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SPECIAL PAPERS
ON
THE PUMPING ACTION OF CONCRETE PAVEMENTS

1945

COMMITTEE
ON
MAINTENANCE OF JOINTS IN CONCRETE PAVEMENTS
AS RELATED TO
THE PUMPING ACTION OF THE SLABS

Highway Research Board
Division of Engineering and Industrial Research
National Research Council

COMMITTEE ON MAINTENANCE OF JOINTS IN CONCRETE PAVEMENTS AS RELATED TO THE PUMPING ACTION OF THE SLABS

Special Papers Presented
for
The Twenty-fourth Annual Meeting
(Unassembled)

Edited by
Fred Burggraf
Assistant Director, Highway Research Board

Washington, D. C.
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The committee was organized in 1942 to study the causes of pumping and to find, if possible, corrective measures which might be effective.

The membership consists of representatives from the State Highway Departments of Oklahoma, Kansas, Missouri, Illinois, Indiana, Ohio, and New Jersey, the Portland Cement Association, the Koehring Company, and the Public Roads Administration.

The first activity of the committee was the preparation of a Wartime Road Bulletin No. 4 on Maintenance Methods for Preventing and Correcting the Pumping Action of Concrete Pavement Slabs.[1] This bulletin was published in October, 1942 and has been of considerable value to maintenance engineers who were called upon to use mudjacking methods for the first time.

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[1] - See page 131 of this publication.

In 1943 the committee sponsored two papers which were published in the Twenty-third Annual Proceedings — one by K. B. Woods and T. E. Shelburne of Purdue University on the "Pumping of Rigid Pavements in Indiana," and the other by R. W. Couch of the Missouri State Highway Department entitled, "Deflectometer for Measuring Concrete Pavement Deflections under Moving Loads."

For 1944 the committee presents as a progress report papers covering experiences and methods of control used in Missouri, Ohio, Indiana, New Jersey, and Tennessee. These papers describe work that has been done in an effort to determine the causes of pumping, to develop maintenance procedures that will relieve the condition on existing pavements, and to develop pavement and subgrade details that will prevent its occurrence on future construction. The conclusions presented are those of each author and do not necessarily represent the opinion of the committee as a whole.

Reports are being prepared on the work done in Kansas and North Carolina and these reports should be available for presentation at the next annual meeting of the Highway Research Board. The committee plans to start work on its final report during 1945.

MAINTENANCE METHODS FOR PREVENTING AND CORRECTING THE PUMPING ACTION OF CONCRETE PAVEMENT SLABS

Rex M. Whitton, Engineer of Maintenance,
Missouri State Highway Department

SYNOPSIS

This paper describes experience in Missouri with the correction of pumping at joints in concrete pavements by the use of a semi-fluid soil-cement mixture forced under the slab with a mudjack. The slurry used consisted of four sacks of cement per cubic yard of topsoil and 50 to 55 per cent of water. The spacing of the holes for elimination of pumping and correction of faulting is discussed. The equipment and personnel required is given in detail. On an average section of pavement, it was found that 354 holes per mile and 35.45 cubic yards of slurry were required per mile of road. The average cost of the work was $24.78 for drilling 1½-inch holes and $256.66 per mile for the material and pumping operation.

In a study of the deflections of the pavement under moving loads, it was noted that deflections increased immediately after filling the voids under the pavement with the soil-cement slurry and then decreased after a period of time had elapsed. For a 12,000-pound rear axle load, the deflections were reduced as much as 0.007 in. and for a 16,000-pound wheel load as much as .011-in. between measurements made 9 days and 153 days after mudjacking.

It was found that a few of the mudjacked slabs resumed pumping and that the work had to be supplemented with joint and crack water-proofing to keep surface water from reaching the subgrade. On pavements that were cracked extensively, the best method of waterproofing joints was by the use of a substantial bituminous surface or upper deck not less than 1-in. in thickness.

After a study of design features from the viewpoint of a maintenance engineer, the author concludes that expansion or contraction joints should not be used in concrete pavements except at highway intersections, bridge ends or other locations where the pavement abutts a fixed object. The relation of the type of aggregate to the crack interval in concrete pavements in which no joints were placed is discussed.

The type of concrete pavement failure that is causing the most concern today in Missouri is the result of "slab pumping" or the deflection of the pavement under a moving load. "Slab pumping" is the ejection of water thru the joints and cracks in concrete pavements carrying soil particles from the subgrade. Continued "slab pumping" results in voids under the concrete pavement and finally the breakdown of the pavement itself. "Slab pumping" is indeed a malignant disease of concrete pavement.

It has been proven through both research and observation that four factors must be present to create a "pumping slab". They are: (1) heavy axle loads; (2) joints or cracks in the pavement; (3) unsuitable subgrade soil; (4) "free" water under the slab.

The elimination of any one or more of these factors extends the life of the concrete pavement for many years.

Since the legal limit for axle loads in Missouri has been increased during the present war from 16,000 lb. to 18,000 lb., and since it seems to be generally accepted that an 18,000 lb. axle limitation should be adopted as standard in all the States, this axle load factor cannot be readily changed.

Joints and cracks which have been delibberately placed, or cracks which have occurred from natural causes, in the concrete pavement cannot be eliminated; but their effectiveness as a means by which free water gets under the pavement can be reduced very materially in most instances by keeping the joints and/or cracks properly filled and by placing a bituminous upper deck over the entire area of the affected pavement.

It is not feasible to remove and replace the undesirable subgrade soil now in place under the concrete pavements constructed in the days when little or no thought was given to obtaining the most satisfactory type of supporting soil. It is, however, possible to stabilize the undesirable subgrade soil to such an extent that it is more effective in sustaining the load.

The range of soil types on which pumping has occurred is quite extensive. Any soil that can be puddled into a suspension in water will react to the rocking action at pavement joints and cracks and in the presence of water will develop pumping. There are some soil types developed in areas of low relief possessing a distinct clay-pan, such as the Putnam Silt or Lebanon Silt, which are relatively more susceptible to pumping than the more drainable types such as the Knox Silt or the Marshall Silt. However, any clay or silt type soil in the proper environment will develop pumping.

It is also possible to overcome the "free" water under the pavement and extend the life of much concrete pavement until a post war reconstruction program permits the older pavements to be replaced or adapted to the needs of today and tomorrow. By pumping a slurry mixture of soil, cement, and water through the pavement, the void is filled and the reservoir for the accumulation of "free" water is thereby eliminated. The history of the development and methods adopted in Missouri will be discussed briefly for performing this operation.

As early as 1935, attention was directed to "slab pumping" on the most heavily traveled routes in Missouri, which forced the free water thru the cracks in the pavements, causing erosion and displacement of the subgrade and the resulting voids under the pavement. In 1937, one division of the State Highway Department used a mudjack to pump a semi-stiff mixture of soil and cement under the badly damaged pavement to fill these voids, force out the free water, stabilize the subgrade, and reduce the "slab pumping". The soil-cement mix was injected through a hole drilled at the intersection of each transverse crack with the longitudinal centerline. In 1938 the method was extended over other sections, but the original single hole at each crack was supplemented by two other holes; one being drilled in the transverse crack at the middle point of each lane of traffic. The results obtained by the use of the three holes were more satisfactory than by the one hole method of 1937. During the next two years, the sections pumped by both methods did not progress in failure as rapidly as the sections not treated.

In 1941, when the pumping action was noted on other heavily traveled routes where the condition had heretofore been present but less intense, a special study was made of the problem. After obtaining the experience of other States, a decision was made to use a slurry mix of loam top soil and portland cement instead of the semi-stiff mix. A more fluid mix was necessary to give a greater spread of the mixture under the pavement and to fill more of the small voids and channels.

This slurry mix was obtained by mixing the soil and cement, the proportions being four sacks of portland cement per cubic yard of top soil with 50 to 55 per cent water, in the pugmill of the mudjack.

Observations in the field indicated that, by the use of this slurry mix, the soil-cement would be pushed or pumped through to the top of the pavement at cracks 20 and 25 ft. from the point of injection. Also by using this mix the pavement was not set up on stools as was the probable condition in previous years when the stiff mix was employed for raising settlements.

Experiments have been instigated and are still being carried on to determine admixtures for the present soil-cement slurry mix to improve its quality. A mix is sought that will set up rapidly, will have strength without too much rigidity, and will have a minimum shrinkage.

The holes for corrective and preventive mudjack work are now placed 10 in. to 12 in. from the centerline and 10 in. to 12 in. beyond the transverse crack or joint in the direction of traffic. Since the use of air in conjunction with the pressure supplied by the mudjack has not been utilized due to lack of equipment, we have come to the conclusion that one hole should be drilled in diagonal corners at the location mentioned above so that the greatest amount of slurry can be pumped under the pavement. Experimental sections were set up using both the one hole method and the two hole method and it was found that 0.09 cubic yards more slurry was pumped per crack where two holes were used instead of one. When it is necessary to use a hole on the outside of the pavement for filling or for raising purposes, it should be placed 30 in. in from the edge of the pavement and 30 in. beyond the crack in the direction of traffic. By using this spacing, the inside and outside hole will not be in the same line; which, if it were the case, would establish a plane of weakness and could cause a break if the treated joint should resume pumping in future years.

The holes are drilled with a pneumatic jack hammer using 18 in. drill steel and a 1½ in. removable bit. Where the selected soil desired is not available, it is more desirable to use a 1-3/4 in. or 2 in. hole so that a larger nozzle opening can be used and thus avoid delays from the rocks and roots in the mixture. However, the 1½ in. hole is used in all cases where desirable soil can be obtained for the work. Usually a single man not only drills the holes, but moves the equipment ahead as his work progresses, takes care of the compressor, and shifts the barricade which protects him from traffic. He may drill as many as 325 holes 1½ in. in diameter per ten hour day, but the total varies with the condition of the pavement to be treated and the number of moves required per mile, as well as with the kind of coarse aggregate that has been used in the concrete.

The 1½ in. holes cost an average of $0.07 per hole, and in some sections of the State, 402 holes are required per mile to treat every joint and crack.

The mudjack is mounted on a four wheel pneumatic tired trailer equipped with a four cylinder gasoline motor that supplies the power to operate a pugmill and the two pistons delivering the slurry through a 2 in. diameter, 5-ply rubber hose to a tapered steel nozzle. The nozzle is equipped with a 2 in. quick opening gate valve and is tapered from 1¼-in. to 2½-in. outside dimensions.

The mudjack trailer unit is towed by a truck carrying a 500 gallon water tank and a small gas driven centrifugal water pump. The water tank is connected to the water pump of the mudjack by a 1-in. hose, which can be readily disconnected when it is necessary to refill the water tank.

The mudjacking work is performed by an eight-man crew, consisting of a foreman, mudjack operator, nozzle operator, a laborer adding soil and cement to the pugmill, a truck driver operating both trucks between the mudjack and the dirt pit, a laborer who plugs the holes and cleans the surface of the pavement after jacking, a laborer at the soil pit, and a flagman. Such a crew has pumped as much as 28 cubic yards of slurry per ten hour day and averaged 18 cubic yards per day over the 1943 season.

One division so far this season has treated every joint and crack on 31.7 miles of U.S. Route 40, drilling 11,212 holes and pumping 1,125 cubic yards of soil-cement slurry under the pavement. This is an average of 354 holes per mile and 35.45 cubic yards of slurry per mile. This work is costing on the basis of $24.78 per mile for drilling the 1½-in. holes and $256.66 per mile for the material and pumping operation, or a total cost of $281.44 per mile.

Usually no attempt is made to raise the pavement from its warped position, unless a local condition is encountered where the pavement surface still has good riding quality on either side of the settlement. It is now our policy in Missouri to merely fill the voids under the pavement, thereby materially reducing the deflection of the slab; and then restore the riding surface and relieve the impact by resurfacing the concrete pavement with a bituminous upper deck, which will also aid in water-proofing the surface of the old pavement.

Deflection tests have been made in the field to ascertain the deflections of the concrete pavement under a moving load and some of the results obtained are tabulated in Table 1.

Deflections were measured before and after mudjacking or filling the voids with a soil-cement slurry. It was noted that the deflections increased nine days after mudjacking. This was caused from the quantity of water necessary to prepare the slurry mixture which saturates the subgrade, but 153 days after mudjacking, when additional tests were made, the deflections had reduced materially. This condition does not always exist, depending on how badly the pavement is cracked before mudjacking. In the case of the 12,000 lb. rear axle load the deflections were reduced as much as .007 in. whereas in the case of the 16,000 lb. rear axle load the deflections were reduced as much as .011-in. between the tests run 9 days after mudjacking and 153 days after mudjacking.

TABLE 1

Deflection Tests - Route 40 - Cooper County

Test Slab No. 1					:	Test Slab No. 1A				
Run No.	Velocity M.P.H.	Axle Loads Rear	Axle Loads Front	Deflec. Inches	:	Run No.	Velocity M.P.H.	Axle Loads Rear	Axle Loads Front	Deflec. Inches
(Before Mudjacking)					:					
1	9.9	12000	3100	.018	:	No measurements made before mud-jacking				
2	18.8	12000	3100	.014	:					
3	28.4	12000	3100	.013	:					
4	38.3	12000	3100	.009	:					
(Same Day - After Mudjacking)					:					
9	9.4	12000	3100	.016	:	No measurements made on same day after mudjacking.				
10	19.4	12000	3100	.013	:					
11	28.6	12000	3100	.010	:					
12	38.7	12000	3100	.009	:					
(9 Days After Mudjacking)					:	(9 Days After Mudjacking)				
21	10.2	12000	3200	.020	:	1	10.4	16000	3000	.011
22	18.8	12000	3200	.015	:	2	18.6	16000	3000	.008
23	29.2	12000	3200	.018	:	3	28.2	16000	3000	.011
24	38.0	12000	3200	.016	:	4	38.0	16000	3000	.014
(153 Days After Mudjacking)					:	(153 Days After Mudjacking)				
29	8.3	12000	2900	.012	:	7	13.6	16000	3000	.000
30	9.9	12000	2900	.012	:	8	19.6	16000	3000	.000
31	19.8	12000	2900	.007	:	9	29.0	16000	3000	.003
32	29.0	12000	2900	.006	:	10	39.5	16000	3000	.003
33	40.6	12000	2900	.006	:					

Not only do we receive a benefit from the decrease in deflections of the pavement by filling the voids beneath the pavement in the subgrade; but, if the pavement can be made to stay in contact with the subgrade, the weight of the pavement will keep the subgrade in compression, and will thereby reduce the quantity of water which can be absorbed by the soil, since soil under pressure will not absorb water nearly as readily as soil that is not under pressure.

The slurry mixture is pumped under the pavement until all of the free water under the pavement is forced out of the voids and is drained off through the cracks and holes dug at the edge of the pavement at the transverse crack for that purpose, and the slab begins to rise. This drainage hole, which is made with a spud bar or a pick, is extended about 2-in. below the bottom of the pavement. As soon as the water has been forced out and the slurry mixture appears at the shoulder, the hole is closed and tamped with earth.

When the nozzle operator has added sufficient slurry at the hole, he inserts a tapered wooden peg to retain all of the mix under the pavement until the cleanup man can plug the hole with a mix of soil and cement. This workman, after the soil and cement plug is added, passes his shoe over the hole and so obtains a slick finish that will not be destroyed by traffic. Enough moisture seeps through from below to provide the necessary dampness to make this plug set.

After the first pumping, the slab is usually disturbed by traffic before the mixture has taken its initial set, so that the voids remaining are not completely filled. Also the shrinkage of the slurry mix due to the high water content leaves some small voids. Thus, it is usually necessary to repump all sections where pavement failure has progressed to any great extent.

Results obtained by mudjacking, during the past ten years, indicate that voids underneath the slab where "free" water might collect can be eliminated. However, due to the impact from heavy axle loads and the depth of the muck in the subgrade (caused by both "free" water and capillary action), the layer of soil and cement injected under the slab by mudjacking is not a cure-all and generally lacks sufficient thickness to withstand heavy loads for long periods. A percentage of joints and cracks stabilized by mudjacking will in time resume pumping.

In view of the fact that a few of the "mudjacked" slabs will resume "pumping", it is our theory that mudjack work must be supplemented with some type of joint and crack waterproofing to keep the surface water from the subgrade and so obtain the best results in the maintenance of "slab pumping" pavements.

Keeping the joints and cracks in concrete pavement properly filled and waterproofed is a maintenance problem that has not been satisfactorily solved as yet, even though it has been a subject for research and field investigation for many years. However, with the advent of heavy axle loads and "pumping slabs", it seems imperative that every effort should be made to keep the joints and cracks filled and as waterproofed as possible in an attempt to prevent surface water from reaching the subgrade.

Depending on the degree of failure in the concrete pavement from "slab pumping", another method of waterproofing the joints and cracks is used. This is the placing of a bituminous surface or upper-deck on the pavement.

The degree of failure necessary to require upper-decking can best be described as that condition where the slab pumping has progressed to such a point that the riding surface of the pavement has been definitely impaired. With this condition of pavement prevalent, the best maintenance procedure is to follow the mudjack work with a bituminous deck of a thickness of 1-in. upward depending on the condition of the concrete pavement. It is desirable to prevent the cracks in the concrete pavement from continuing upward thru the bituminous surfacing. The thickness of the bituminous upper-deck will affect the number of cracks being reflected thru it from the base. Experiments are now being carried on to determine more accurately the effect of thickness of the bituminous surfacing on the number of cracks reflected thru it from the base. It is, however, easier to keep the cracks in bituminous surface sealed than those in concrete pavement.

It has, also, been noted that if the concrete pavement failure from "slab pumping" has progressed to such a point that the pavement has broken either at the corners or into narrow half-lane blocks, it is difficult to stabilize such areas by the use of the mudjack; and so the only solution in those cases is to replace the badly broken areas with concrete patches. The pavement surrounding a concrete replacement area is "mudjacked" so as to insure the minimum deflection of the old pavement in the vicinity of the new patch. This will prolong the life of the pavement surrounding the patch, as well as the patch itself. This mudjacking procedure is performed ahead of the concrete replacement work.

Thus it can be concluded that the proper maintenance methods for preventing and correcting the pumping action of concrete pavement slabs involves the first two and sometimes more of the following activities: (1) mudjacking, (2) joint and crack sealing, (3) patching full depth with concrete, (4) upper-decking with bituminous mats.

It is well to keep in mind in this work that the common goal in all highway maintenance is a smooth riding surface.

Up to this point attention has been devoted entirely to maintenance; but now, with the experience that has been obtained, it seems imperative that thought be given to the design and construction of concrete pavement to the end that "slab pumping" may be prevented.

It is generally accepted that "free" water under concrete pavement is one of the causes "slab pumping". The only way that "free" water can enter the subgrade from the surface is through the expansion and contraction joints, through natural contraction cracks, and along the edge of the concrete pavement. Therefore, the more joints and cracks present in the pavement surface, the more water will be found between the bottom of the pavement and the subgrade.

It is the opinion of our Bureau of Maintenance Engineers that the use of joints and/or contraction cracks should be omitted except at such locations where it is required to maintain adequate expansion at bridge ends, at the intersections of two concrete surfaced highways, and at other locations where the pavement is abutting any fixed object.

The theory for the use of expansion joints and contraction cracks may be well founded, in that they are designed to allow for pavement expansion and to control transverse cracks in the concrete pavement; but actually to our knowledge a joint has not been designed that is 100 per cent efficient. A joint to be 100 per cent efficient must transfer the load from one slab to the other with no deflection and be absolutely waterproof throughout its life in the pavement. To date these two features have not been attained in Missouri.

The maintenance of occasional concrete pavement "blow-ups" due to expansion, is certainly more economical and desirable than the maintenance of joints constructed at 20 to 50 ft. intervals. It has been the experience in this State that concrete pavements constructed with expansion joints are shorter lived, rougher riding, and more expensive to maintain than those constructed without joints. As a result, a Maintenance Engineer fails to see the justification of expansion joints, since the pavements with joints are also more expensive to construct.

The crack interval in concrete pavement has a very definite relation to slab pumping. The shorter the interval the more apt the pavement is to pump. Due to this fact, it is, therefore, advisable from the viewpoint of present slab pumping to have as great a distance as possible between cracks. This greatest distance possible can best be accomplished by not putting in contraction cracks. If contraction cracks are placed, it is necessary to predetermine the crack interval of a certain aggregate, and it almost invariably happens that additional contraction cracks occur between the installed contraction cracks. This results in a short crack interval that is so conducive to slab pumping. If the pavement is constructed without contraction cracks, the cracks will then form naturally at the greatest interval for the aggregate; which is the desired condition.

It so happens that we have two excellent examples in Missouri that very definitely show the effect of crack interval on pavement life. On Route 40 in Warren County there is a 17 mi. section of pavement, that was constructed in 1925 and 1926, using high expansive type coarse aggregate that causes a short interval - 22.9 ft. - between contraction cracks. It also happens that there is a half-mile section of pavement in this longer section constructed at the same time and under the same conditions, but using a low expansive type coarse aggregate which results in a longer crack interval - 76.8 ft. (See Figure 1). The difference in maintenance cost and pavement life has been very material. The pavement with the shorter crack interval has been patched with concrete, mudjacked and upper-decked, while only minor surface maintenance work has been done on the short section with the longer crack interval. A similar example can be found on Route 66 in Webster County. The crack interval in 1939 ranged from 21.6 ft. for the high expansive type coarse aggregate to 92.8 ft. for the low expansive type coarse aggregate. (See Figure 2.)

In view of the fact that a short crack interval encourages "slab pumping", it seems logical that considerable thought should be given to the aggregate used in the construction of concrete pavement.

Another of the generally accepted causes of "slab pumping" is unsuitable subgrade soil. This condition can be and is being corrected in many instances by the use of suitable subgrade soil or the proper stabilization of the subgrade prior to the construction of the concrete pavement.

It has been stated previously, but it can bear repeating, that the common goal of all highway maintenance activity is a smooth riding surface. This statement can be enlarged to read that the common goal of all highway design, construction, and maintenance activity is a smooth riding surface and maximum life. The elimination of expansion joints and contraction cracks, selection of proper aggregate, and the proper stabilization of the subgrade in the construction of concrete pavement will constitute a step in the direction of the goal of a long life and a smooth riding surface.

