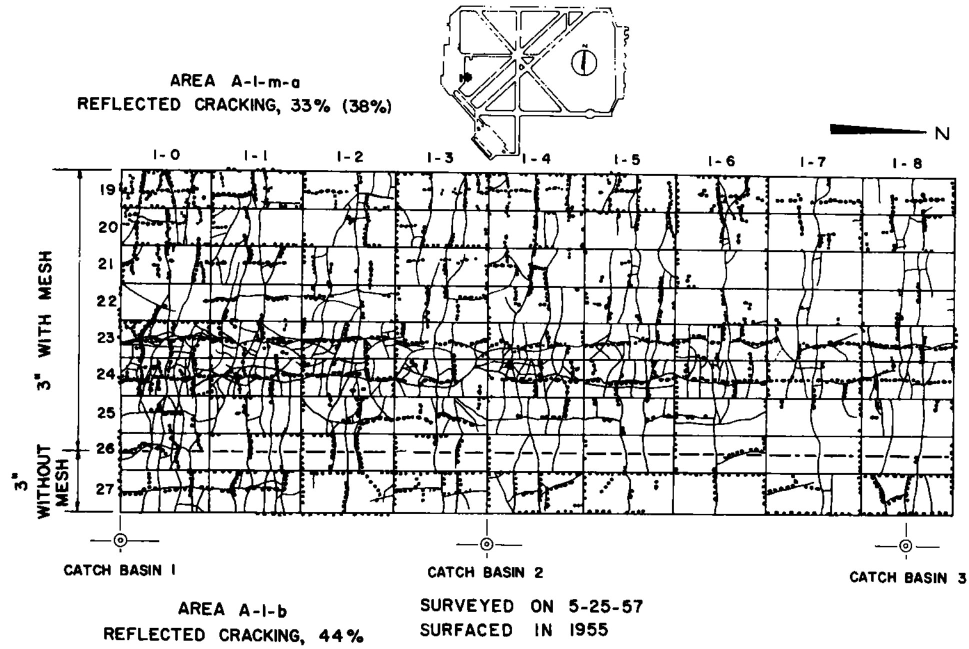


Figure 9.

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 percent and the total cracking, including translated cracking, as (75)



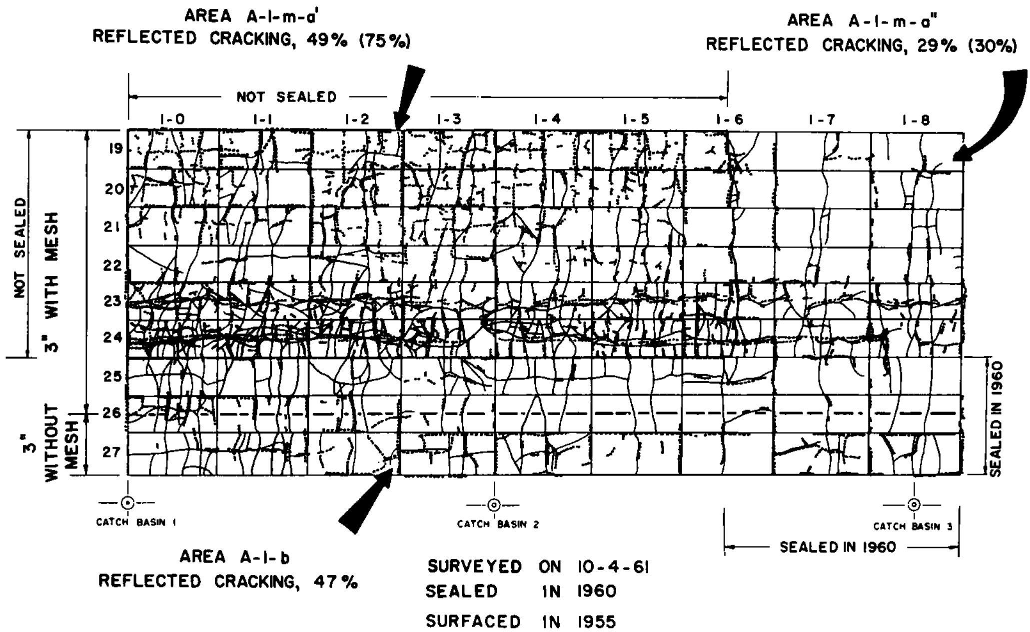


Figure 10.