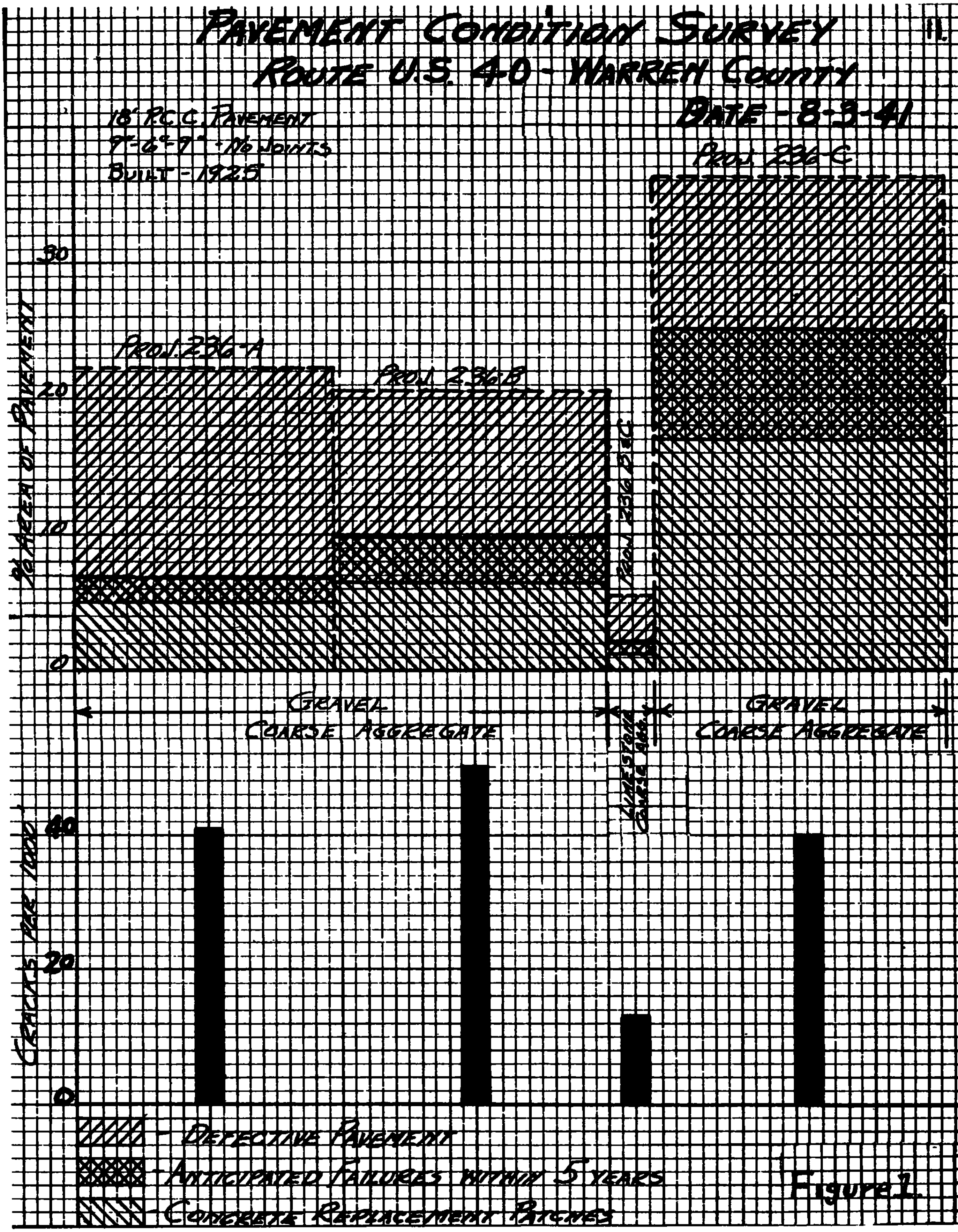

PAVEMENT CONDITION SURVEY
ROUTE U.S. 40 - WARREN COUNTY
DATE - 8-3-41
18' P.C.C. PAVEMENT
9"-6"-9" - NO JOINTS
BUILT - 1925
PROJ. 236-A
PROJ. 236-B
PROJ. 236 D & C
PROJ. 236-C
% AREA OF PAVEMENT
0
10
20
30
GRAVEL
COARSE AGGREGATE
LIMESTONE
COARSE AGG.
GRAVEL
COARSE AGGREGATE
CRACKS PER 1000'
0
20
40
- DEFECTIVE PAVEMENT
- ANTICIPATED FAILURES WITHIN 5 YEARS
- CONCRETE REPLACEMENT PATCHES

Figure 1.

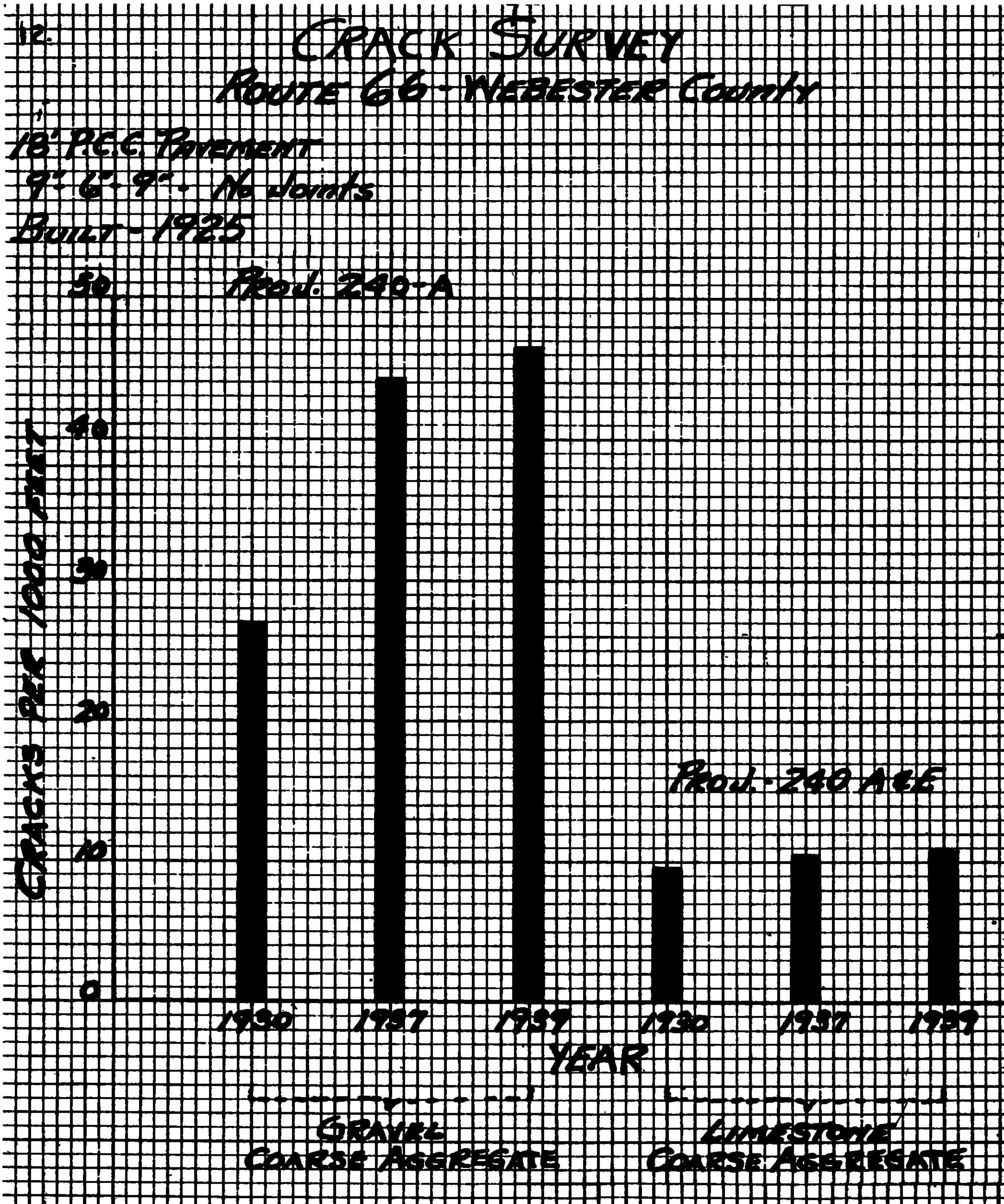

CRACK SURVEY
ROUTE 66 - WEBESTER COUNTY
18' P.C.C. PAVEMENT
9"-6"-9" - NO JOINTS
BUILT - 1925
PROJ. 240-A
PROJ. 240 A&E
CRACKS PER 1000 FEET
0
10
20
30
40
50
1930
1937
1939
1930
1937
1939
YEAR
GRAVEL
COARSE AGGREGATE
LIMESTONE
COARSE AGGREGATE

Figure 2

CORRECTING PAVEMENT PUMPING BY MUD JACKING

Robert E. Frost
Research Engineer, Purdue University

SYNPOSIS

This report covers some field experiments designed to correct the pumping action of rigid pavement slabs. In 1942 a performance survey on a portion of U.S. 30 between U.S. 41 and Valparaiso, Indiana, was made by representatives of the Joint Highway Research Project covering detailed analyses of pumping conditions on twenty-four miles of this road. Among other things the results of the survey showed that all of the experimental subgrade treatments (with the exception of the water-saturated section) were successful in minimizing or eliminating pumping and that pumping prevailed on untreated sections where the pavement was constructed on silty-clay soils.

Following this survey, a series of joints in a two-mile section near the Lake-Porter County line was selected for treatment by mud jacking. Treatment of these joints was performed in October and November of 1942. Four mixes were used:

1. Mix A; 77 per cent soil, 7 per cent RC-3, 16 per cent cement.
2. Mix B; 77 per cent soil, 7 per cent Road Oil, 16 per cent cement.
3. Mix C; 77 per cent soil, 7 per cent Tar, 16 per cent cement.
4. Mix D; 79 per cent soil, 3½ per cent Tar, 17½ per cent cement.

Even though work was hampered by cold weather and numerous equipment breakdowns, a total of 434 cu. ft. of mix was pumped under fifty pumping joints (an average of 8.7 cu. ft. per joint) in twelve working days.

Several performance surveys of this two-mile section have been made since treatment to determine the permanence of the treatments. The most recent survey (Oct.-Nov. 1944) showed that pumping had been reduced considerably. However, the installation of subgrade drains on U.S. No. 30 between S.R. 49 and S.R. 53 together with a particularly dry year (1944) made it difficult to rate the success on the basis of pumping alone. The settlement at the joints of both treated and untreated slabs showed considerable success for mud-jack treatment. It was found that the average settlement of the outer edges of treated slabs was 0.093 in. as compared to 0.194 in. for the untreated slabs. Further, it was found that 68 per cent of the treated slabs had settled 0.125 in. or less as compared to 53 per cent for untreated slabs.

A crack survey showed that mud jacking had been successful in reducing the expected number of cracks on this two-mile section. The data further show that cracks within 13 ft. of a joint are caused by slab movement and pumping, and that cracks in the middle third of a slab are from causes other than pumping. Of the four mixes used, those treated with Mix A and Mix D contained less cracks than those treated with Mix B and C.

Early in 1942 a performance survey was made on a portion of U.S. No. 30 located between U.S. No. 41 and a point just south of Valparaiso, Indiana (1).[a] A report of this survey stressed the seriousness of the pumping action taking place on the outside lanes of parts of this road - not five years old at that time. The report further contained suggestions for methods of treating pumping joints and presented an outline for a research program for the continuation of pumping studies. The results of this performance survey and others in Indiana and other states were presented at the November, 1943, Highway Research Board meeting in Chicago (2).

Among the new studies were suggestions for treatment of the slabs by mud jacking and by the installation of French drains. As a result of this report, arrangements were made to conduct mud-jacking research on a section of this road near the Lake-Porter County line. A two-mile section of road was selected for experimentation in which various mixes of soil, cement, and bituminous materials were to be forced under some of the slabs that had settled due to pumping action.

In October and November, 1942, 50 pumping joints were treated by mud jacking. The remainder of the joints in the two-mile section were left for comparative purposes. In addition, several joints were treated by draining with various types of drains. This report is a detailed study of that two-mile section of U.S. No. 30 under observation showing how both the treated and untreated slabs have performed during the two-year period following treatment.

U.S. HIGHWAY NO. 30

This report is concerned with a portion of U. S. No. 30 - a four-lane-divided-concrete pavement in Lake and Porter Counties extending from U.S. No. 41 to Valparaiso, Indiana, a distance of about 24 miles. This portion of roadway is of modern design in which the 200-ft. right-of-way contains two, 22-ft. slabs separated by a 44-ft. dividing strip and a 56-ft. strip on either side of the pavement. The pavement slabs are 9-in. - 7-in. - 9-in. in section, are reinforced with steel mesh, and were constructed with joints. The first portion of this road was constructed in 1937 and the last was completed in 1940. Several experimental-subgrade-treatment sections were installed on the western end of the road. Seven types were employed including saturating the subgrade with water, treating with bituminous materials (AES-1, T-C, MC-1), and replacing the subgrade with granular materials (sand, limestone dust, and crushed limestone).

The soils and topography of the area crossed by U.S. No. 30 owe their engineering characteristics to glacial activity. The road crosses or follows a large glacial deposit known as the Valparaiso Moraine. The topography varies from level to quite rolling. The topography near Valparaiso is somewhat rugged as the road crosses lake beds, terraces, sand dunes, and morainic deposits. From the junction of S.R. 330 to U.S. No. 41 the topography is level to moderately rolling and near U.S. No. 41 it becomes more rolling as the road crosses a series of sand dunes. The engineering characteristics of the soils contained in this area vary according to their origin and

[a] Numbers refer to the list of references at the end of the report.

topographic position. The soils of the morainic areas vary from granular materials to silts, clays, and silty clays. The drift soils of the moraine proper contain a considerable amount of shale fragments which weather readily to silty clay and clay. In general, the presence of this weathered shale has produced a plastic, poorly-drained soil in both the upland flat and the rolling moraine, the only difference being that the weathered profiles of the soils on the upland flat to rolling topography are much more developed and extend to greater depths than do the weathered profiles of the soils in the more rolling morainic areas. However, in some of the more rolling (nearly rugged) areas, the soils vary considerably. Some highway cuts show laminated soils while others contain silt, clay, or gravel pockets intermixed - all of which tend to add to the complexity of the engineering problems. Frost heaving occurs where the grade intersects or comes within a few feet of silt deposits and pumping occurs in areas of impervious, plastic, silty clays.

The dunes of both the east and west ends of this 24-mile section consist of windblown sand and are of post-glacial origin. The highway also crosses a few muck deposits near Valparaiso.

MUD JACK RESEARCH

Several states have tried mud jacking with soil-cement-bituminous mixtures as means of correcting pavement pumping. A description of some of the mixtures used by several states is given in the "Wartime Road Problems," No. 4, on pumping corrections, published by the Highway Research Board(3). The results of the April, 1942, performance survey showed the need for continuing studies on this section of U.S. No. 30 particularly in conducting research in mud jacking several pumping slabs.

Through co-operation with the LaPorte District and the Valparaiso Sub-district, many drainage and mud-jacking experiments were installed on a portion of U.S. No. 30 near the Lake-Porter County line for the purpose of combating pumping. Field operations were started on October 21, 1942, and were completed on November 23, 1942. Even though the research was conducted under adverse weather conditions accompanied by numerous equipment failures, the following was accomplished during 12 working days:

1. 36 French drains were installed.
2. 50 joints were treated by mud jacking.
3. 434 cu. ft. of mud-jack mixture were pumped under faulted slabs.

In June, 1942, 47 drains were installed by maintenance crews at some of the more severely pumping joints. During October and November, 1942, many of these drains were inspected and it was found that several were effective in stopping pumping. However, in locations of flat topography where drainage conditions were poor, many of the drains had clogged and the joints were pumping again.

The two-mile section selected for treatment contained soils that, for the most part, are similar in origin, texture, and engineering characteristics. Since traffic, rainfall, and soils do not vary in this section, the principle variables incorporated in the project were: the proportions and ingredients of the mud-jack mix; the operation procedure; the design of the drains installed; and the stone sizes in the drains.

The mud-jacking equipment used in conducting this research consisted of the following (See Figs. 1 and 2.):

1. Ingersoll-Rand air compressor operating at about 80 pounds, mounted on a truck.

2. A compressed air drill, operated at about 80 pounds and equipped with several 2¼-in. drill bits.

3. Mud jack apparatus consisting of a pressure chamber mixer, hose, nozzle, and various compressed-air fittings and lines all mounted on a heavy four-wheel trailer.

4. A 500-gallon water tank mounted on a truck.

5. Various types of small concrete mixers.

6. Regular highway maintenance truck for transporting soil, stone, and cement supply.

7. Tar kettle.

8. Wheelbarrow, spades, shovels, auger, and miscellaneous tools.

The soil used in conducting the mud-jacking research consisted of 46-per cent sand, 34-per cent silt, and 20-per cent clay. This soil was found to have a liquid limit of 29.3 and a plastic limit of 22.2. In choosing soil for this purpose an attempt was made to select material free from organic acids and solids.

The mixes used are shown in Table I. Four different mixes, A, B, C, and D, were employed in the entire project. Three different bituminous materials, A, B, and C, were used. These were RC-3, road oil, and tar, respectively. The fourth mix, D, contained one-half as much tar as mix C. The percentages were calculated on a dry-weight basis. However, for convenience in the field, the materials were proportioned in calibrated five-gallon buckets. For the first three mixes, A, B, and C, the percentages of materials remained the same. They were: soil - 77 per cent; bituminous materials - 7 per cent; and portland cement - 16 per cent. On a bucket basis, this was roughly eight, five-gallon buckets of soil; one, five-gallon bucket of bituminous material, and one bag of cement. Mix D called for one-half of a five-gallon bucket of tar.

Table I.

MUD-JACK ADMIXTURES USED AND WORK RECORD

Mix	Parts	Field Use - 5 gal. Bkt.	% by Vol.	Days	Joints Pumped	Cu. Ft. Pumped	Av. Mix per Joint
A	Soil	8 - 5 gal. Buckets	77	3	7	105.8	15.1
	Water	3-4 - 5 gal. Buckets					
	RC-3	1 - 5 gal. Bucket	7				
	Cement	1 Bag	16				
B	Soil	8 - 5 gal. Buckets	77	5	22	150.4	6.8
	Water	3-4 - 5 gal. Buckets					
	Road Oil	1 - 5 gal. Bucket	7				
	Cement	1 Bag	16				
C	Soil	8 - 5 gal. Buckets	77	2	16	128.4	8.0
	Water	3-4 - 5 gal. Buckets					
	Tar	1 - 5 gal. Bucket	7				
	Cement	1 Bag	16				
D	Soil	8 - 5 gal. Buckets	79	1	4	37.4	9.4
	Water	3-4 - 5 gal. Buckets					
	Tar	½ - 5 gal. Bucket	3½				
	Cement	1 Bag	17½				
Note:	1 joint "B" + "C" Mix			1	1	12.3	12.3
			TOTALS	12	50	434.3	8.7

Average joints per day.............................. 4+
Average mix per joint............................... 8.7 cu. ft.
Average mix per day.................................36.2 cu. ft.
Drains installed in June, 1942......................47
Drains installed in Oct.-Nov., 1942.................36

For convenience in making future observations of the treated joints, large letters, A, B, C, and D were painted on the pavement opposite each joint indicating which mix was used.

It was found that the amount of mixing water needed depended on the prevailing air temperature and the amount of water in the soil. If the weather was warm, it was noted that the bituminous materials mixed readily with the soil and water. When the weather was cold, the hot bituminous material did not mix readily with the soil and water. The chilling of the bituminous material seemed to stiffen the mix, thus requiring more water to obtain a creamy consistency. In addition, the bituminous material would become stringy and difficult to mix and pump.

The best mixing procedure developed was as follows:

1. A soil-water mix was made in which just enough water was added to obtain a fluid mix.

2. Bituminous material and additional water were added as needed, to obtain a uniform mix.

3. The cement was added. (The consistency should be such that the mix can be readily pumped through the hose and under the pavement.)

4. It was found that prolonged mixing should be avoided.

The following is a description of operations as they were performed on a typical joint during the experimental work.

1. Holes were drilled at desired joints.

2. A trench was dug along the pavement where pumping was severe.

3. If a drain was desired, or if there was considerable water under the pavement, a lateral trench was dug to the ditch line.

4. The mixture was then poured into the mud-jack hopper.

5. Air pressure was gradually applied which forced the mixture through the hose and under the pavement.

   Note: It was found that if too much pressure was suddenly applied, the mix would blow out the sides and out the cracks and other holes in the pavement and raise the pavement too fast, which in one case broke it.

6. A small amount of mixture was pumped in each hole, gradually raising each side to grade. On several occasions the material could be seen extruding in a long ribbon from under the pavement (in the open trench, (See Fig. 3). When this condition was reached before the slab was raised to the desired elevation the trench would then be filled and compacted in order to confine the mud-jack mixture.

7. As soon as the mud-jack nozzle was withdrawn from the hole in the slab, large wooden wedges were driven in the holes. These prevented the mixture from being forced out on the pavement by the pressure of the slab weight. The holes were left plugged about six or seven hours (See Fig. 4).

8. Traffic was kept off for a period of about two days. This allowed the mixture to harden.

Many methods of operation were tried. Included were the following:

1. Mud jacking without pre-draining the joint.

2. Mud jacking with pre-drainage of the pumping area.

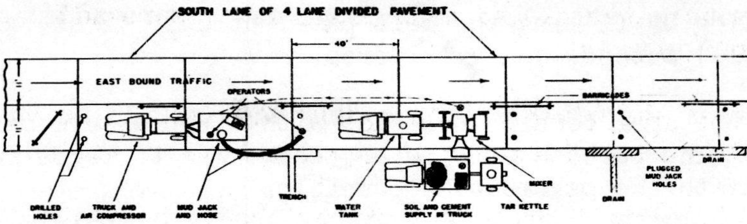


FIGURE 1. DIAGRAM SHOWING EQUIPMENT LAYOUT FOR MUD-JACK RESEARCH ON U. S. 30.

FIGURE 2. MUD-JACKING EQUIPMENT - AIR COMPRESSOR, MUD-JACK, WATER TANK MIXER, TAR KETTLE, AND SUPPLY TRUCK.

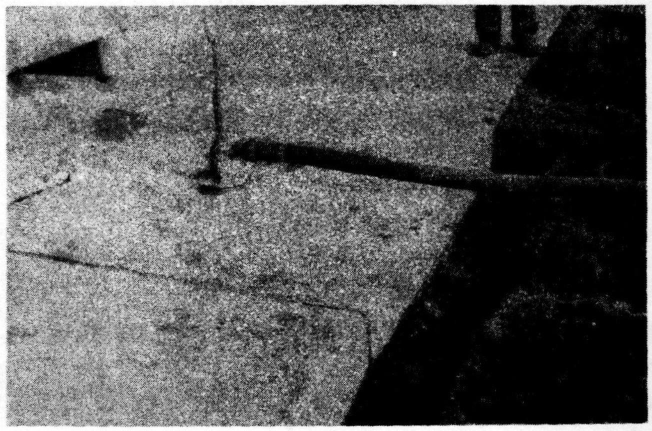


FIGURE 3. SUCCESSFUL MUDJACKING. HERE THE SLAB HAS BEEN RAISED TO GRADE AND A LONG RIBBON OF THE MUD-JACKED MIXTURE IS SEEN EXTRUDED FROM BENEATH THE SLAB.

FIGURE 4. THE HOLES WERE PLUGGED TO PREVENT THE SLAB FROM SETTLING AND SQUEEZING OUT THE MIX UPON COMPLETION OF MUD-JACKING.

3. Mud jacking by filling the left pavement hole first, and then the right (edge hole) second, and vice versa.

4. Venting the area beneath the slab with air pressure before pumping the mixture. The purpose of this procedure was to force as much water out from beneath the pavement as possible.

5. Admixture variations, such as tar, RC-3, and road oil.

6. Mud jacking at a joint with and without supplemental drainage.

7. Drainage without mud jacking.

8. Some severely pumping joints were left untreated.

The four types of drains used in this project were: dry well; French drain with a lateral at the joint; French drain with a lateral below the joint; and French drain consisting of a lateral only.

1944 PERFORMANCE SURVEY OF 24 MILES

In October 1944, two and one-half years after the first survey and two years after mud jacking, another survey was made of U.S. No. 30. A detailed survey of the entire 24 miles was not made, but several pictures were taken to show the condition of many of the joints. It was observed that pumping had progressed at a rapid rate and had become a serious problem. Both the north and south (particularly the outside) lanes were suffering from pumping but pumping was more severe on the south lane. At present, this 24-mile pavement is seven years old and has served as an excellent "test track". Because of the variations in design and construction of the highway, it has been possible to study the pumping problem, its causes, effects, and, to a limited extent, its cures. Since the entire 24 miles carries the same amount of traffic (with the exception that the south lane carries the heavier truck traffic) and receives the same amount of rainfall, it has been possible to evaluate the various types of subgrade soils. A visual inspection of the 24-mile section showed no pumping on the natural sand section, the 6-in. sand section, or the 3-in. stone stabilization section. Pumping was very slight at a few of the joints on the limestone, AES, and the TC sections. Pumping was exceptionally severe on the water-saturated section and several of the slabs had settled sufficiently to warrant patching. Figures 5 and 6 show the condition of one of the slabs on the water-saturated subgrade in October, 1944.

Pumping was found to be severe in the silty-clay drift areas. Figures 7 and 8 are views of the same joint taken in 1943 and 1944 and show the extent of the failure of some of the severely pumping joints on drift soils. One section of this road, constructed on silty-clay drift, contained a total of 50 patched slabs in a distance of 2700 ft. (See Fig. 9).

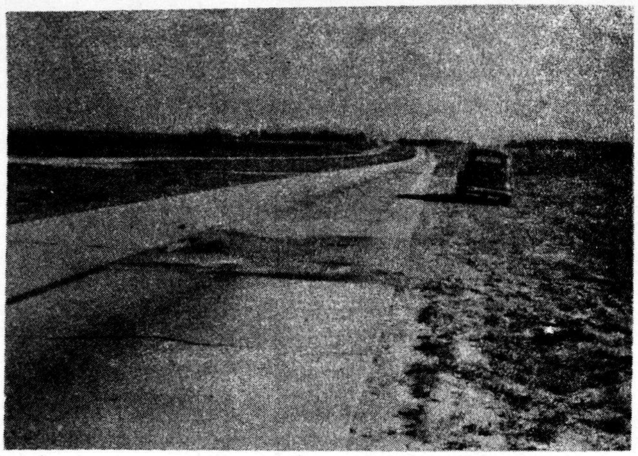


FIGURE 5. PERFORMANCE IN THE WATER-SATURATED SUBGRADE SECTION IN OCTOBER 1944.

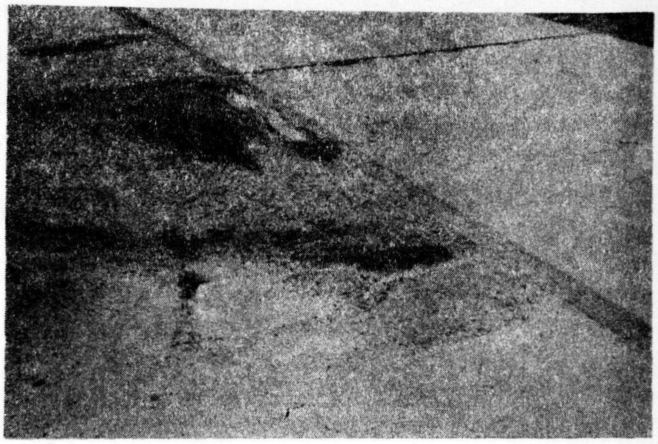


FIGURE 6. CLOSE-UP OF A JOINT IN THE WATER-SATURATED SECTION (OCTOBER, 1944).

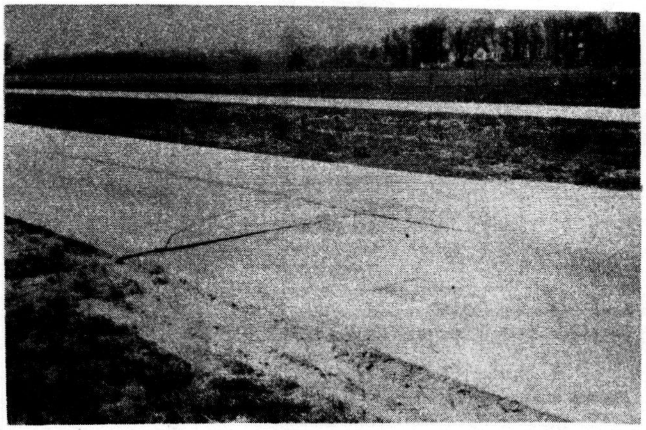


FIGURE 7. VIEW OF A SEVERELY PUMPING SLAB IN DRIFT SOIL OF THE VALPARAISO MORAINE IN NOVEMBER, 1943.

FIGURE 8. VIEW OF THF PATCH ON THE SAME SLAB (FIGURE 7) IN NOVEMBER, 1944. PUMPING IS STILL SEVERE.

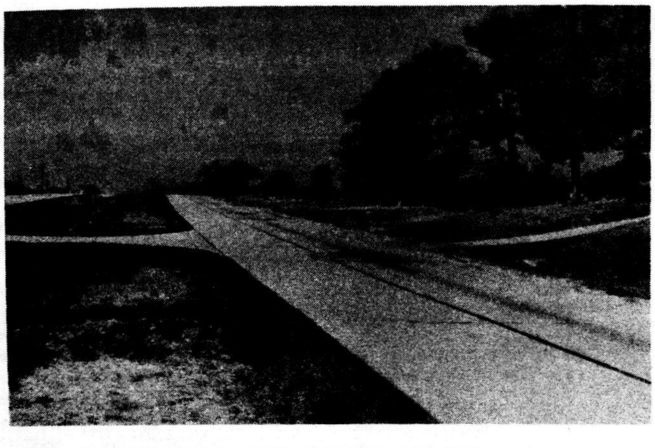


FIGURE 9. A SERIES OF PATCHES AT JOINTS OF SLABS THAT HAVE SETTLED DUE TO PUMPING. ON ONE SUCH SECTION A TOTAL OF 50 PATCHES WAS COUNTED IN 2700 FEET.

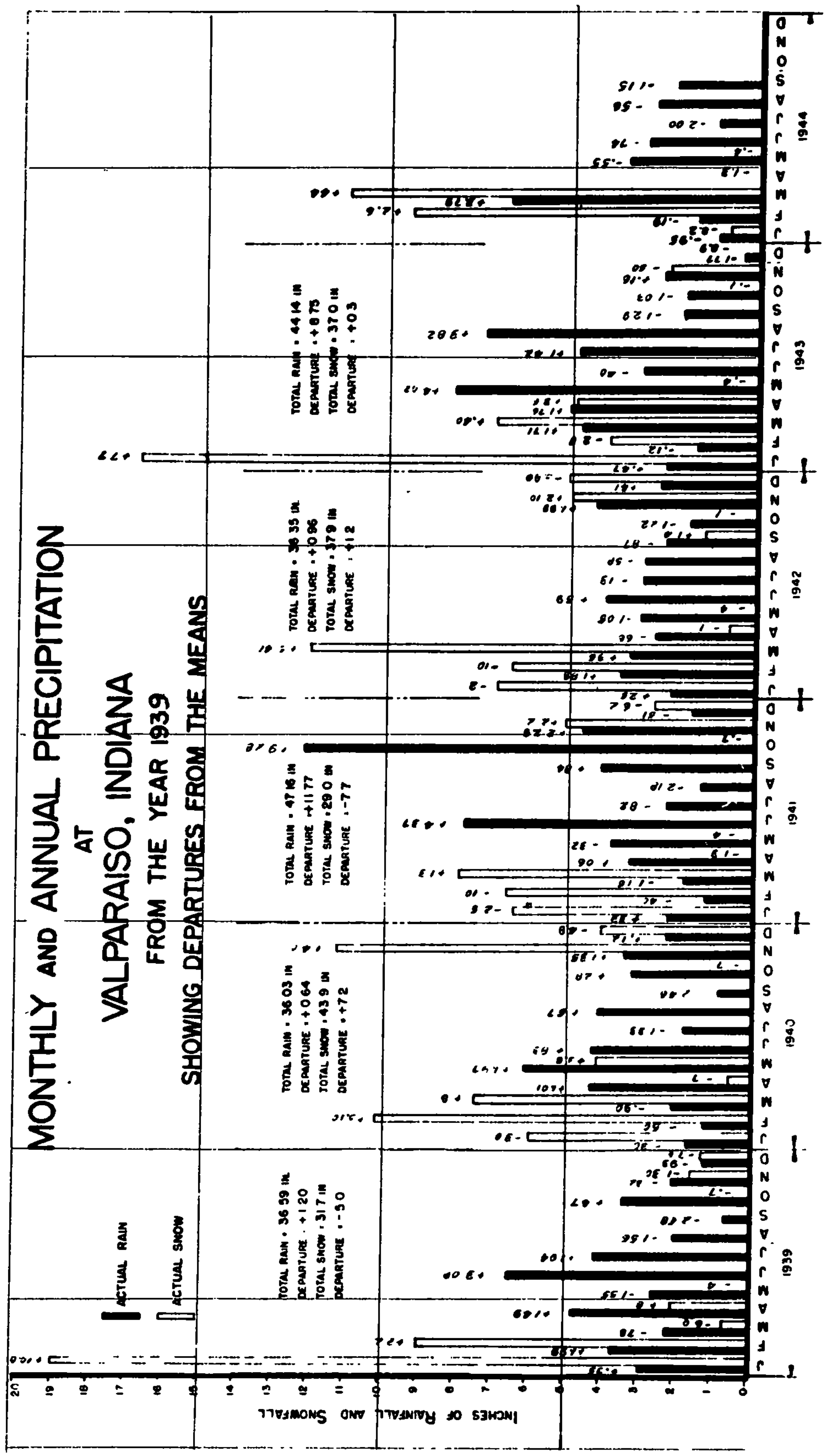

MONTHLY AND ANNUAL PRECIPITATION
AT
VALPARAISO, INDIANA
FROM THE YEAR 1939
SHOWING DEPARTURES FROM THE MEANS
ACTUAL RAIN
ACTUAL SNOW
INCHES OF RAINFALL AND SNOWFALL
TOTAL RAIN - 36.59 IN. DEPARTURE - +1.20 TOTAL SNOW - 31.7 IN DEPARTURE - -5.0
TOTAL RAIN - 36.03 IN DEPARTURE - +0.64 TOTAL SNOW - 43.9 IN DEPARTURE - +7.2
TOTAL RAIN - 47.16 IN DEPARTURE - +11.77 TOTAL SNOW - 29.0 IN DEPARTURE - -7.7
TOTAL RAIN - 36.35 IN. DEPARTURE - +0.96 TOTAL SNOW - 37.9 IN DEPARTURE - +1.2
TOTAL RAIN - 44.14 IN DEPARTURE - +8.75 TOTAL SNOW - 37.0 IN DEPARTURE - +0.3
1939
1940
1941
1942
1943
1944

FIGURE 10.

PERFORMANCE SURVEYS OF THE MUD-JACK SECTION

In the October, 1942, survey, previous to mud jacking, all cracks were stationed and counted, and all joints were rated as to degree of pumping (slight, p-1; moderate, p-2; severe, p-3). Six months after treatment, in May, 1943, another performance survey was conducted on the same two-mile section. In this survey new cracks were stationed and counted and again the joints were rated according to degree of pumping.

The next survey was conducted in October, 1944 (just two years after treatment) during which time pumping was found to be slight. The results of this survey were somewhat difficult to evaluate since drainage had been installed beneath the outside edge of the slabs for the major portion of the road from Valparaiso to U.S. No. 41. These drains and a deficiency of rainfall during the months of May to September, 1944, may account for the fact that pumping was not observed at the time of the October survey. During that period 14.26 in. of rain fell, which was 72 per cent of the normal rainfall for that period (a deficiency of 4.53 in.). See Figure 10 for a rainfall curve covering the period 1939 to 1944.

In view of these two conditions, a rating of the degree of pumping during that survey cannot be taken as a measure of the success or failure of treatment. However, a crack survey was also made in which the development of all new cracks was observed in both the treated and untreated section. (For purposes of comparison, a detailed crack survey was made on the 6-in. sand experimental section.) In addition, the settlement at each joint, inner edge and outer edge of the slab, was noted for both treated and untreated joints.

On November 6, 11, and 27, similar surveys were conducted (pumping only) since that area was receiving rainfall. These three additional surveys were made for the purpose of studying the effectiveness of the drainage system.

Table 2 shows a comparison of the 1942, 1943, and the 1944 pumping surveys. Table 3 contains results of pumping surveys of the 50 treated joints only. Tables 4 and 5 contain a summary of the settlement survey made in 1944 for both the treated and untreated joints. Table 6 is an analysis of cracking in cut and fill areas. Figures 13 to 22 are photographs of typical treated and untreated joints. Figure 23 is a distribution of cracks in 59 slabs constructed on the 6-inch sand section. Figures 24 to 34 are a set of distribution curves that help in the analyzing of the cracking of slabs with respect to pumping.

Pumping Surveys

The first performance survey of the two-mile mud-jack section made in October, 1942, prior to treatment, showed that 69 per cent of 277 joints were pumping. (There are 280 joints between station 610+10 and 722+86, but three in the Deep River Bridge are not included.) Of that number, 31.7 per cent were pumping slightly, 26.7 per cent moderately, and 11.2 per cent severely. (See Table 2). Table 3 shows the results of a pumping survey of the 50 joints that were treated later. This table shows that all

Table 2.

PUMPING SURVEY OF 2-MILE SECTION OF U.S. 30
(This table shows the number and percentage of joints pumping between stations 610+10 and 722+86. The table includes 50 joints treated by mud jacking.)

Degree of Pumping	OCTOBER, 1942 Before Treatment		MAY 6, 1943 6 Months After		OCTOBER 11, 1944 2 Years After	
	No.	Percent	No.	Percent	No.	Percent
0-None	84	30.3	96	34.6	259	93.5
1-Slight	88	31.7	112	40.4	7	2.5
2- Moderate	74	26.7	57	20.5	5	1.8
3-Severe	31	11.2	11	3.9	0	0
4-Very Severe	0	0	1	0.5	6	2.1
TOTALS	277	99.9	277	99.9	277	100.9

Degree of Pumping	NOVEMBER 6, 1944 After 0.71 in.*		NOVEMBER 11, 1944 0.71+0.43= 1.14 in.		NOVEMBER 27, 1944 1.14+1.21=2.35 in.	
	No.	Percent	No.	Percent	No.	Percent
0-None	260	93.8	259	93.5	256	92.4
1-Slight	7	2.5	6	2.1	12	4.3
2-Moderate	3	1.0	4	1.4	5	1.8
3-Severe	1	0.5	1	0.5	1	0.5
4-Very Severe	6	2.1	7	2.5	3	1.0
TOTALS	277	99.9	277	100.0	277	100.0

*The October 11, 1944, survey showed that pumping had practically stopped. Between October 11 and November 6, a total of 0.71 in. of rain fell and another survey was made. Between November 6 and 11 an additional 0.43 in. of rain fell, and another survey was made. The last survey was made on November 27 after an additional 1.21 in. (total 2.35 in.) of rainfall.

Table 3 - PUMPING SURVEY OF 50 JOINTS TREATED BY MUD JACKING

Degree of Pumping	OCTOBER, 1942							MAY 6, 1943						
	Before Mud Jacking							Six Months After Treatment						
	Mix					Total		Mix					Total	
	A	B	B+C	C	D	Joints	%	A	B	B+C	C	D	Joints	%
0-None	0	0	0	0	0	0	0	3	9	1	7	1	21	42
1-Slight	0	5	1	8	2	16	32	3	9	0	5	3	20	40
2-Moderate	1	8	0	6	2	17	34	1	4	0	2	0	7	14
3-Severe	6	9	0	2	0	17	34	0	0	0	2	0	2	4
4-Very Severe	0	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	7	22	1	16	4	50	100	7	22	1	16	4	50	100

Degree of Pumping	OCTOBER 11, 1944							NOVEMBER 6, 1944						
	Two Years After Treatment							After 0.71 inches Rain*						
	Mix					Total		Mix					Total	
	A	B	B+C	C	D	Joints	%	A	B	B+C	C	D	Joints	%
0-None	6	20	1	16	4	47	94	5	20	1	16	4	46	92
1-Slight	0	0	0	0	0	0	0	1	0	0	0	0	1	2
2-Moderate	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3-Severe	1	2	0	0	0	3	6	0	0	0	0	0	0	0
4-Very Severe	0	0	0	0	0	0	0	1	2	0	0	0	3	6
TOTAL	7	22	1	16	4	50	100	7	22	1	16	4	50	100

Degree of Pumping	NOVEMBER 11, 1944							NOVEMBER 27, 1944						
	0.71" + 0.43" = 1.14" Rain							1.14" + 1.21" = 2.35" Rain						
	Mix					Total		Mix					Total	
	A	B	B+C	C	D	Joints	%	A	B	B+C	C	D	Joints	%
0-None	5	20	1	15	4	45	90	5	18	1	15	4	43	86
1-Slight	1	0	0	0	0	1	2	1	2	0	1	0	4	8
2-Moderate	0	0	0	1	0	1	2	0	0	0	0	0	0	0
3-Severe	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4-Very Severe	1	2	0	0	0	3	6	1	2	0	0	0	3	6
TOTAL	7	22	1	16	4	50	100	7	22	1	16	4	50	100

*The October 11, 1944, survey showed that pumping had practically stopped. Between October 11 and November 6, a total of 0.71 in. of rain fell and another survey was made. Between November 6 and 11 an additional 0.43 in. of rain fell, and another survey was made. The last survey was made on November 27 after an additional 1.21 in.(total 2.35 in.) of rainfall.

TABLE: 4 SUMMARY OF SETTLEMENT SURVEY

Settlement in Inches	UNTREATED - 227						TREATED - 50					
	INNER			OUTER			INNER			OUTER		
	NO	%	ACC %	NO	%	ACC %	NO	%	ACC %	NO	%	ACC %
0	73	32.7	32.7	58	26.1	26.1	24	48	48	24	48	48
1/8	96	31.8	74.5	63	27.5	53.6	13	26	74	10	20	68
1/4	35	15.3	89.8	42	21.3	74.9	6	12	86	2	4	72
3/8	7	3.0	92.8	27	18.8	83.7	1	2	88	4	8	80
1/2	5	2.2	95.0	19	8.2	91.9	2	4	92	5	10	90
5/8	1	0.4	95.4	5	2.2	94.1				0		
3/4	2	0.9	96.3	2	0.9	95.0				1	2	92
7/8				1	0.4	95.4						
1				2	1.0	96.4						
PATCH*	8	3.4	99.8	8	3.4	99.8	4	8	100	4	8	100
TOTAL	227	100 %	100 %	227	100 %	100 %	50	100 %	100 %	50	100 %	100 %
AV.	0.123"			0.194"			0.098"			0.093"		

*See Text for Patch Explanation.

TABLE 5 - SUMMARY OF SETTLEMENT SURVEY
50 MUD-JACKED JOINTS

Settlement	MIX "A"						MIX "B"					
in	INNER			OUTER			INNER			OUTER		
Inches	NO.	%	ACC %	NO.	%	ACC %	NO.	%	ACC %	NO.	%	ACC %
0	5	71	71	4	57	57	10	45	45	9	41	41
1/8	1	14	85	2	28	85	5	23	68	4	18	59
1/4							3	13	81	1	5	64
3/8							1	5	86	2	10	74
1/2										3	13	87
5/8												
3/4												
7/8												
1												
PATCH*	1	14	99	1	14	99	3	13	99	3	13	100
TOTAL	7	99	99	7	99	99	22	99	99	22	100	100
AV.	0.02"			0.03"			0.08"			0.13"		

*See Text for Patch Explanation
(1 Joint - Mix B+C Had Not Settled)

Settlement	MIX "C"						Mix "D"					
in	INNER			OUTER			INNER			OUTER		
Inches	NO.	%	ACC %	NO.	%	ACC %	NO.	%	ACC %	NO.	%	ACC %
0	5	31	31	8	50	50	3	75	75	2	50	50
1/8	5	31	62	2	12.5	62.5	1	25	100	2	50	100
1/4	4	25	87	1	60	69						
3/8				2	12.5	81.5						
1/2	2	12.5	99.5	2	12.5	94						
5/8												
3/4				1	6	100						
7/8												
1												
PATCH*												
TOTAL	16	100	100	16	100	100	4	100	100	4	100	100
	0.16"			0.19"			0.03			0.04		

*See Text for Patch Explanation
(1 Joint - Mix B+C Had Not Settled)

TABLE 6 - COMPARISON OF CRACKS IN CUT AND FILL ON A TWO MILE SECTION OF U.S. 30

Date	Pavement Data:	Cut	Fill	All
	No. Slabs	134	143	277 *
Oct.	No. Cracks	222	244	466
1942	No. Cracks Per Slab	1.65	1.70	1.68
	No. Slabs	134	143	277
May	New Cracks	11	19	30
1943	No. Cracks	233	263	496
6 Mo.	No. Cracks Per Slab	1.73	1.83	1.79
	No. Slabs	134	143	277
Oct.	New Cracks	110	91	201
1944	No. Cracks	343	354	697
2 Yr.	No. Cracks Per Slab	2.59	2.47	2.51

*There are 280 slabs in the section under observation. 3 of these are in the Deep River Bridge.

of the joints selected for treatment were pumping, and that one-third of them were pumping slightly, one-third moderately, and one third were pumping severely.

On May 6, 1943, six months after treatment, another pumping survey was conducted on the 277-slab section. This survey showed that of the 277 slabs, 65.3 per cent were pumping (Table 2). However, of the 50 slabs treated (Table 3), 42 percent were not pumping, 40 percent were pumping slightly, 14 percent moderately, and 4 per cent were pumping severely. Of the 68 percent of the joints pumping moderately and severely in 1942, only 18 percent were rated as pumping similarly in 1943.

Two years after treatment, the October, 1944, survey showed that 93.5 percent of the 277 slabs in the experimental section were not pumping (Table 2). A survey of the 50 treated joints showed that 94 percent were not pumping. Three of the 50 treated joints had failed to the extent that they had to be patched. However, study of the original work record shows that the failures are reasonable and might be expected (See Appendix for details).

Following the October, 1944, survey, three additional surveys were made at intervals of approximately one week since that area was receiving rainfall. Study of Table 3 shows that the additional rainfall caused a slight increase in pumping on both the treated and untreated joints.

It is difficult to measure the success of mud jacking by pumping surveys, since the effects of the relatively dry weather and the drainage installations are not known. However, pumping (or evidences of pumping) was noted at joints on other areas of the entire 24 miles of road.

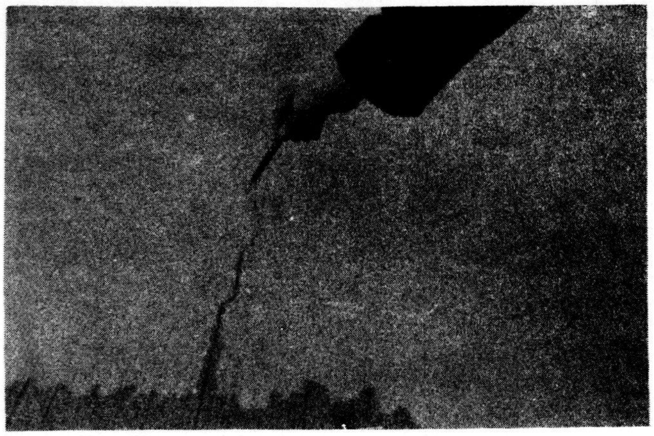


FIGURE 11. FIRST STAGES OF CRACKING DUE TO SLAB PUMPING.

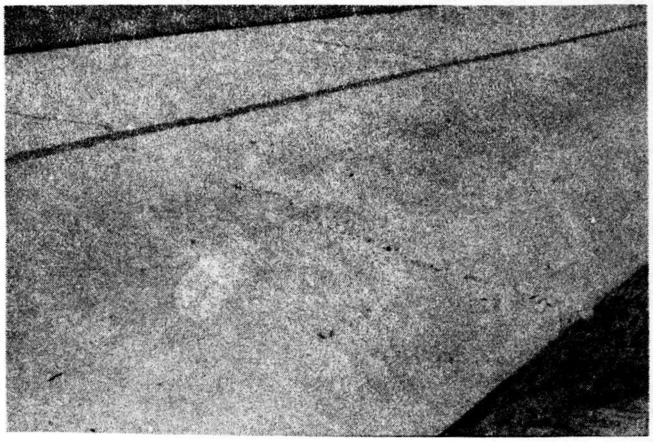


FIGURE 12. ADVANCED STAGES OF CRACKING IN WHICH SPALLING OCCURS CAUSED BY SEVERE SLAB MOVEMENT.

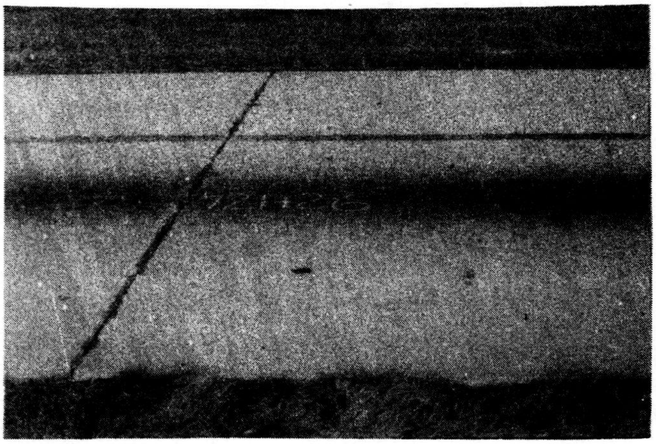


FIGURE 13. SUCCESSFUL MUD JACKING. THIS JOINT TREATED WITH MIX "A" HAD NOT SETTLED OR PUMPED DURING TWO YEARS OF SERVICE.