percent. 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 that go somewhat beyond the main investigation of reflected cracking. In Test Area A-2,

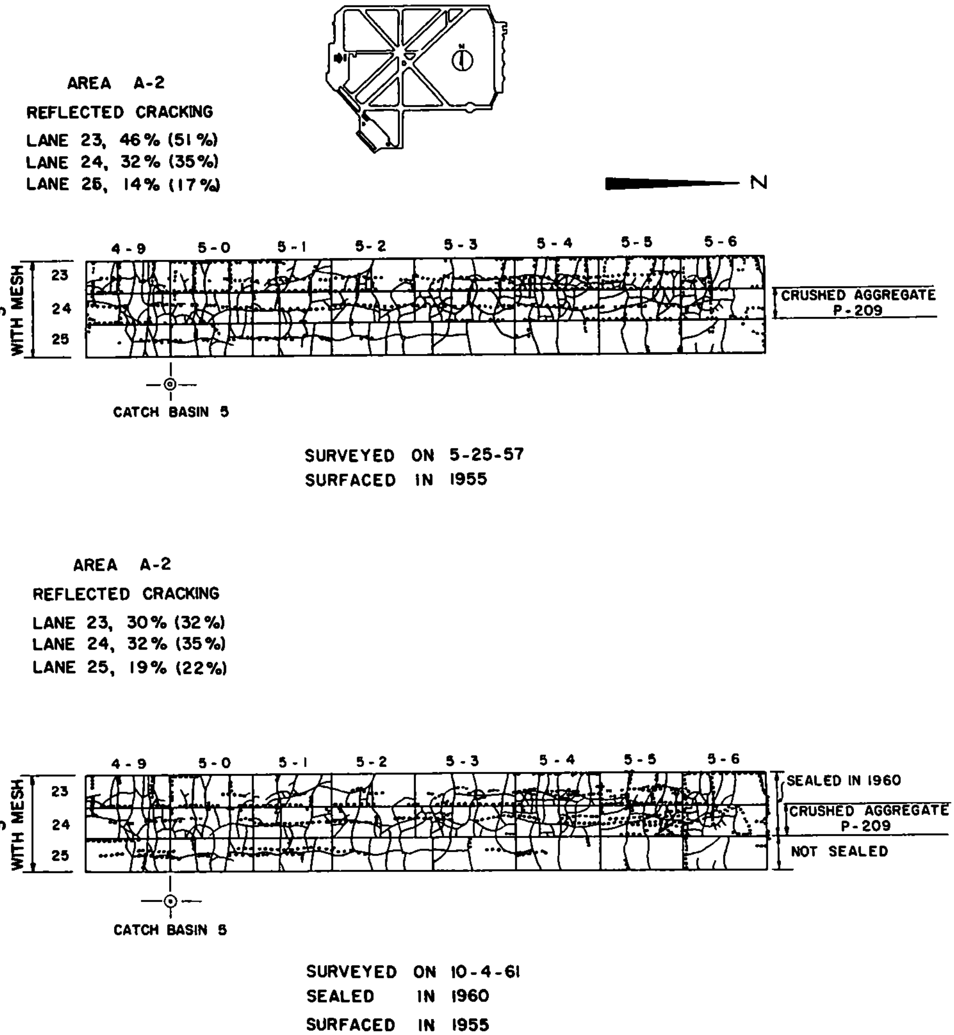
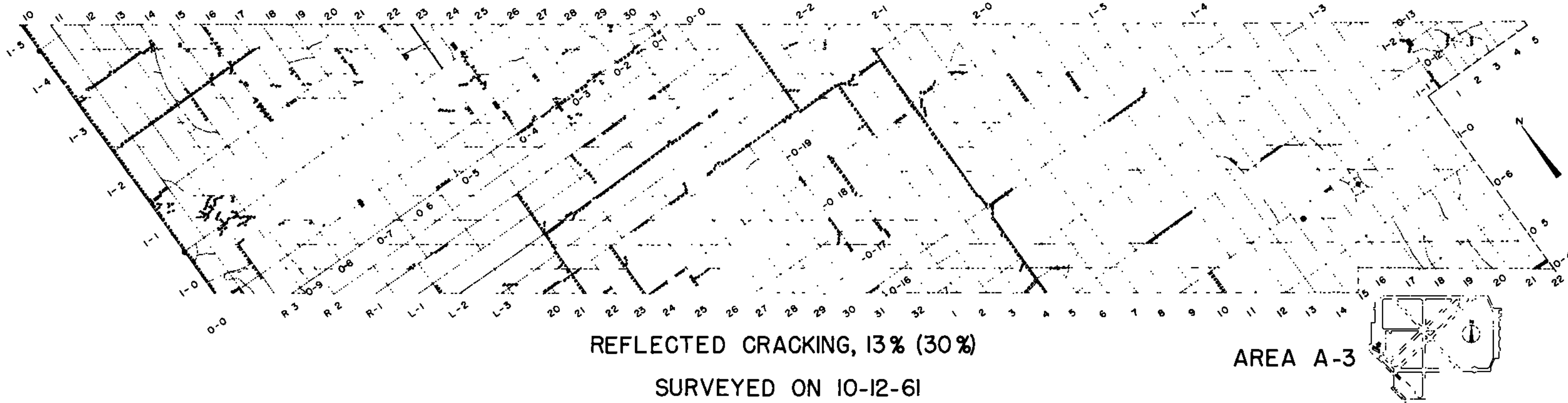