FIGURE 14. SEVERE PUMPING AT AN UNTREATED JOINT IN THE SECTION OF PAVEMENT IN WHICH JOINTS WERE TREATED WITH MIX "A"

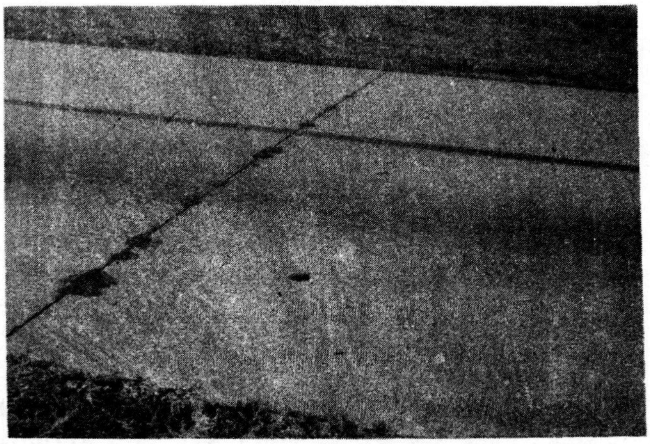


FIGURE 15. A SLAB TREATED WITH MIX "B" THAT HAS GIVEN TWO YEARS OF GOOD SERVICE.

FIGURE 16. AN UNTREATED SLAB (IN THE AREA IN WHICH THE JOINTS WERE TREATED WITH MIX "B") THAT HAD SETTLED 3/4 OF AN INCH.

FIGURE 17. THIS SLAB TREATED WITH MIX "B" HAD FAILED DUE TO AN EQUIPMENT FAILURE DURING TREATMENT.

FIGURE 18. THIS IS A VIEW OF THE FIRST CRACK PAST THE JOINT IN FIGURE 17.

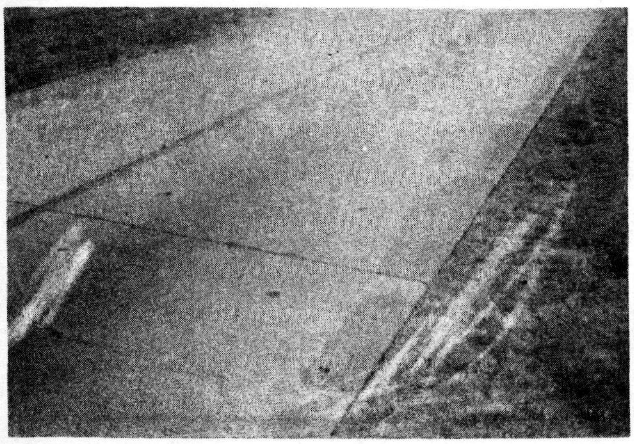


FIGURE 19. THE JOINT AT STATION 662+86 (MIX "C") HAS PERFORMED SATISFACTORILY FOR TWO YEARS.

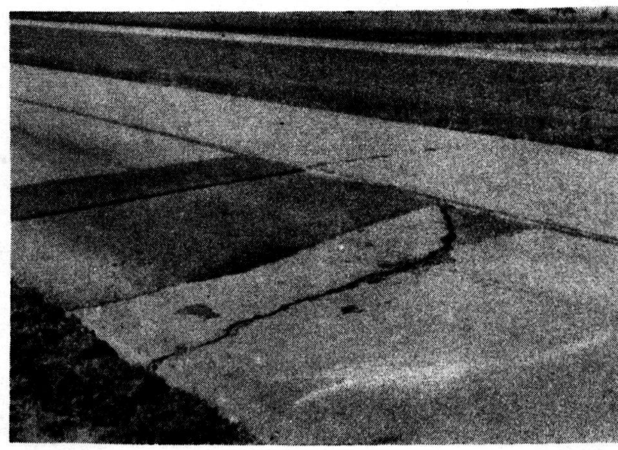


FIGURE 20. THE JOINT AT STATION 662+46 (ADJACENT TO SLAB IN FIG. 19 WAS NOT TREATED AND HAS GIVEN TWO YEARS OF UNSATISFACTORY PERFORMANCE.

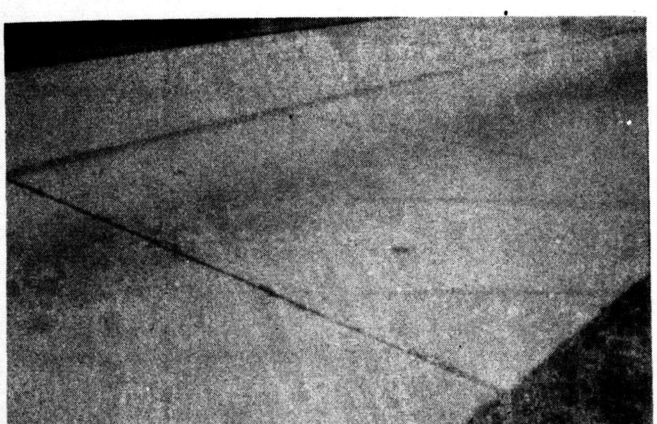


FIGURE 21. ANOTHER SUCCESSFULLY TREATED JOINT (MIX "C").

FIGURE 22. THIS JOINT, TREATED WITH MIX "D", PERFORMED SATISFACTORILY DURING TWO YEARS OF SERVICE.

Settlement Survey

Since the success of mud jacking could not be determined by rating the joints as to the degree of pumping, the settlement at each joint of both treated and untreated joints was measured. Since all slabs treated by mud jacking were raised to grade at the time of treatment, it is believed that this might be used as one measure of the success of treatment. The results of this survey are contained in Tables 4 and 5. Table 4 shows the settlement at the joints of the inner (centerline) and outer (shoulder) ends of all untreated slabs as compared to settlement of 50 mud-jacked joints. The table shows that the average settlement of the outer edge of 227 untreated slabs was 0.194 in. as compared with 0.093 in. for the 50 treated slabs./b The inner slabs had settled 0.123 in. for the untreated slabs and 0.098 in. for the treated slabs. The table shows that settlement at the outer edge of the slab is greater than the settlement at the inner edge. (This corresponds favorably to severity of pumping with respect to the shoulder and centerline.) Some of the untreated slabs had settled as much as one inch. (A few that were patched obviously had settled to a much greater degree but were not measured).

Comparing percentages of slabs settling in increments of one-eighth inch showed that 48 percent of the outer slabs of treated joints had not settled, and 20 percent had settled one-eighth inch. This means that in two years, 68 percent of the treated slabs (outer edge of the joints) had settled one-eighth inch or less. These data can be compared to 26 percent of the untreated joints that have not settled and 27 percent that had settled one-eighth inch; or, that 53.6 percent of the untreated slabs had settled one-eighth inch or less. It should be pointed out that the percentages are based on the total number of cases and that eight untreated joints and four treated joints requiring patches are included; also, that the table for the treated joints includes all four mixes, some of which have failed more than others.

Table 5 compares the settlement at the joints by mix design. This table shows that both mixes "A" and "D" were effective in stopping settling at the joints. Of the seven joints treated with mix "A", one had failed and required a patch (equipment failure), four had not settled (outer portion), and two had settled an eighth-inch. Of the four joints treated with mix "D", two had not settled and two had settled only an eighth-inch. The joints treated with mix "A" had an average settlement (outer) of 0.03 in. and the joints treated with mix "D" had an average settlement of 0.04 in.

Comparison of the settlement of joints treated with mix "B" and "C" shows that those treated with "C" had settled the most. The average settlement of both outer and inner portions of the slabs by mixes are: Mix "C", outer edge - 0.19 in., inner edge - 0.16 in.; mix "B", outer edge - 0.13 in., inner edge, 0.08 in. However, 50 percent of the joints treated with mix "C" had not settled and 62.5 percent had settled one-eighth inch or less. This is to be compared to 41 percent with a settlement of an eighth-inch or less.

/b - This includes treated slabs which had failed because of faulty treatment resulting from equipment failure.

Crack Survey

Detailed crack surveys of this two-mile section of U.S. 30 were made in 1942 (prior to treatment), in May, 1943 (six months after treatment), and in October, 1944 (two years after treatment). Each survey consisted of stationing all cracks and plotting their position with respect to joints on plan sheets of the two-mile section. The results of each survey have been analyzed to determine the number of new cracks occurring during the various periods following treatment and to determine their positions relative to joints. The crack survey data have been divided into the following:

1. Distribution of cracks on cut and fill (Table 6).
2. Cracks on the 6-in. sand section (Fig. 23).
3. Distribution of all cracks by years (Figs. 24 and 25).
4. Distribution of cracks on the forward and rear slabs by years (Figs. 26 and 27).
5. Distribution of all cracks within 13 feet of the joints (Figs. 28, 29, and 30).
6. Distribution of new cracks - those occurring in two years only (Figs. 31 to 34).

Each of the above has been further divided into treated and untreated joints so that the effect of treatment on cracking can be studied.

In order to study the possibility of the cracking in cuts exceeding the cracking in fills a table was prepared in which these two variables are compared. Study of Table 6 shows that there are 134 slabs in cut areas and 143 in fill areas, and that by 1944 the "cut" slabs had 343 cracks and the "fill" slabs had 354 cracks. This small difference of the cracking in the cut and fill areas shows that cut or fill sections had little or nothing to do with the cracking in this particular area. This can be explained because of the similarity of engineering properties of the soils of both cut and fill areas.

On the 6-in. sand-treatment section all of the cracks in the 40-ft. slabs occurred in the middle third of the slabs. Figure 23 shows the distribution of cracks in the slabs in the 6-in. sand section. There are only two cracks less than 13 ft. either side of a joint in fifty-nine, 40-ft. slabs of this section. Further, it was found that after seven years, 42 percent of the slabs had no cracks and 47 percent contained only one crack. Since this section receives the same traffic and rainfall, is of the same pavement design, and was constructed approximately at the same time as the two-mile section under observation, it is believed that the cracking distribution on this section can be taken as typical of a pavement placed on a firm, well-drained support. Assuming this to be the case, cracks occurring less than 13 ft. either side of a joint can be considered as the result of slab movement caused by pumping action and that cracks occurring in the middle third of a 40-ft. slab are the result of other causes.

YEAR BUILT: 1937

DATE OF SURVEY: DEC. 1944

NO. CONSTRUCTION SLABS	= 4	
NO. 40 FT. SLABS IN JOB	= 59	= 100%
NO. 40 FT. SLABS WITH NO CRACKS	= 25	= 42%
NO. 40 FT. SLABS WITH 1 CRACK	= 28	= 47%
NO. 40 FT. SLABS WITH 2 CRACKS	= 6	= 11%

6" SAND SECTION
START: STA 145 + 01
END: STA 170 + 16

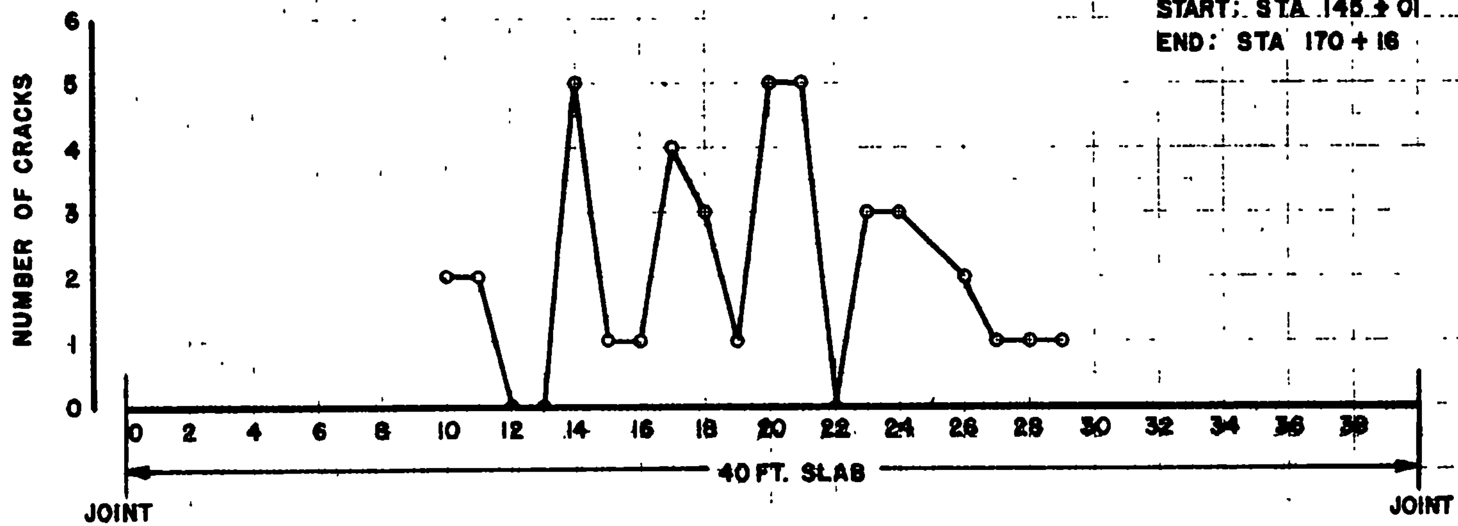


FIGURE 23. DISTRIBUTION OF CRACKS IN 59 SLABS IN 6 INCH SAND SECTION

Figures 24 and 25 show the distribution of all cracks (both forward and rear slabs combined) with respect to any joint for both treated and untreated joints for the 1942 and 1944 surveys. Both curves show a decided drop in the number of cracks at 13 ft. in the distribution curves. It is interesting to note, by comparing the two curves in each figure, that the cracking less than 13 ft. from the joint was considerably more in 1944 than in 1942 and that cracking past 13 ft. from the joints remained fairly constant during the same two-year period. This observation confirms the statement that cracking up to 13 ft. from the joint must be directly influenced by slab movement initiated by pumping action, and that cracks past 13 ft. either side of the joint are not caused by any movement at the joint.

Additiomal study of Figs. 24 and 25 shows that the distribution of cracking less than 13 ft. in 1944 is bi-modal,/c having a mode between 5 and 6 ft. and one at 10 ft. This perhaps shows that once a slab cracks about 10 ft. from the joint, a new and smaller slab, lighter in weight and able to move more freely, is formed. This, in turn, begins to break up usually at a point 5 or 6 ft. from the joint.

Since the curves shown in Figs. 24 and 25 are bi-modal, a second distribution was plotted in which cracking is divided into cracks occurring in forward and rear slab areas. This distribution is contained in Figs. 26 and 27. Study of Figs. 26 and 27 shows considerable difference in the cracking of the rear and forward slabs and that in 1942 cracking less than 15 ft. ahead of the joint was more severe than cracking less than 13 ft. to the rear of the joint. However, by 1944 cracking in the rear slabs had developed considerably in proportion to cracking on the forward slabs. The 1944 curve of the forward slab distribution of cracks contains modes at 6 and 10 ft. as compared to 5 and 10 ft. on the rear slabs. However, cracking at 5 ft. in the rear slabs was more severe than cracking at 5 ft. in the forward slabs in the 1944 survey; also, that cracking at 10 ft. on the forward slabs was more severe than at 10 ft. on the rear slabs in 1944.

In 1942, at the time the pavement was five years old, there were 466 cracks on 277 slabs (1.68 cracks per full slab). This is divided into 218 cracks on 277 rear slabs (0.78 per half slab) and 248 cracks on 277 forward slabs (0.89) per half slab - up to 20 ft. to the rear of a joint. By 1943, six months after treatment of 50 slabs, there was a total of 496 cracks (or 30 new ones) on the 277 slabs. Of the 30 new cracks, four occurred on the rear slabs of mix "C" joints; two on forward slabs of mix "A" joints; three on forward slabs of mix "B" joints; one on a forward slab of a mix "C" joint; and the remainder on untreated slabs.

By 1944, two years after treatment, there was a total of 697 cracks in 277 slabs (2.4 per slab) or a total of 231 new cracks in two years. This can be divided imto 326 cracks on 277 rear slabs (1.18 per half slab) and 371 cracks on forward slabs (1.34 per half slab). The above data, stated differently, show that at the time the pavement was five years old, there were 233 cracks per mile, and that by the end of the next two years (1944) this had increased to 348 cracks per mile, which is

/c - The mode is that value of a distribution that occurs most frequently. A bi-modal distribution is one in which two sets of values occur as prominent peaks on a distribution curve.

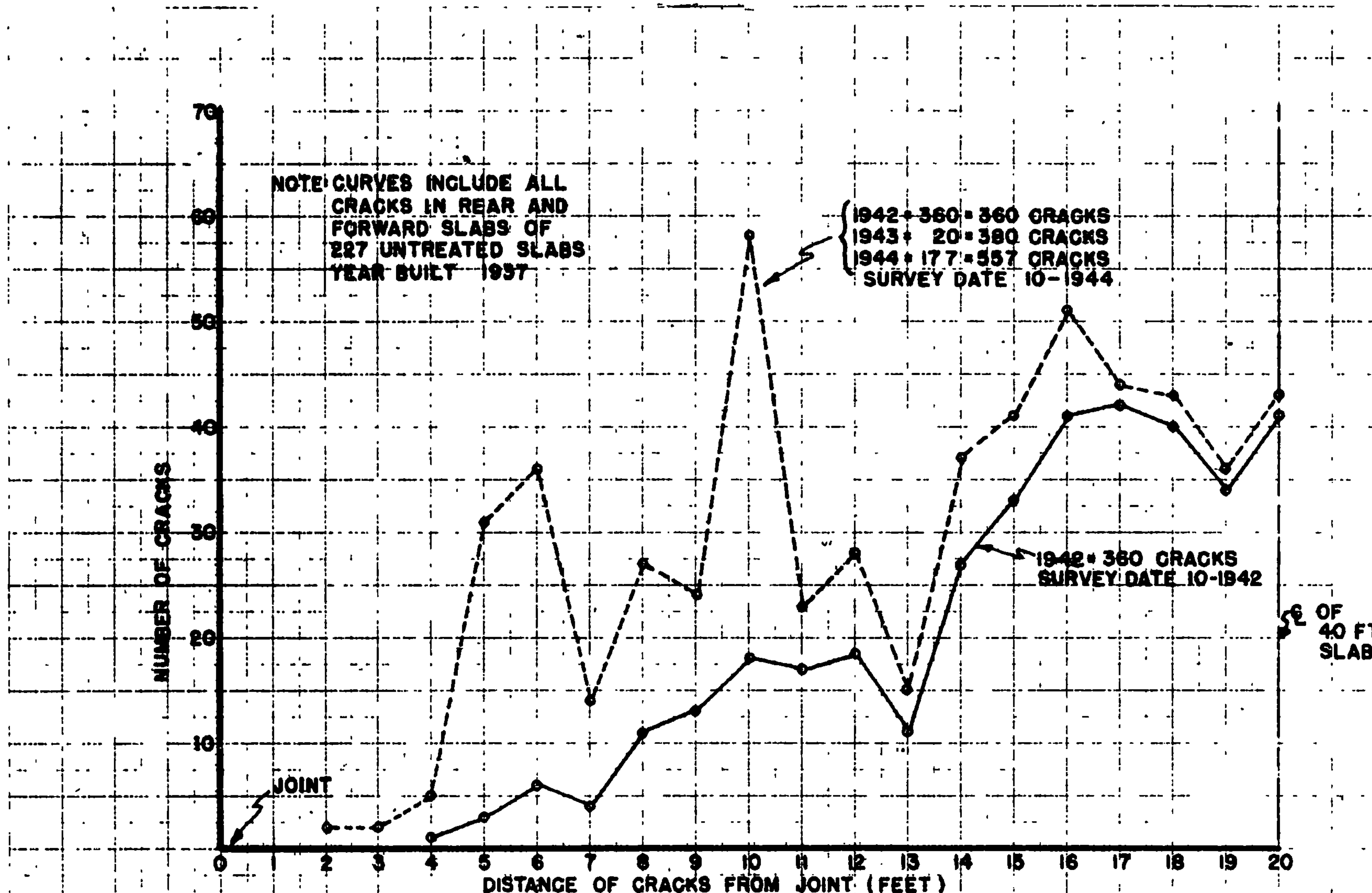


FIGURE 24. DISTRIBUTION OF CRACKS IN UNTREATED SLABS 1942-1944

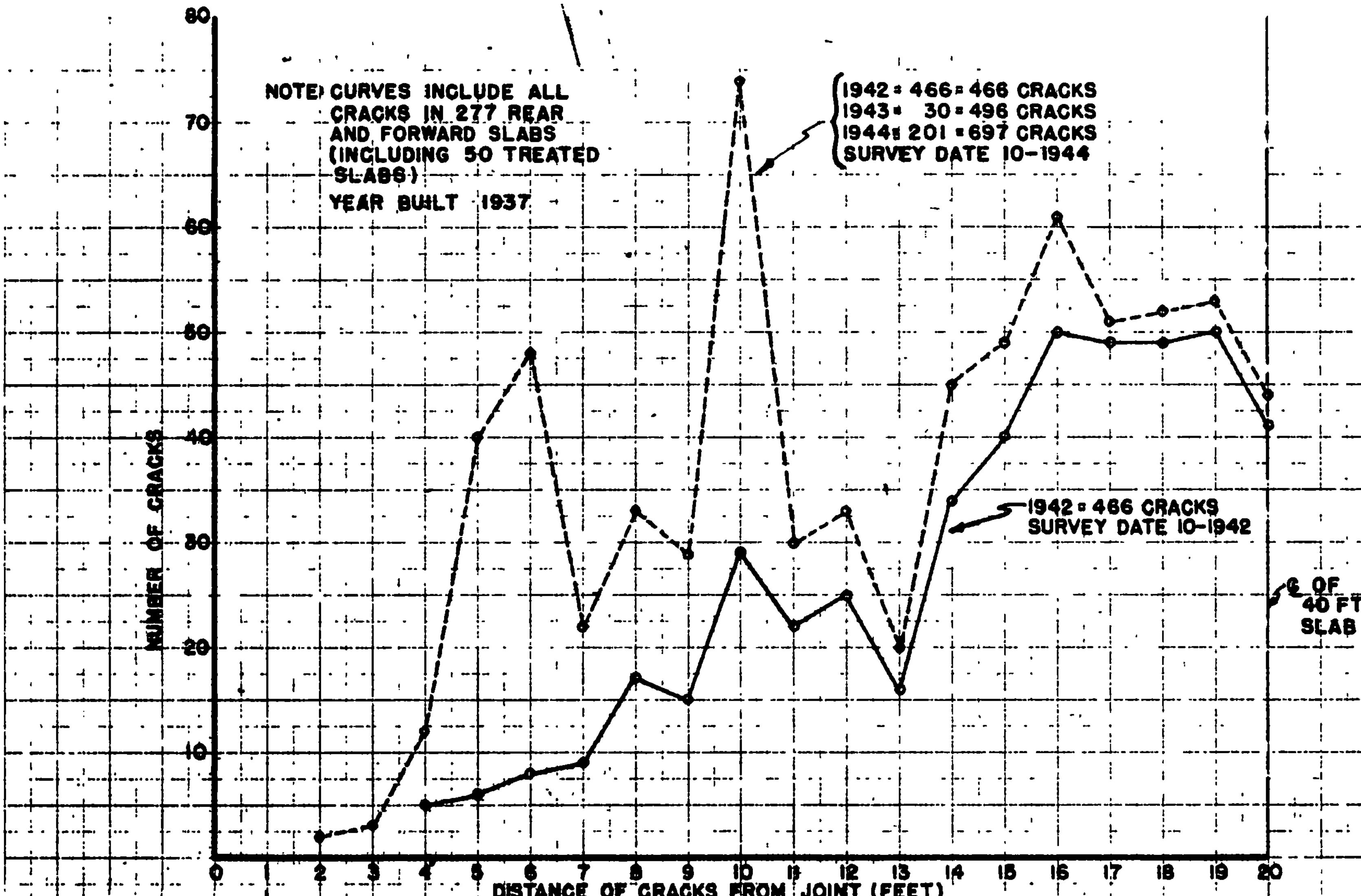


FIGURE 25. DISTRIBUTION OF CRACKS IN ALL SLABS 1942-1944

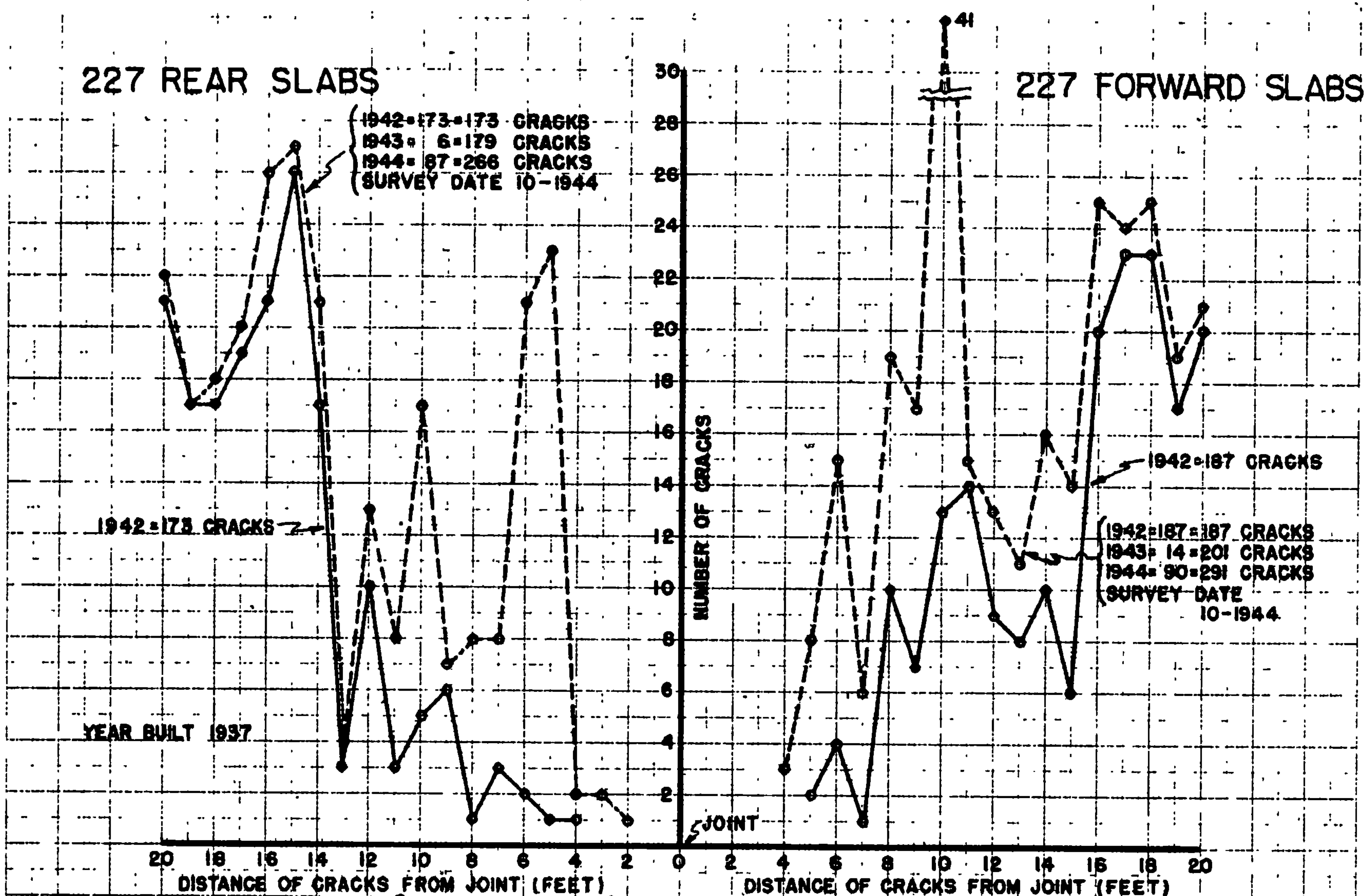


FIGURE 26. DISTRIBUTION OF CRACKS IN UNTREATED SLABS
1942–1944

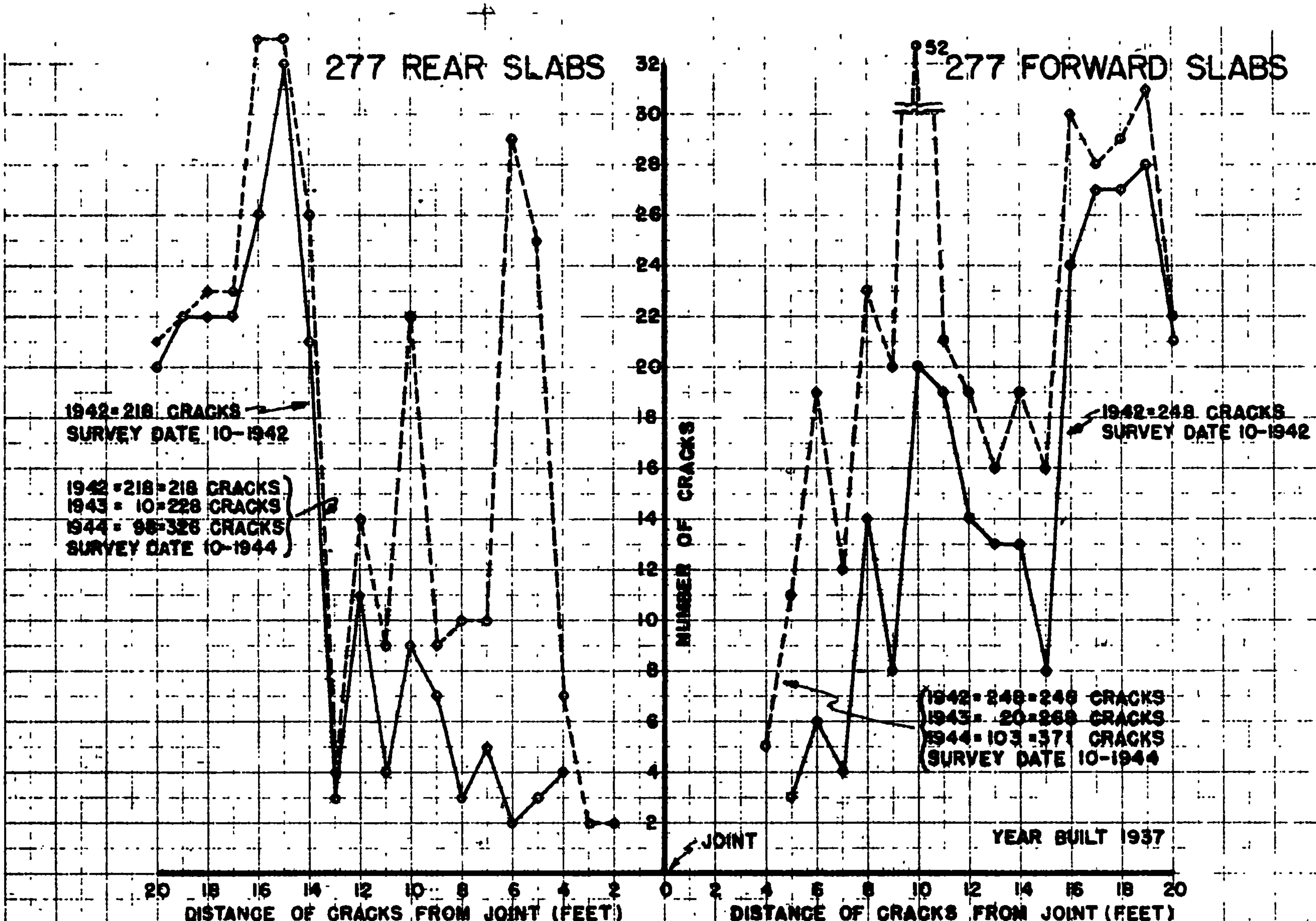


FIGURE 27. DISTRIBUTION OF CRACKS IN ALL SLABS 1942 – 1944

an increase of 49.3 percent during the two-year period. Further, the number of new cracks occurring in the forward slabs was nearly equal to the number of new cracks occurring in rear slabs (of the entire 277 treated and untreated sections). However, study of Fig. 27 shows a remarkable difference between the position of new cracks in forward and rear slabs with respect to the joints.

Figures 28, 29, and 30 are distribution curves of cracks less than 13 ft. from the joints of untreated slabs, treated slabs, and individual mixes for each of the three surveys. Further study of Figs. 28, 29 and 30 illustrate the effect of mud jacking on cracking. Assuming that all cracks less than 13 ft. either side of a joint are caused by pumping action, these data show that treatment by mud jacking has decreased considerably the expected number of cracks on both rear and forward slabs. Study of Fig. 28 shows that in 1942 there were 35 cracks in 227 untreated rear slabs and that by 1944 there were 114 cracks which is an increase of 226 per cent during the two-year period. This can be compared to the data contained in Fig. 29 which shows that in 1942 there were 17 cracks in 50 rear slabs, that were later treated, and that by 1944 there were 29 cracks, which is an increase of only 70.5 per cent. In other words, the expected cracking in untreated rear slabs should have been a 226 percent increase but treatment reduced this to a 70.5 percent increase. A similar study of forward slabs shows that the expected cracking was reduced from a 117 percent normal two-year increase to an increase of 51.5 percent for the treated slabs.

The distribution of new cracks, those occurring during the two-year period only, less than 13 ft. from the joints for each class of pavement is contained in the series of curves, Figs. 31 to 34. Study of Fig. 31 shows that the new cracking of untreated slabs during the two-year period was nearly equally divided between rear and forward slabs with rear slabs receiving 79 and forward slabs 80 new cracks. The curve shows that the distribution of cracks about the joint varies considerably between rear and forward slabs. Cracking was excessive between five and six ft. to the rear of the joints and excessive at 10 ft. ahead of the joint. These data, representing cracking of untreated slabs, can be compared with the data contained in Fig. 33 which is a distribution of cracking in the 50 treated slabs. This curve shows that cracking of the forward slabs remained fairly constant - that is, the distribution curve does not contain a mode. However, the distribution of cracking on the rear slabs shows that cracking was quite severe at six feet from the joint, which is the modal point of the distribution. The difference in cracking of rear and forward slabs of treated joints is significant because 47 of the 50 joints treated were treated on the forward slabs only. This can be attributed to the fact that, at the time of treatment, the rear slabs of treated joints had not settled and did not require treatment as did the forward slabs.