Figure 11.

shown in Figure 11, an experiment was tried on a somewhat limited scale that 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 that, at any rate, was well compacted to give it every chance for survival. The results, shown in Figure 11, are 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 percent, in the cracking with the aggregate base.

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

Figure 12.

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 Figure 12, was supplied with a 1-in. 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 Figure 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-in. 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 percent, as compared to 54 percent in the conventional bituminous surface. If the special cracking between lanes is included in the Epon area, the total cracking would be 30 percent, 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 15 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. Although 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 hopes 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 in. per mi, 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 land being subjected to severe frost action. The extreme roughness that 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 in. per mi, 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 in. per mi, which would also be rated good. The center lanes of the runway, and particularly the center taxiway, are subject to heavy load repetitions; and, though 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 lb gross plane weight for the runways and 41,600 lb for the field, as limited by the 1943 construction. "Capacity Operation," as then defined, was based on a 20-yr life and 100 scheduled operations per day. When a traffic analysis was made in 1954, commercial planes supplied 80 percent of the traffic, the remaining 20 percent 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 lb. An analysis indicated that 5 percent of the traffic exceeded the rated capacity by 200 percent, 20 percent of the traffic exceeded the rated capacity by 100 percent, and only approximately 15 percent 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.

Though 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 that 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.

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

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.

CONCLUSIONS

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 that may be used for reference in connection with these final conclusions.

1. Excessive cracking in portions of the 6-in. plain concrete pavement at Willow Run Airfield 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 percent of the total lineal feet of joints and cracks in the underlying concrete pavement, or 30 to 40 percent 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 in., or 33 percent, in the bituminous resurfacing reduced the reflected cracking some 8 percent in terms of the total lineal feet of joints and cracks, and from 20 to 25 percent of the reflected cracking in the area of standard 3-in. thickness. The percentage of improvement was the same before and after sealing.

7. A special experiment with a 1-in. 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.

ACKNOWLEDGMENTS

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 (Michigan Trucking Association, American Trucking Associations, Inc., 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. G.R. Ingimarsson, Research Fellow of the National Petroleum Refiners Association, is assisting in the current studies.

REFERENCE

1. Wakefield, F.G., "The Practical and Laboratory Use of Wire Fabric in Bituminous Resurfacing at Willow Run Airfield." ARBA Tech. Bull. 215 (1956).

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