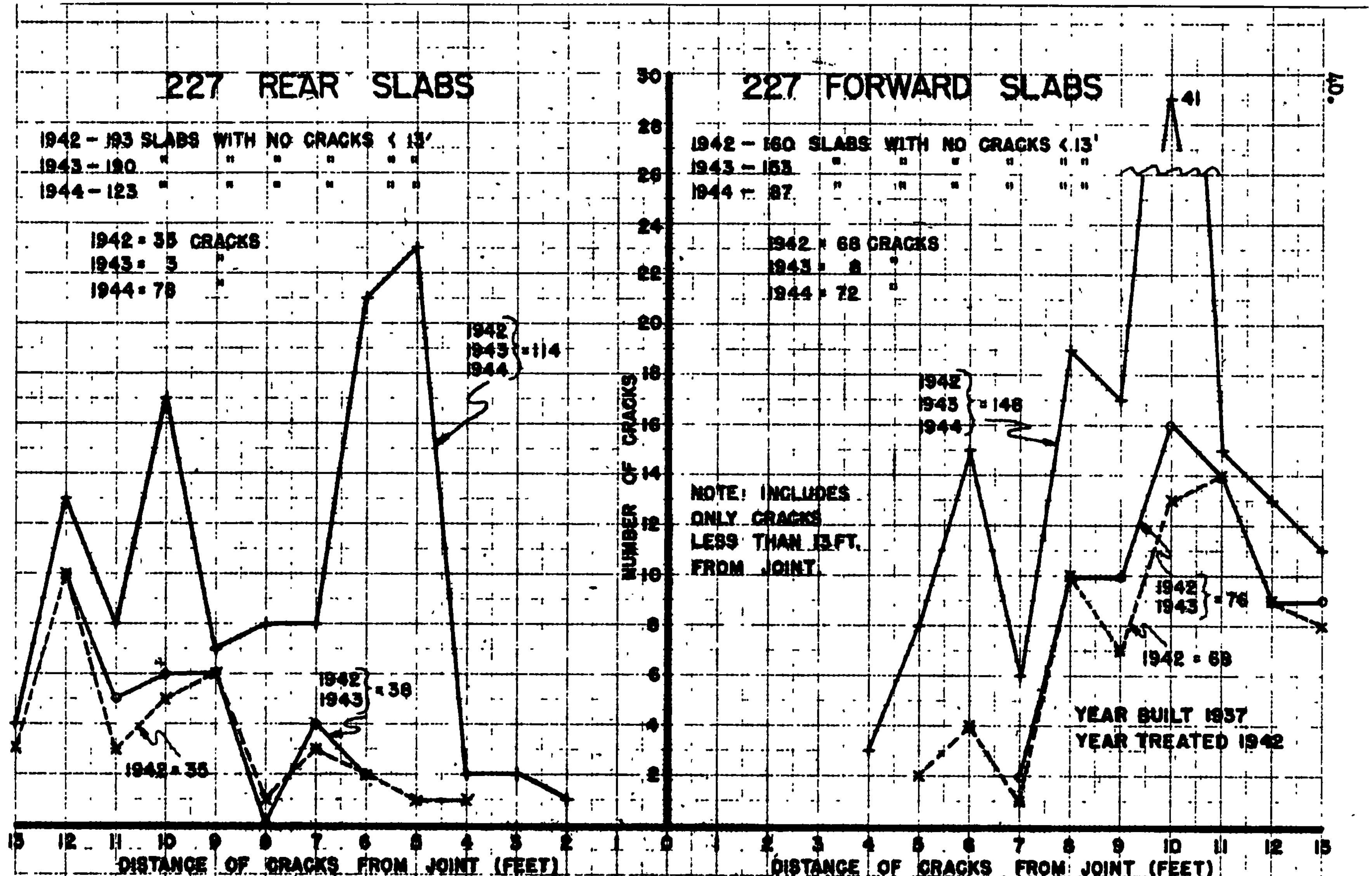


FIGURE 28. DISTRIBUTION OF CRACKS LESS THAN 13 FEET FROM THE END JOINTS OF THE 227 UNTREATED SLABS

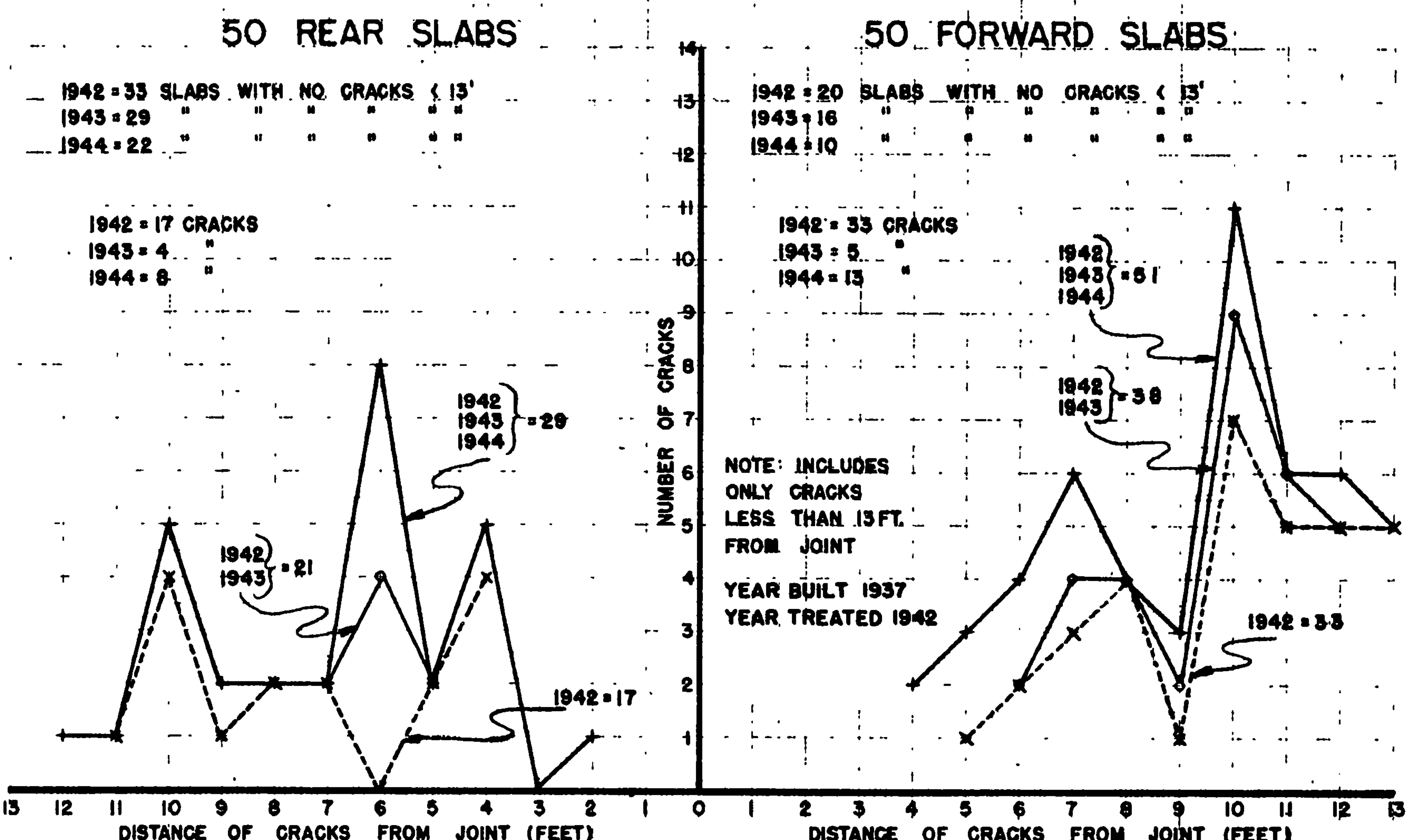


FIGURE 29. DISTRIBUTION OF CRACKS LESS THAN 13 FEET FROM EITHER END OF 50 TREATED SLABS

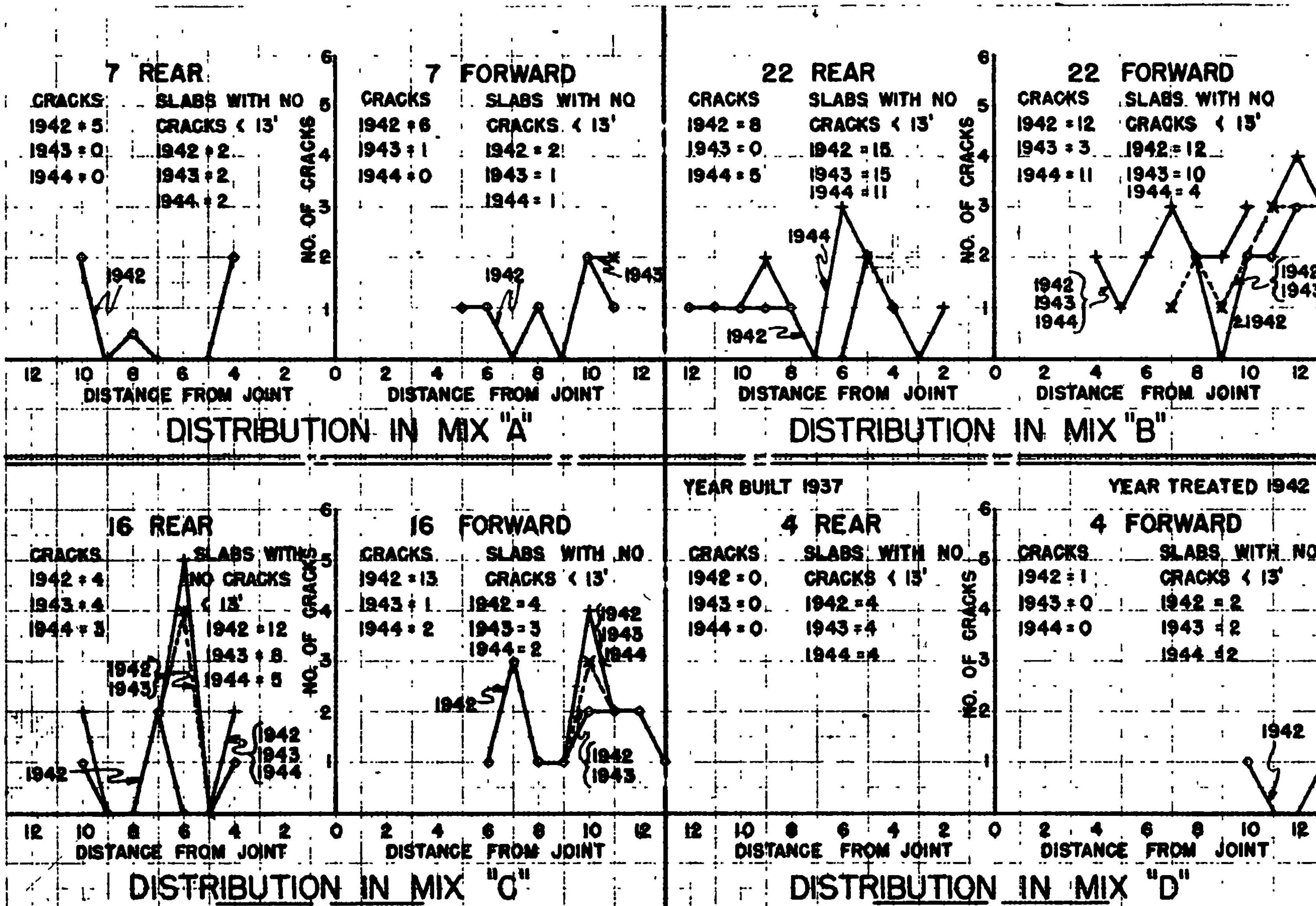


FIGURE 30. DISTRIBUTION OF CRACKS LESS THAN 13 FEET FROM THE JOINT BY MIXES

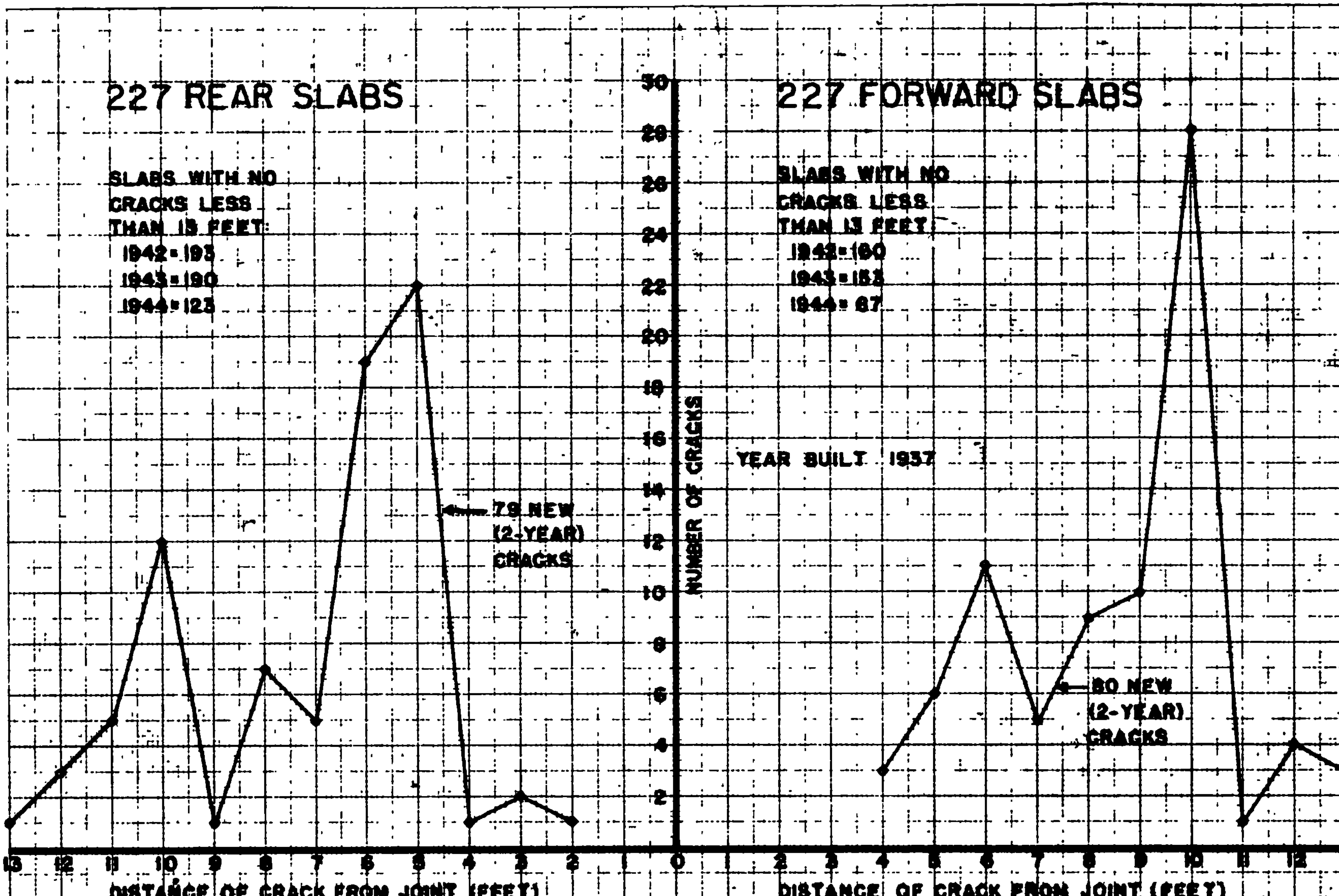


FIGURE 37 DISTRIBUTION OF NEW CRACKS OCCURRING IN TWO YEARS - 227 UNTREATED SLABS

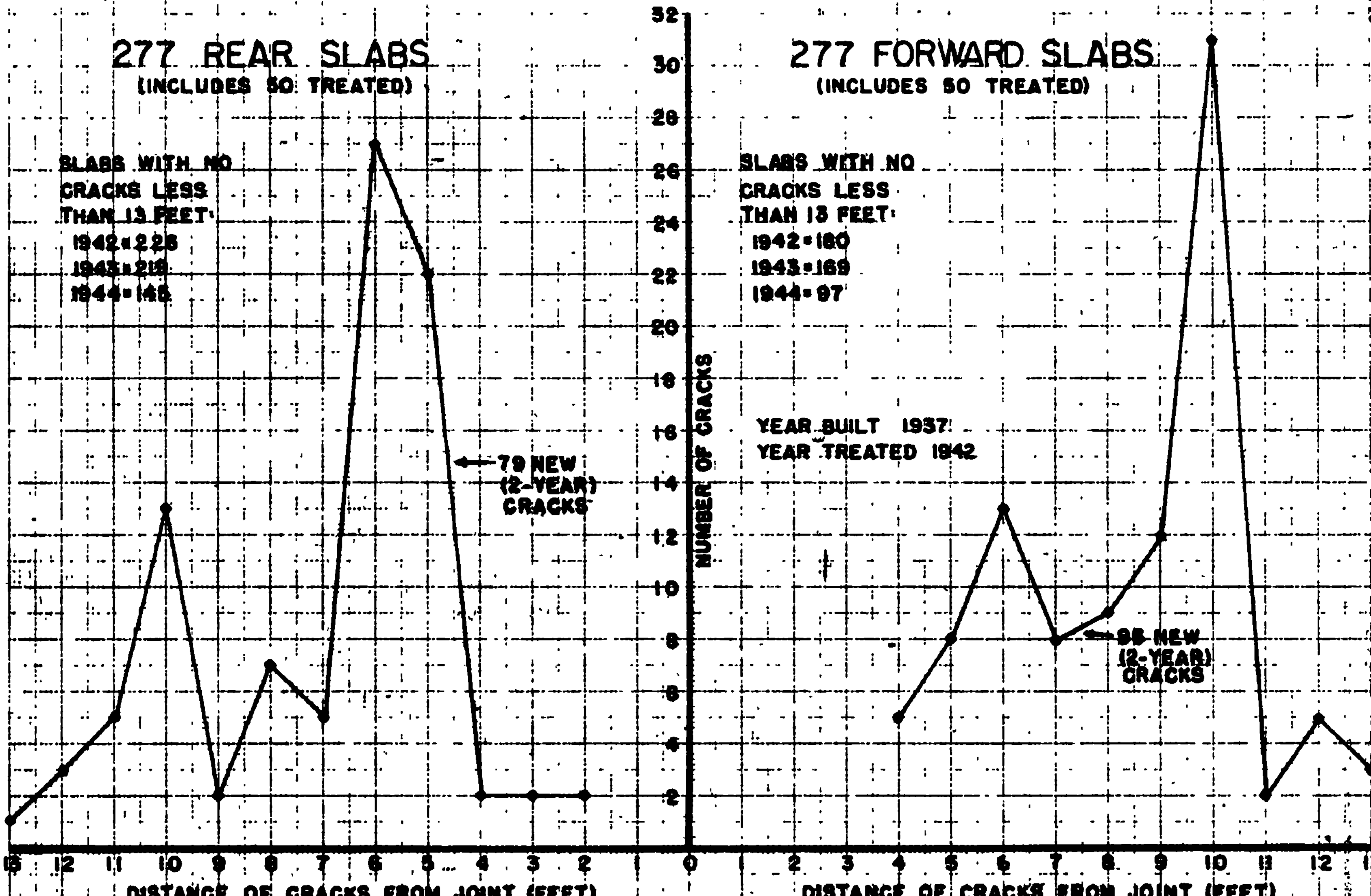


FIGURE 32. DISTRIBUTION OF NEW CRACKS OCCURRING IN TWO YEARS - TWO MILE SECTION

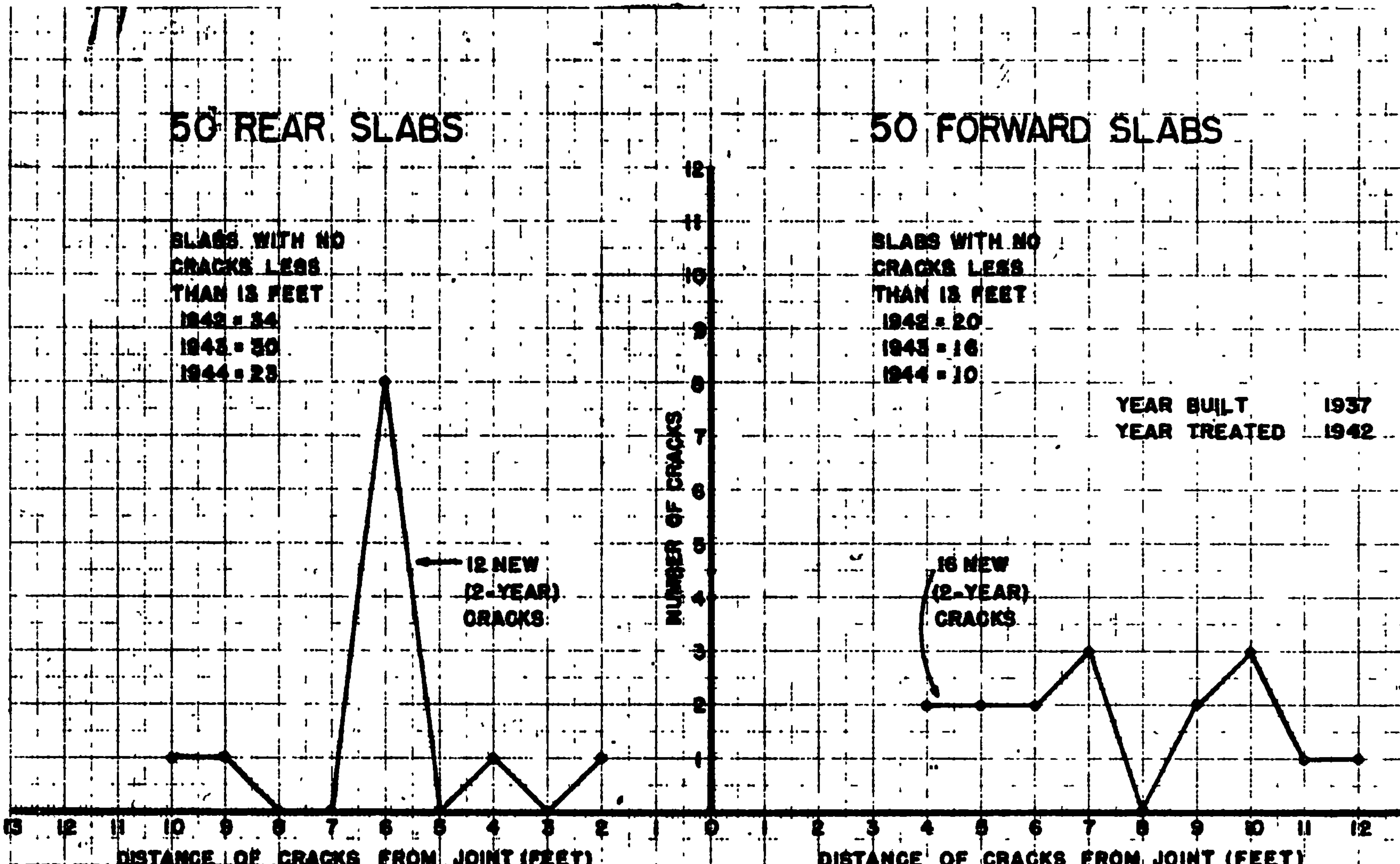


FIGURE 23. DISTRIBUTION OF NEW CRACKS OCCURRING IN TWO YEARS – 50 TREATED SLABS

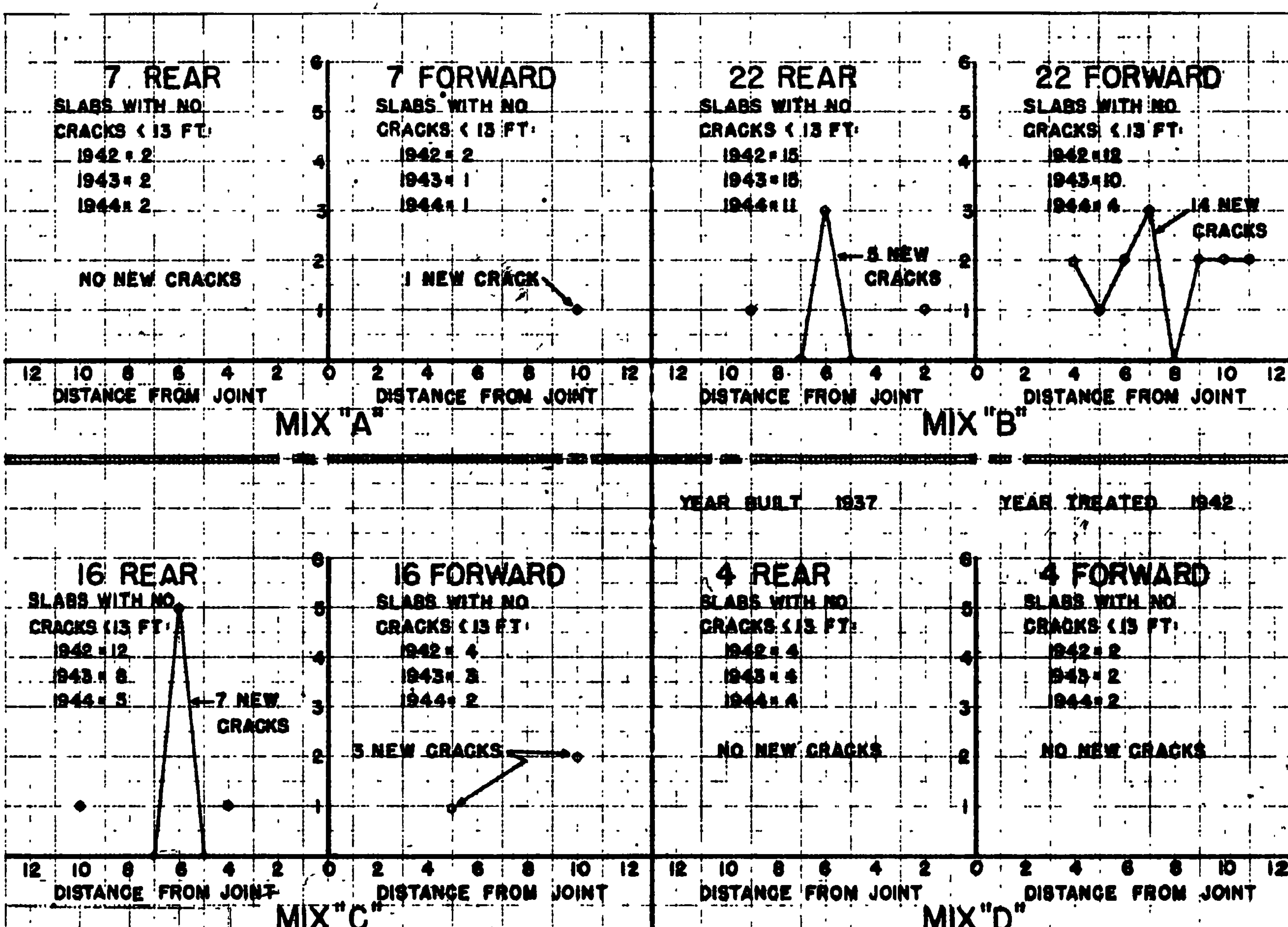


FIGURE 34. DISTRIBUTION OF NEW CRACKS (TWO YEARS) BY MIXES

REMOVAL OF FAILED SLABS

On November 27, 1944, portions of three slabs that had failed and settled considerably were removed and replaced with sand and given a limestone surface. Each of the failed slabs had been patched; some containing two patches. Two of the slabs had been treated by mud jacking (mix B) and one was of the untreated section. The two treated slabs that failed were located in a cut section where the grade was such that drainage was exceptionally poor. Both were rated P-3 (severely pumping) at the time of treatment. A description of each joint and the condition during each performance survey are contained in the Appendix (Record of mud-jacked joints that were patched). Figures 35 and 36 were taken at the time of removal of these slabs. Figure 37(A) is a photo of a sample of mud-jack mix taken from beneath the slab at Sta. 694+86. The mix under this slab was laminated and strings of bituminous material were found in the mix. This is perhaps because of the fact that mixing and treating were done during cold weather and that the hot bituminous material chilled and became stringy when it came in contact with the cold mixing water. Figure 38 contains a series of sketches showing the condition of the subgrade at each location at the time the slabs were removed.

Examination of the mix under the slab at Sta. 695+26 showed the mix to be somewhat disintegrated (cracked) but that it was fairly uniformly mixed and not plastic. The area in which the mix was found seemed to be somewhat drier than the rest of the subgrade. A band of mud about two feet wide was found under the joints and cracks. It appeared that the broken slab sections were rocking on or being supported by the drier area in which mud-jack mix was found. This was true in the other treated section which was removed. A similar muddy condition was found under the untreated slab. The subgrade was dry under the inner portions of the slab and extremely wet under the cracks and at the joint and edge.

The drains in all three cases had clogged from the bottom of the slab to the tile. The stone particles were completely coated with silt and clay pumped from beneath the subgrade. Figure 39 shows the condition of the stone in some of the drains.

A sample of the mix under the slab at Sta. 695+26 was compared to a sample that had been exposed for two years in a ditch on the Deep River hill and found to be similar in all details. Both were hard, non-plastic, uniformly mixed, somewhat porous, and relatively dry.

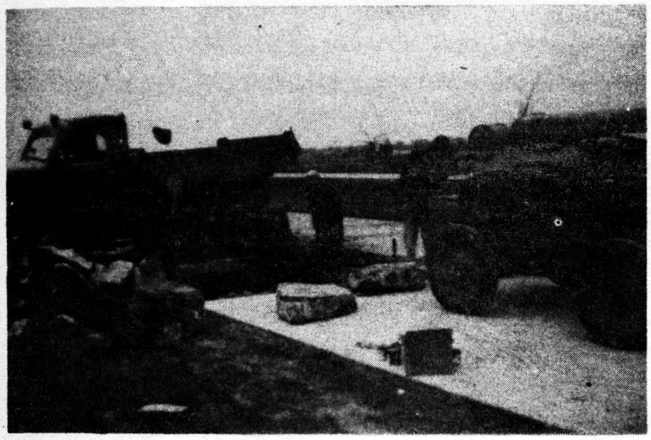


FIGURE 35. REMOVING A SLAB THAT HAD FAILED DUE TO PUMPING.

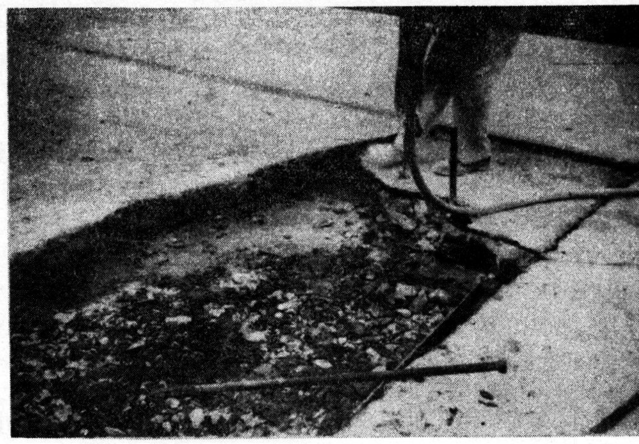


FIGURE 36. AN ILLUSTRATION OF THE ZONE OF MUD FOLLOWING THE CRACK.

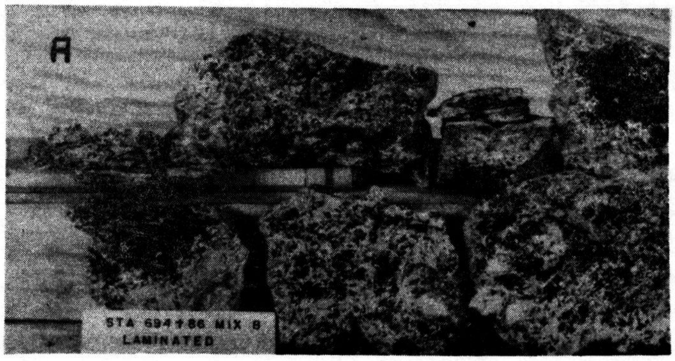
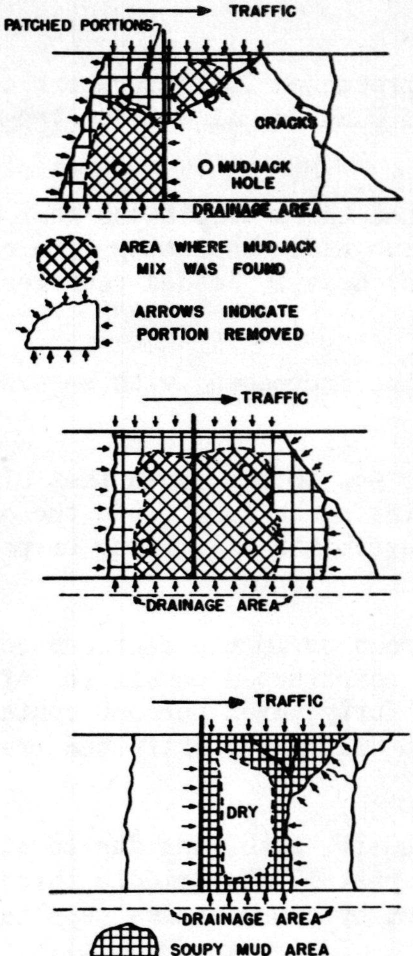


FIGURE 37. PHOTOGRAPHS OF THREE SAMPLES OF MUD-JACK MIX. THE SAMPLES IN PICTURES A AND B WERE TAKEN FROM BENEATH THE PAVEMENT OF TWO TREATED SLABS, AFTER TWO YEARS OF SERVICE. THE SAMPLE IN PICTURE C WAS EXPOSED TO WEATHERING FOR TWO YEARS IN THE DITCH LINE. COMPARE THE TEXTURE OF SAMPLE A WITH THAT OF SAMPLES B AND C. STATION 694+86 WAS TREATED WITH MIX B DURING COLD WEATHER AND THE CHILLING OF THE BITUMINOUS MATERIAL RESULTED IN A NON-UNIFORM MIX. NOTE THE LAMINATIONS IN SOME OF THE SAMPLES IN PICTURE A.



MIX "B" STATION 694+86

The mix under the slab varied in thickness and texture. The thickness varied from 0 to 2"; the thickest being farthest from the holes and the maximum under the holes. No trace of the mix was found in the inner rear portion of the section. The mix was laminated. Mud was found under the parts adjacent to the mix. The stone in the drain had clogged to the tile.

MIX "B" STATION 695+26

The mix under the rear slab had disintegrated and only a slight trace was found. The mix under the forward slab was also disintegrated but there was more present. The thickness varied from 0 to ½". The mix had cored to 3" under the holes. An area of soupy mud was found around the crack areas. The drain (stone) was clogged to the tile.

UNTREATED STATION 662+46

This slab was not treated. A zone of soupy mud was found directly under the cracks and the joint. The remainder of the subgrade-pavement contact area was fairly dry. The drain at this joint had also clogged to the tile.

FIGURE 38. SKETCH OF SLAB AREAS FOR REPLACEMENT.

FIGURE 39. COMPARISON BETWEEN CLEAN STONE AND STONE COATED WITH MUD SLURRY FROM A PUMPING JOINT. THIS STONE WAS REMOVED FROM A DRAIN THAT HAD CLOGGED.

SUMMARY OF RESULTS

The following statements summarize the data presented in this report concerning the performance of U.S. No. 30 from Valparaiso to U.S. No. 41 and the two-mile section under observation:

1. This work confirms previous studies in that three conditions must be satisfied in order to have pumping. These are: (a) A subgrade consisting of a relatively plastic and impervious clay or silty-clay soil; (b) Heavily loaded vehicles; and (c) free water.

2. Once pumping starts the rate of progression increases, with severe pavement cracking and settlement resulting.

3. Each performance survey conducted on U.S. No. 30 has shown that all of the experimental subgrade treatments constructed on the south lane (with the exception of the water-saturated section) have been successful in minimizing or preventing pumping.

4. Pumping does not occur in natural sand areas or in the sections constructed on 6 in. of sand. Forty-two percent of the slabs constructed on six in. of sand contained no cracks after seven years of service and forty-seven percent contained only one crack. All slabs on the two-mile section on silty clay drift had cracked by the end of seven years.

5. Cracking within 13 ft. of a joint, in a 40-ft. slab, was due to slab movement initiated by pumping action and that cracks past 13 ft. (middle third of slab) were not caused by any movement at the joint but by other causes (applies to untreated slabs only).

6. The destruction of 40-ft. slabs by cracking initiated by pumping at joints on U.S. No. 30 appears to follow a definite pattern: the forward slabs pump and appear to break off first, usually 10 ft. from the joint and then six ft. from the joint; the rear slabs follow by breaking off usually at five feet from the joint which is followed by a break at 10 ft. (applies to untreated slabs only).

7. Between the fifth and seventh year of this pavement's life cracking less than 13 ft. from the joint increased 154 percent while cracking past 13 ft. (or in the middle third) increased only 10 percent (applies to untreated slabs only). (See Fig. 28.)

8. During the two-year period (fifth and seventh year) cracking less than 13 ft. to the rear of untreated joints increased 226 percent, and cracking on the forward slabs increased 117 percent. During the same period, cracking within 13 ft. to the rear of treated joints increased only 70.5 percent and on forward slabs only 51 percent.

9. Treatment by mud jacking reduced the expected increase in two-year cracks (fifth to seventh year) within 13 ft. from joints from an increase of 154 per cent to an increase of 58 percent. (See Fig. 28)

10. Each mud-jack mix was effective in reducing cracking. Only one new crack occurred in 7 slabs treated with mix "A"; none occurred on 4 slabs treated with mix "D"; and 10 new cracks occurred on 16 slabs treated with mix "C".

11. During the two years following treatment, mud jacking has been successful in reducing the average settlement of slabs at pumping joints. The average settlement of the outer edge of treated slabs two years after treatment was found to be 0.093 inches as compared to 0.194 inches for untreated slabs.

12. During the two years following treatment 48 percent of the treated slabs had not settled as compared to 26 percent not settling for untreated slabs. Sixty-eight percent of the treated joints had settled less than one-eighth inch as compared to 53 percent of the untreated slabs.

13. Mixes "A" and "D" were the most effective in reducing both cracking and settlement. Of mixes "B" and "C", slabs treated with mix "B" settled less than slabs treated with mix "C".

14. It was found that the amount of material pumped from beneath a slab due to pumping action is appreciable, as indicated by an average of 8.7 cu. ft. of material per joint which was pumped beneath the slabs.

15. When bituminous material is used in mud jacking, the operation should be done during periods of warm weather to avoid chilling of the bituminous material in the mix.

16. Observations extraneous to the data contained indicate that periodic maintenance of mud-jacked joints by pouring of the cracks and joints and by keeping the drilled holes filled will prolong the effective life of treatment.

17. These data show that since cracking, settling, and pumping on rear slabs increases rapidly following the failure of forward slabs, they should also be treated by mud jacking which, if done during the initial stages, would consist of a void-filling operation while that of the forward would be a raising operation.

18. The limited data contained in this report regarding mud-jacking procedures prevent recommendation for operational procedures; however, by way of observation the following are pertinent:

(a) The work should be under the care of an experienced operator or one competent to judge when a slab has been properly treated, since mistakes will perhaps prove detrimental rather than beneficial.

(b) Each slab should be handled as an individual case rather than following a standard procedure of operation.

(c) Traffic should be kept off of treated slabs for at least 24 hours following treatment.

(d) In areas where surface drainage is exceptionally poor, the slab should be given some form of supplemental drainage that should receive periodic maintenance.

19. The results of this survey show that mud jacking can prolong the life of a pavement being destroyed by pumping action, and the above results show a need for further research in which other materials should be used and in which various operational procedures are investigated.

REFERENCES

1. "Performance Survey on a Portion of U.S. 30, Four-Lane Divided Pavement in Lake and Porter Counties", Unpublished Report No. 4 on Concrete Performance Survey, Project C-36-35, by T. E. Shelburne, Research Engineer, Joint Highway Research Project, Purdue University.

2. Woods, K. B. and Shelburne, T. E. - "Pumping of Rigid Pavements in Indiana", Proc., 23rd Annual Meeting, Highway Research Board, 1943.

3. Wartime Road Problems No. 4, page 9, Highway Research Board, Washington, D. C. (See page 133 of this publication).

APPENDIX

RECORD OF MUD-JACKED JOINTS THAT WERE PATCHED (FIELD NOTES)

Mix A. One joint out of seven had been patched and was rated P-4 (pumping very severely) after two years' service.

Sta. 721+66

10/21/42 - During the mud-jacking operations at this joint the hose broke and operations ceased for the day, and were resumed on October 27.

10/27/42 - The RC-3 mix was very stiff due to cold weather and more water had to be added to make the mix workable. Total mix applied = 11¼ cu. ft.

5/ 6/43 - Performance Survey
Rated the joint as P-2 (moderately pumping). Pumping was observed along the edge of the pavement and along the joint. One new crack had developed in the forward slab. The French drain was plugged.

10/12/44 - Performance Survey
Rated the joint as P-4. Pumping very severe. Even though the joint had been patched with a bituminous patch, the joint was pumping and the slabs had settled an additional amount.

Mix B. Three joints out of 22 had been patched, two of which were rated P-4 and one rated OK after two years' service.

Sta. 717+26

10/28/42 - Treated by mud-jacking. The subgrade was dry and there was no soupy mud under the pavement. The pavement could not be raised by pumping material into two holes so it was necessary to use three holes along the joint. A gasket leak developed in the pressure chamber which was sufficient to prevent complete mudjacking. The mix applied on 10/28/42 was $4\frac{1}{2}$ cu. ft. (Road oil mix).

5/ 6/43 - Performance Survey
The joint was rated as P-2. No new cracks were found in either slab but two rear slab cracks were pumping P-1 and P-2. The mud-jack holes were full.

10/12/44 - Performance Survey
The joint was rated OK since pumping was not observed. One new crack was observed in the forward slab and one corner break in the rear slab.

Sta. 695+26

11/ 2/42 - Mud-jack operations. The treatment at this joint was somewhat complex. A small amount of mix was added to each hole to raise both slabs to grade. Each slab had settled and two holes were drilled in each slab. The forward slab required 12 cu. ft. to raise; the rear slab four cu. ft. or a total of 16 cu. ft. of Road Oil mix. The joint was then drained with French drain consisting of No. 8 stone. However, the drain had insufficient outlet as the pavement section was in a cut area.

5/ 6/43 - Performance Survey
This survey showed three of the mud-jack holes half full and one empty. The joint was rated P-1 and the first crack in the rear slab (nearest the joint) was rated P-2 and was spalled severely. One new crack (corner break on inner side of forward slab) had developed.

10/12/44 - Performance Survey
This survey showed the joint to be a complete failure. Even though the joint had been patched (area between cracks), the inner section of the slabs had settled 1-3/4 inches since patching. The joint was rated as P-4 because pumping was severe. Two new cracks had developed in the forward slab.

11/27/44 - Slab removed for replacement.

Sta. 694+86

11/ 2/44 - Mud-jack operations. This joint required using four mud-jack holes (two on the forward slab and two on the rear slab) to raise to grade. The forward slab required 12 cu.ft. and the rear slab four cu.ft. The pavement (rear slab) was cracked during mud-jacking operations (corner break). The pavement was in a cut area and the joint was drained using a French drain (#8) stone. The outlet was insufficient for successful drainage.

5/ 6/43 - Performance Survey
The joint was rated as P-1. One new crack had developed (corner crack on inner part of slab) on the forward slab. The crack nearest the joint on the rear slab was rated P-1. One mud-jack hole was empty.

10/12/44 - Performance Survey
The joint was rated as P-4 because the inner part of both slabs had been patched between the corner cracks and pumping was severe. The rear slab section had settled one inch (to the top of the patch). The actual settlement was over four inches.

THE USE OF BITUMINOUS MATERIALS AS A CORRECTIVE MEASURE FOR PUMPING CONCRETE PAVEMENTS

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and Harry E. Marshall, Geologist,
Bureau of Tests,
Ohio Department of Highways

SYNOPSIS

The pumping of concrete pavement slabs on pavements in Ohio, which has developed since 1940, has followed the increase in volume of heavy truck traffic that has resulted from the concentration of war industries. A survey indicates that pumping has occurred mostly on soils of the A-4, A-6, or A-7 groups but is not confined exclusively to soils of these types. A study of the moisture contents of the subgrades at various depths indicates a maximum immediately beneath the pavements and a decrease with depth indicating that surface water is the chief source of the subgrade moisture that causes pumping. The use of transfer devices at joints and of granular subgrades have been found most effective in the prevention of pumping. The use of French drains in the shoulder was not effective in stopping pumping.

After experimenting with various soil-bituminous, portland cement mixtures and several grades of semisolid asphalts, it was found that an oil asphalt filler having a penetration of 30 to 45 at 77°F. was most satisfactory for filling the voids under pumping concrete slabs. This material is forced under the pavement by means of the hand spray equipment of a standard bituminous distributor through holes drilled in the pavement with a standard jackhammer and drill. The bituminous material forms a tight seal beneath the pavement and prevents the entrance of surface water and its stability is not affected by moisture from the subgrade. Although the costs of the asphalt is somewhat higher than any of the various soil-mixtures, a portion of this cost differential is equalized in the labor saved on the assembling and mixing of the various materials used in slurries.

Prior to 1940 pumping of concrete pavements was not prevalent in Ohio. However, the considerable concentration of war industries in the State with its attendant increase in truck loads and in volume of truck traffic has resulted in a very rapid increase in both the distribution and the rate of pumping concrete pavements.

In this area pumping is particularly prevalent over silty-clay and clay soils, Public Roads Administration Classes A-6, A-7 and plastic A-4. However, it is not confined exclusively to these types. On a heavily traveled section of U.S. Route 52, East of Portsmouth, constructed recently on a subgrade made up in part of about equal amounts of silts and clays, considerable pumping has developed. This project is described in detail in a paper by Mr. H. L. Krauser/1 and it is sufficient to point out

/1 - See page 67 of this publication.

in this discussion that 37 per cent of the joints were pumping where the subgrade consisted of sandy silt and silt, Ohio soil classifications Nos. 8 and 9, while 57 per cent of the joints were pumping over the silty clay and clay subgrade, Ohio classifications Nos. 11, 15 and 16. The Ohio soil classifications are shown in Table 1. The average test constants of the soil on this project are shown in Table 2.

It is generally agreed that most of the water contributing to pumping is surface water. A number of observations made in Ohio substantiate this point. On a project constructed about twelve years ago which had shown no signs of distress until wartime restrictions on a refinery in the southern part of the State resulted in a tremendous increase in truck shipments, pumping occurred generally throughout the section except for areas where the joints were tightly sealed, preventing the entrance of surface water. Further evidence that the water is derived principally from the surface is afforded by the fact that on most of the projects observed there is practically no difference in the amount of pumping on fill and in cut sections. It has also been noted on several projects that less pumping occurs in areas where there is a paved gutter at the edge of the pavement which carries off the surface water before it has an opportunity to reach the subgrade.

In connection with our study of pumping pavements during the past two years, a considerable number of samples have been taken of subgrade through holes drilled in the pavement, in an effort to learn something of the moisture condition of the subgrade soil. A sheet of a typical soil survey is shown in Figure 1. The locations of the joints, cracks and pavement failures together with the points where the pavement jacking was done and the samples taken are shown in the lower part of the Figure, while in the upper portion the soil type and moisture content of the subgrade are shown. It is of interest to note that the moisture content of the subgrade decreases with the depth through the range measured by these samples; averaging 26.4 per cent in the 0.6 of a foot immediately beneath the slab, 24.4 per cent in the next 0.9 of a foot and 21.4 per cent in the bottom 0.9 of a foot.

From the summary of test results in Figure 1, it will be noted that the subgrade in this section consists almost entirely of clay for which the average lower liquid limit is 46.8 and the average plasticity index is 25.2.

Table 3 shows average test results and moisture contents for samples of the subgrade soil on several projects which were treated during the unusually dry summer of 1944. This Table shows a considerable variation in moisture contents of the subgrade soil in the various groups. However, in about two-thirds of the cases in which samples were obtained at different depths beneath the pavement, the moisture content of the subgrade soil was highest immediately beneath the pavement and decreased with the depth through the range sampled.

PREVENTATIVE MEASURES

The treatment of pumping may be divided into two parts (1) measures which tend to check pumping on existing roads and (2) treatments during construction which tend to minimize or eliminate entirely the conditions which are conducive to pumping.

The use of steel reinforcing, load transfer devices, and the spacing and type of joints all have an important bearing on the susceptibility of a pavement to pumping. Pumping has been particularly severe on pavements constructed without load transfer

LEGEND AND CLASSIFICATION FOR SOIL TYPE IDENTIFICATION ON SOIL PROFILES

SOIL CLASS NUMBER	SYMBOL	DESCRIPTION	PUBLIC ROADS ADMINISTRATION GROUPS	COARSE AGGREGATE RETAINED ON #10 SIEVE	COARSE SAND #10 TO #60	FINE SAND #60 – #200	SILT #200–.005 mm	CLAY BELOW .005 mm	LIQUID LIMIT	PLASTICITY INDEX	FIELD MOISTURE EQUIVALENT	SHRINKAGE LIMIT	MAXIMUM DRY WEIGHT
1		GRAVEL	A-3	50-100	0-40	0-30	0 —	10		NP-10			
2		GRAVEL, SAND & SILT	A-2	30-60	15-30	15-20	15 —	40	15-35	NP-15	10-30	10-25	120-135
3		GRAVEL & SAND	A-1 A-2	30-70	15-40	15-30	0 —	20	15-35	NP-10	10-30	10-25	120-130
4		SAND	A-3	0-30	50 —	100	0 —	35		NP-5			100-115
5		SANDY SILT WITH COARSE MATERIAL	A-4 WITH C M	20-30	15-35	5-20	20 —	50	15-35	NP-15			
6		CLAY WITH COARSE MATERIAL	A-7 WITH C M	20-30	15-35	5-20	20 —	50	35-50	15-30			
7		BERM MATERIAL		CLASSIFIED BY VISUAL INSPECTION									
8		SILT	A-4	0-5	0-10	0-30	50-95	5-35	15-30	NP-12			105-115
9		SANDY SILT	A-4	0-30	10-40	10-40	20-50	5-30	15-30	NP-10	10-25		110-120
10		TOP SOIL	A-4	LOAMY MATERIAL CONTAINING DECAYED VEGETABLE MATTER AND HUMUS CLASSIFIED BY VISUAL INSPECTION									
11		SANDY SILT & CLAY	A-4	0-10	0 —	35	30-65	15-35	20-35	10-15	10-30		105-115
12		ELASTIC SILT & CLAY WITH ORGANIC MATERIAL	A-5						35+	P.I. LESS THAN $\frac{L.L.-14.0}{1.6}$			
13		ELASTIC SILT & CLAY WITH MICA	A-5						35+	P.I. LESS THAN $\frac{L.L.-14.0}{1.6}$			
14													
15		CLAY & SILT	A-7	0-10	0-25	0-15	50 —	100	35-40	15-25	25-35		100-110
16		CLAY	A-7	0-10	0-25	0-15	50 —	100	40+	20+	35+		90-105
17		CLAY	A-6						35+	P.I. GREATER THAN $\frac{L.L.-14.0}{1.2}$			
18		CINDERS		CLASSIFIED BY VISUAL INSPECTION									
19		ROCK-SOIL MIXTURE		30-90 % LARGE ROCK — CLASSIFIED BY VISUAL INSPECTION									
20		ORGANIC MATERIAL: PEAT, COAL OR COAL BLOSSOM	A-8	0 —		30			60+				
21		SOFT SHALE		CLASSIFIED BY VISUAL INSPECTION									
22		LIMESTONE		CLASSIFIED BY VISUAL INSPECTION									
23		SANDSTONE		CLASSIFIED BY VISUAL INSPECTION									
24		SHALE		CLASSIFIED BY VISUAL INSPECTION									

LIQUID LIMIT:– The moisture content, expressed as a percentage by weight of the oven-dried soil, at which the soil will just begin to flow when jarred slightly.

PLASTIC LIMIT:– The lowest moisture content, expressed as a percentage by weight of the oven-dried soil, at which the soil can be rolled into threads ⅛ inch in diameter without breaking into pieces.

PLASTICITY INDEX:– The difference between the liquid limit and the plastic limit.

FIELD MOISTURE EQUIVALENT:– The minimum moisture content, expressed as a percentage by weight of the oven-dried soil at which a drop of water placed on the smooth surface of the soil will not be immediately absorbed but will spread out over the surface and give it a shiny appearance.

SHRINKAGE LIMIT:– The moisture content, expressed as a percentage by weight of the oven-dried soil at which a reduction in moisture content will not cause a decrease in volume of the soil mass but at which an increase in moisture content will cause an increase in the volume of the soil mass.

BUREAU OF TESTS
OHIO DEPARTMENT OF HIGHWAYS

DATE 4-20-44

SOIL CLASSIFICATION CHART

PLAN PREPARATION MANUAL

APPROVED [signature] ACTING CHIEF ENGR.

No 1

TABLE 1

TABLE 2

SUMMARY OF TESTS OF SUBGRADE SAMPLES FROM SECTION OF U.S. 52 EAST OF PORTSMOUTH, OHIO

SHTL Class	No. of Samples	Mechanical Analysis: Agg. %	C. Sand: %	F. Sand: %	Silt: %	Clay: %	Physical Characteristics: Liquid Limit	Plasticity Index	Total Number Joints (Spacing: 20')	Per Cent of Joints Pumping
8	6	1.8	4.8	10.4	52.8	30.2	28.6	8.4	154	33.1
9	7	7.1	4.7	21.8	42.4	24.0	24.7	6.2	220	39.0
Ave. 8 & 9	13	4.6	4.7	16.1	47.8	26.8	26.5	7.2	374	36.6
11	6	1.4	3.8	8.2	51.6	35.0	30.2	10.7	164	43.0
15 & 16	8	0.1	1.1	2.3	40.1	55.4	39.8	16.1	212	68.0

devices of any kind. It has been noted that in pavements which are practically continuously under compression the severity of pumping is very much reduced.

Perhaps the most generally accepted means of prolonging the life of the pavement is the improvement of the subgrade. During the past several years subbase courses consisting of predominantly granular materials have been provided under many of our new pavements. The thickness of the material used varies for different subgrade soils and traffic conditions from 6 to 24 in. Most of the material used to date has met one of the grading requirements given in Table 4. In general, very little distress has been observed to date in pavements constructed over this type of subgrade. However, a few instances of pumping have been noted, and it has been rather frequently observed that material furnished under these requirements may have very low permeability. To assure more positive drainage in this subbase the grading requirements have been changed to those given in Table 5.

CORRECTIVE MEASURES

The Maintenance Bureau of the Ohio Department of Highways has of necessity in the past few years done a considerable amount of corrective work in an attempt to minimize the damage done by pumping. Early efforts consisted of attempting to drain away the free water by stone drains at the edge of the pavement either parallel to the slab or, as was more frequently the case, by French drains through the shoulder, Figure 2. The voids beneath the slabs were then filled using the mixture of soil and cement which had been previously found successful in raising depressed slabs. It was soon found from observation of the drains, particularly the open French drains extending through the shoulder, that these drains in themselves were not sufficient to remove the water and to stop pumping which had already started. During the past several years various combinations of soil, cement and other materials have been used for mudjacking. During the fall of 1942 the following mixtures were tried on different sections of U.S. Route 20 near Oberlin:

Mix 1

Asphalt Cement
50-60 Penetration

Mix 2

MC-1	1.0
Portland Cement	1.0
Gypsum Plaster	1.0
Soil	5.5

Mix 3

MC-1	1.0
Portland Cement	1.0
Limestone Dust	1.0
Soil	5.5

Mix 4

Vinsol Resin	1.0
Portland Cement	1.0
Gypsum Plaster	1.0
Soil	5.5

Mix 5

Vinsol Resin	1.0
Portland Cement	1.0
Limestone Dust	1.0
Soil	5.5

It was originally planned to get this treatment in during the fall of 1942, however, freezing weather made it necessary to postpone most of the work until the following spring, by which time the pavement had become so badly cracked and broken that comparison of the effectiveness of the various mixes is very difficult.

During the past summer voids beneath a considerable number of pumping joints have been filled by the use of slurries composed of 1 part cement, 1 part liquid asphalt, 1 part limestone dust and 5 parts of soil, by volume, or in approximately the proportions reported used by the Illinois State Highway Department.[2] This mix is reported to be the most satisfactory of any of the slurries tried.

Most of the soil mixes which have been tried in this State to prevent pumping have been only temporarily satisfactory. Where slurries have been used it has usually been necessary to return one or two years after the initial filling and refill the voids. During the removal of some badly cracked slabs on a project several years ago a sample was tken of old mudjack material. Of particular interest among the test results obtained on this sample is the very high moisture content of 50 per cent. The soil was a very poor subgrade material containing a high percentage of silt and having a high liquid limit and low plasticity index and should not be considered as typical of all mudjack material.

In order to overcome some of the objections to the usual mudjack materials, the Ohio Department of Highways in 1941 began experimenting with various mixes in which bituminous materials made up a principal part. The first mixes tried were mixtures of slow curing liquid asphalts, powdered asphalt, soil and cement.[3]

In the summer of 1942 in an attempt to find a material more satisfactory than the mud mixtures, the following materials were used: Mixtures of slow curing liquid asphalt and powdered asphalt, 60-70 penetration asphalt cement, and 50-60 penetration asphalt cement. The mixture of slow curing liquid asphalt and powdered asphalt was found to be impractical because of the difficulty of pumping the material with the equipment available and because the fluxing of the powdered asphalt with the liquid asphalt was very slow.

[2] - See Page 133 of Appendix

[3] - See Page 133 of Appendix

TABLE 3 - SUMMARY OF TEST RESULTS AND MOISTURE CONTENTS FOR 213 SUBGRADE SAMPLES TAKEN THROUGH HOLES DRILLED IN CONCRETE PAVEMENTS

Depth beneath pavement Inches	Soil Type Ohio Classification	No. of Samples	Average Test Constants: Pass #200 Sieve, %	Silt %	Clay %	Liquid Limit	Plasticity Index	Moisture Content, %: Max.	Min.	Ave.
LORAIN COUNTY, S. H. 291, SECTION E (PT.) & F (PT.), S. R. NO. 18										
3-6	2	2	36.4	-	-	29.1	10.1	14.6	13.6	14.1
13-15		1	36.1	-	-	27.6	11.3	12.9	12.9	12.9
3-6	9	2	56.7	-	-	24.6	9.4	15.9	10.6	13.2
15-18		2	68.4	-	-	24.7	8.5	13.6	11.7	12.6
3-6		5	71.3	-	-	28.8	11.5	18.7	11.7	16.5
15-18	11	6	69.5	-	-	31.6	13.2	20.8	13.6	17.9
27-30		8	75.0	-	-	30.1	12.6	17.1	13.7	15.4
3-6		1	83.6	-	-	36.0	16.9	20.1	20.1	20.1
15-18	15	2	80.2	-	-	33.6	17.0	17.0	16.1	16.6
27-30		1	81.4	-	-	35.5	16.6	19.1	19.1	19.1
3-6	17	1	76.2	-	-	37.4	21.1	21.4	21.4	21.4
15-18		1	83.7	-	-	52.5	34.3	17.6	17.6	17.6
SCIOTO COUNTY, S.H. 7, SECTION O (PT.), R-1, R-2a & R-2b, U.S.R. #52										
3-6	**	11	19.8	-	-	20.9	2.4	25.1	4.1	9.5
15-18	SS-112	11	5.9	-	-	21.0	1.2	9.2	4.6	7.0
27-30		3	7.4	-	-	Non-Plastic		7.2	6.4	6.7
27-30	2	1	34.2	-	-	31.2	12.4	23.0	23.0	23.0
3-6		1	14.0	-	-	17.6	3.5	10.8	10.8	10.8
15-18	3	1	8.1	-	-	Non-Plastic		6.3	6.3	6.3
27-30		5	7.7	-	-	Non-Plastic		6.9	5.7	6.6
3-6		6	87.1	-	-	30.1	7.7	31.9	18.5	23.1
15-18	8	6	80.2	-	-	28.2	7.6	21.7	15.5	18.5
27-30		11	82.5	-	-	28.0	6.6	26.3	12.7	18.9
3-6		8	69.5	-	-	25.6	6.8	20.7	15.0	17.6
15-18	9	8	67.4	-	-	24.9	5.2	21.1	11.0	16.2
27-30		8	64.0	-	-	24.3	5.9	18.5	12.9	16.2
SCIOTO COUNTY (CONTINUED)										
3-6		8	77.4	-	-	33.5	10.9	25.6	17.0	20.7
15-18	11	7	80.4	-	-	34.1	11.6	24.2	17.0	21.0
27-30		6	89.0	-	-	34.2	11.5	27.4	16.3	20.6
3-6		1	97.2	-	-	39.3	13.1	21.6	21.6	21.6
15-18	12	3	92.0	-	-	40.0	13.0	29.4	20.3	24.1
27-30		2	90.0	-	-	39.9	12.5	33.3	24.3	28.8
3-6		5	93.6	-	-	41.1	16.0	26.5	21.5	24.9
15-18	15	4	95.2	-	-	41.0	16.2	28.0	20.5	25.[illegible]
27-30		3	97.5	-	-	39.9	15.3	26.9	24.0	25.0
27-30	16	2	96.3	-	-	48.9	22.2	27.5	25.8	26.6
*MEDINA COUNTY, S.H. 95, SECTION A & B (PT.), S.R. #18										
0-10	8	3	78.6	52.2	26.4	24.0	7.1	24.7	14.8	19.2
10-22		2	82.5	61.9	20.6	23.1	5.7	22.3	18.8	20.6
0-8		7	63.1	39.4	23.7	25.4	8.1	26.9	14.9	18.3
8-20	9	5	69.5	42.9	26.6	24.4	8.2	22.0	13.6	16.4
20-30		3	53.8	33.0	20.8	24.3	8.3	17.7	15.0	16.1
0-8	11	4	73.1	44.2	28.9	30.9	13.2	23.1	14.6	18.7
8-20		5	74.9	44.4	30.5	28.5	12.0	20.1	15.4	16.9
0-12	16	1	72.9	43.5	29.4	47.7	27.0	15.2	15.2	15.2
*PORTAGE COUNTY, S.H. 322, SECTION L,W,T,V & U, S.R. #5										
0-10	9	1	82.6	47.7	34.9	28.7	6.6	25.9	25.9	25.9
0-6		10	78.5	43.2	35.3	31.8	12.9	27.2	16.8	21.1
6-18	11	10	79.1	39.8	39.3	31.1	12.1	25.3	15.1	18.3
18-30		7	81.1	38.9	42.2	31.7	12.5	31.1	15.2	18.3
10-22	12	1	84.1	46.6	37.5	40.7	12.8	25.0	25.0	25.0
10-24	15	1	89.8	39.3	50.5	39.2	18.4	26.5	26.5	26.5

** Granular subbase material furnished under specification shown in Table 4.

*Samples on these projects taken through holes drilled for pavement jacking.

SOIL PROFILE
HAMILTON CO.
S.H. 44
SEC.H

STATE HIGHWAY TESTING LABORATORY
O S U CAMPUS, COLUMBUS, OHIO

1/5

NOTE: THE INFORMATION SHOWN BY THIS SUBGRADE PROFILE WAS SECURED FOR THE INFORMATION OF THE STATE OF OHIO. THE STATE DOES NOT GUARANTEE THE CORRECTNESS THEREOF AND DOES NOT INCLUDE IT AS A PART OF THE PLANS GOVERNING THE CONSTRUCTION OF THE PROJECT

LEGEND FOR PROJECT-AVERAGE RESULTS OF TESTS–37 SAMPLES TESTED

	P.R.A. CLASS	B.H.T.L. CLASS	% AGG	% C. SAND	% F SAND	% SILT	% CLAY	LIQUID LIMIT	PLASTICITY INDEX	WATER CONTENT	SAMPLES TESTED
GRAVEL, SAND, & SILT	A-2	2	56.3	17.6	3.7	17.7	4.7	33.7	14.3	12.3	1
GRAVEL & SAND	A-2	3	46.7	32.7	6.2	13.3	1.1	17.4	3.4	4.7	1
SAND	A-3	4	2.8	58.1	14.3	14.4	8.4	18.5	4.4	8.2	1
SILT	A-4	8	-	1.0	0.4	67.2	31.4	32.4	11.5	12.4	1
SANDY SILT	A-4	9	4.8	16.3	22.2	38.2	18.5	21.9	6.2	17.1	2
SANDY SILT & CLAY	A-4	11	7.2	14.0	11.7	37.9	28.6	29.4	11.8	15.0	2
CLAY & SILT	A-7	15	1.0	4.5	6.5	51.3	36.7	36.8	17.8	22.8	4
CLAY	A-7	16	3.7	4.9	2.1	43.3	44.0	44.2	25.2	24.8	23
CLAY	A-6	17	-	6.0	6.7	46.7	40.6	40.5	23.1	22.2	2

AUGER BORINGS - TO VERTICAL SCALE

CRACK IN PAVEMENT

BITUMINOUS CONCRETE PATCH

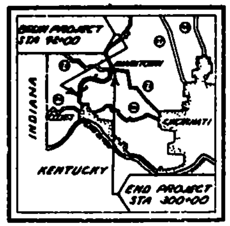
HOLE DRILLED FOR MUD JACKING

HOLE DRILLED FOR MUD JACKING. SUBGRADE SAMPLES TAKEN

PAVEMENT

SAMPLES TESTED *
LAB. NOS 50. 41787-41815, 41868-41909, 41935-41975

* INCLUDES SAMPLES TESTED FOR MOISTURE ONLY



LOCATION MAP

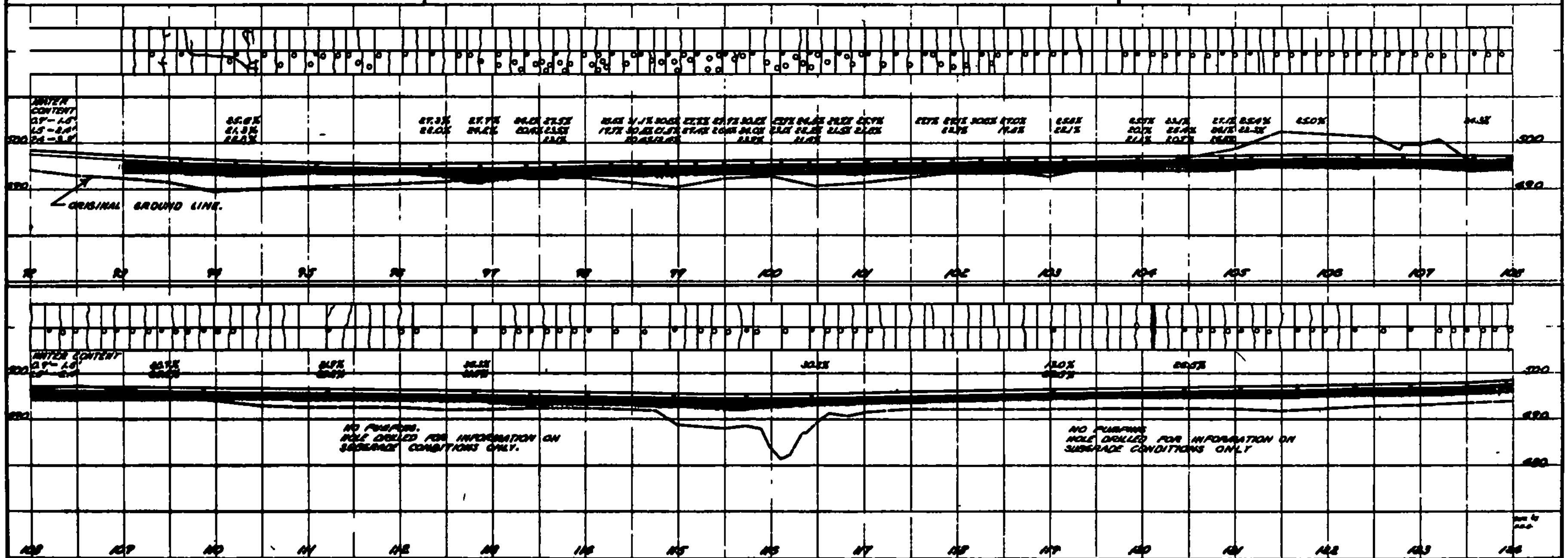


FIGURE 1

TABLE 4 - GRADING REQUIREMENTS FOR SUBBASE MATERIALS COMMONLY USED IN OHIO

Sieve	TOTAL PER CENT PASSING		
	Grading 1	Grading 2	Grading 3
3 Inch	100	100	
2 "			100
1 "	30-70	75-100	
1/2 "		50-90	
#10	0-25	35-75	50-100
#200		0-15	0-15

For the fraction of these materials passing the No. 40 sieve the liquid limit shall be not greater than 35 and the plasticity index not greater than 10.

TABLE 5 - GRADING REQUIREMENTS FOR SUBBASE MATERIALS PROPOSED FOR USE ON FUTURE PROJECTS

Sieve	TOTAL PER CENT PASSING			
	Grading A	Grading B	Grading C	Grading D
3 Inch	100			
2 "	30-70	100	100	100
1 "		70-100	70-100	
1/2 "		40-100		
3/8 "	15-40			
No. 4		0-40		
No. 10	0-15	0-15	35-75	50-90
No. 50			0-10	0-20

For the fraction of these materials passing the No. 40 sieve the liquid limit shall be not greater than 35 and the plasticity index not greater than 10.

Approximately a thousand joints and cracks were treated with 60-70 penetration asphalt in 1942 and only a very few of them were pumping mud in the early fall of 1944. However, there were some instances of exuding of the asphalt cement from the cracks and joints. About two hundred joints were treated this same year with the 50-60 penetration asphalt cement and although this asphalt showed less exuding than the 60-70 penetration material, it was thought that a higher melting point material with a lower temperature susceptibility would be desirable. Therefore, in 1943 and 1944 the Ohio Department of Highways' Specification M-5.4, F-1, approximating the A.A.S.H.O. Oil Asphalt Filler Grade A, Designation: M 18-42 was used. This material has given very satisfactory results to date and no difficulty has been experienced with bitumen exuding from the joints or cracks. In a few instances it has been necessary to go back over the pavement after the first treatment and retreat some joints that still pump.

As an indication of the amount of material necessary to treat pumping joints it was found that on one project treated this past summer that an average of 40 gallons per joint was used to treat 284 joints. The quantity of material, of course, varied considerably for individual joints. At some joints as much as 60 gallons of material have been used without raising the slab. Our Maintenance Bureau outlines the following equipment and procedure that has been used satisfactorily in conducting this work.

EQUIPMENT

Air compressor, jack hammer and drills to drill holes and blow out mud and water.

A bituminous pressure distributor equipped with a patching hose and a home-made barrel, bung type, bituminous pump nozzle which is to be put in the hole in the pavement and driven snug with a hammer before asphalt pumping is started. See Figures 3 and 4. The bituminous distributor should be equipped with a by pass pressure regulator so that pressures between 20 and 40 lbs. per sq. in. can be maintained. It should also be equipped with a reversible pump or a suck back arrangement so that a small amount of the asphalt may be sucked out of the hole immediately before removing the nozzle in order to prevent asphalt squirting out on the pavement.

A sprinkling can and water to wet the pavement around the hole so that any asphalt leakage can be easily removed. The water is also used to chill any asphalt that may break out of a crack or joint before the desired amount has been pumped under the pavement.

Soft wood cylindrical plugs turned to a diameter 1/8 in. larger than the hole to be driven in the hole after the treatment is completed.

LABOR

Six or eight men are required in the gang. Two or three with the compressor to drill and blow out holes and four or five with the distributor to do the pumping.

SEQUENCE OF OPERATION

A short trench is dug at each end of the joint or crack to be pumped to slightly below the depth of the pavement slab. This serves as a well for the mud and water blown from under the slab and also for observation when pumping in the asphalt. A hole is drilled through the pavement usually located about one foot ahead of the joint in

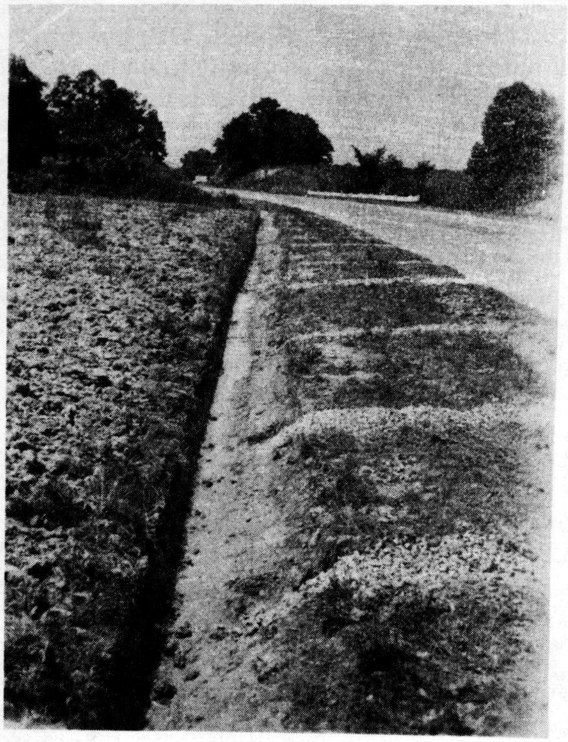


FIGURE 2. STONE DRAIN INSTALLATION FOR REMOVAL OF FREE WATER FROM BENEATH PUMPING JOINTS AND CRACKS.

FIGURE 3. NOZZLE FOR PUMPING BITUMINOUS MATERIAL BENEATH THE PAVEMENT.

the direction of travel and one to two feet away from the center longitudinal joint. Water and mud are blown out by forcing compressed air into the hole using the same type nozzle as described above. In especially wet areas it is desirable to blow out the water and the mud immediately before injecting the asphalt. In dry areas this operation may be carried out considerably in advance of pumping asphalt under the slab.

The asphalt to be used should be heated to a temperature of from 350° to 400° F. The injector nozzle is attached to the patching hose of the distributor and driven into the hole. Some water is sprinkled around the hole to wet the pavement so that any spillage may be removed easily and the asphalt pumping is begun.

Pressures of from 20 to 35 lbs. per sq. in. have been found to be entirely adequate in filling the space beneath the slab and even raising the slab. Using this comparatively low pressure it can be seen that a man standing on the nozzle plate will hold the nozzle securely in the hole.

The pumping is continued until the asphalt exudes from under the pavement at the observation trench or until the slab starts to raise. If the slab starts to raise before asphalt exudes from under the edge of the pavement or if asphalt exudes from one edge of the pavement and not the other, it may be desirable to drill another hole and attempt to force in more asphalt. Should

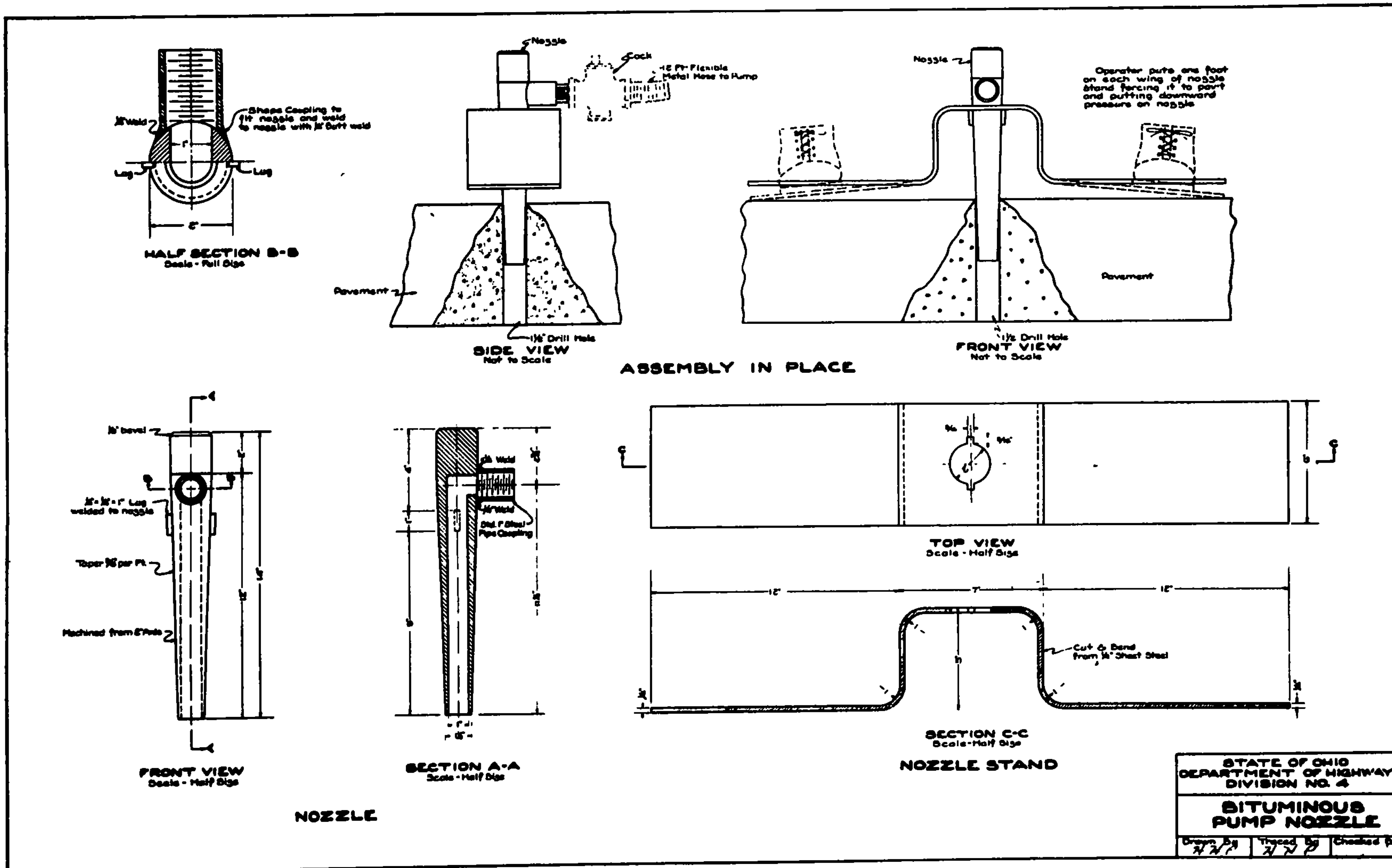

HALF SECTION B-B
Scale - Full Size
Shape Coupling to fit nozzle and weld to nozzle with ⅛" Butt weld
Lug
Nozzle
Cock
12 Ft Flexible Metal Hose to Pump
Pavement
1⅛" Drill Hole
SIDE VIEW
Not to Scale
Operator puts one foot on each wing of nozzle stand forcing it to pavt and putting downward pressure on nozzle
1½" Drill Hole
FRONT VIEW
Not to Scale
ASSEMBLY IN PLACE
⅛" bevel
Taper ⅝" per Ft.
Machined from 2" Axle
FRONT VIEW
Scale - Half Size
Std. 1" Steel Pipe Coupling
SECTION A-A
Scale - Half Size
NOZZLE
TOP VIEW
Scale - Half Size
Cut & Bend from ⅛" Sheet Steel
SECTION C-C
Scale - Half Size
NOZZLE STAND
STATE OF OHIO
DEPARTMENT OF HIGHWAYS
DIVISION NO. 4
BITUMINOUS
PUMP NOZZLE
Drawn By
Traced By
Checked By

Figure 4.

the asphalt break out at a crack or joint on the surface of the pavement during the pumping operation, pumping should be stopped a short time, the asphalt chilled with water and the pumping then continued. If the leakage continues after this treatment it can usually be stopped by placing over it a chunk of stiff clay or mud which is held down by a man standing on it. When sufficient asphalt has been pumped in, the pump is stopped but the nozzle is allowed to remain in the hole for 30 seconds before it is withdrawn. If the distributor is not equipped with a suck back arrangement, the pump motor should be reversed for a few seconds before the nozzle is withdrawn so that the asphalt will not exude from the hole before the plug can be inserted. Immediately upon the removal of the nozzle the soft wood plug is driven into the hole flush with the pavement and the spillage scraped off.

This same procedure has been used successfully to raise slabs except that the holes are usually drilled 18 to 24 in. from the crack or joint and midway between the center joint and edge of the pavement. Should the slab start to raise at a high place a loaded truck may be backed over it and thus hold down the part of the slab that is already high enough. We have generally obtained our asphalt shipped hot in insulated tank cars directly from the refinery. However, it generally arrives at slightly too low a temperature to be pumped into the distributor and some time it is necessary to heat eight or ten hours with steam at 80 to 90 pounds pressure. It usually can be pumped into the distributor at a temperature of from 280° to 300°F., after which it can be brought to application temperature in the distributor if heating in the distributor is found to be more desirable than raising the temperature the rest of the way in the tank car.

We have experienced no casualties in this operation from men being slushed with hot asphalt. However, as a safety precaution it would be desirable for men to wear heavy clothes, gloves and a welder's mask.

CONCLUSION

The use of bituminous material pumped beneath the slab to stop pumping of concrete pavements in this State has been much more successful than the various types of mud mixtures tried. In all probability one of the principal reasons for the success of this material is the fact that it forms a tight seal beneath the pavement and thus prevents the entrance of surface water. Further its stability is not appreciably affected by water which may reach it through the subgrade. Bituminous material is considerably easier to control when being pumped beneath the pavement since it apparently spreads more evenly than slurries. There is considerably less likelihood of cracking the slab than with slurries and it is easier to keep from raising the slab, or to control the amount by which the slab is raised if this is necessary. Although the costs of the material using asphalt are somewhat higher than for slurries, at least a portion of this cost differential is made up in the labor saved in assembling and mixing of the various materials.

From the experience gained to date, it is our opinion that bituminous materials show considerable promise as an effective treatment for the pumping of concrete pavements.

INVESTIGATION OF CONCRETE PAVEMENT PUMPING

H. L. Krauser, Construction Engineer
Ohio Department of Highways
Chillicothe, Ohio

SYNOPSIS

This paper describes the investigation of the pumping at transverse joints in concrete pavement slabs on a project 4.39 miles in length on U.S. 52 in Scioto County, Ohio, near the village of Franklin Furnace.

The soils survey made prior to grading operations showed that on some sections the predominating soil was high in silt content and could be classified in the A-4 group. It was considered necessary to cover these areas with suitable granular material to a depth of 18 inches and provide tile drainage.

The pavement was built without reinforcing or load transfer devices. Records are also included on a short project built in 1941 in which complete reinforcing and load transfer devices were used. Observations were made of slab deflections under moving loads on both projects.

Pumping was found to be much more extensive and severe over the areas where no granular material was used. On the plain concrete section pumping was more severe at contraction joints of the premolded type. The severity on the reinforced section was about the same for expansion and contraction joints. On the plain concrete slabs without granular sub-base, 2 per cent of the expansion joints and 52 per cent of the contraction joints were pumping. On the plain concrete slabs placed on granular sub-base 1.5 per cent of the expansion and 16 per cent of the contraction joints were pumping. On the reinforced section, 71 per cent of the expansion joints and 91 per cent of the contraction joints were pumping.

On most of the work, the gradation of the backfill for the tile drains was from the 3/4-inch to No. 4 sizes. It was observed that these drains silted up badly. On one section 3/8 inch to No. 8 material was used and only a small amount of silting was observed.

The conclusions of observations on these projects are as follows:
1. A sub-base composed of suitable granular material will appreciably reduce the pumping at joints in concrete pavements. 2. The use of small size backfill aggregate will extend the useful life of tile drains.
3. The use of load transfer devices prevents excessive permanent deformation at the joints between concrete slabs after pumping starts.

GENERAL DESCRIPTION

It is the purpose of this paper to describe the investigations that were made and the conclusions and recommendations that resulted in connection with transverse joint pumping on S.N. Federal Aid Project No. 626-H(1) and S.N. Federal Aid Project No. 240-A(3) located on U.S. Route 52 in Scioto County, Ohio near the village of Franklin Furnace.

The length of the project is 4.39 miles and the plans provided for the placing of a Portland Cement concrete wearing surface 24 ft. wide and nine in. thick. It was further provided that the "concrete pavement shall be placed and finished in single lane widths as separate operations and the longitudinal joint separating lanes thus placed shall be a key joint".

Because of wartime restrictions the pavement was designed without reinforcing steel or load transfer bars at any of the joints. The use of Portland Cement containing vinsol resin was required in amounts so that the reduction in weight of the concrete of from four to eight pounds per cu. ft. would be effected.

Most of this project is on new grade and alignment. As a consequence a considerable quantity (230,000 cu. yds.) of earthwork was involved, more than 60 per cent of which was to be secured from roadway, structure and channel excavation. A soil profile was prepared by the Highway Testing Laboratory which set forth the information obtained from the analysis of 112 samples taken throughout the length of the project. An inspection of this profile shows that the predominant soil types are silt soils, P.R.A. Classification A-4, S.H.T.L. Classifications 8, 9 and 11 (See page 57). Since these soils are subject to detrimental capillarity and subsequent frost heave it was decided that, wherever this material would be encountered at subgrade elevation, it would be removed to a depth of at least 18 in. and replaced with a suitable granular material. Deep longitudinal drainage was provided adjacent to these backfilled areas leading to the nearest convenient disposal points. This treatment was provided for seven areas which covered 7858 lin. ft. measured along the center line.

Over one area of 2250 lin. ft. where clay, P.R.A. Classification A-7, S.H.T.L. Classification 16, was encountered a bituminous impregnated paper insulation course was provided.

Actual construction work was started on August 25, 1942 when excavating operations were begun at a point approximately one-half mile south of the northern extremity of the project. Cross road culvert and underdrainage work was begun on September 1, 1942, and the first classified embankment material was placed the following day.

Approximately 20 per cent of the 9 in. Portland Cement concrete pavement was placed between October 8, 1942 and November 12, 1942 when paving work was suspended due to unfavorable weather.

The grading work was vigorously prosecuted until November 28, 1942 when weather conditions made further progress impractical. At that time approximately 50 per cent of the excavation and borrow materials were in place (See Table 1 for Mechanical analysis of borrow pit materials) and nearly 60 per cent of the classified embankment was completed.

It was possible to continue the installation of underdrainage until late in December at which time approximately 90 per cent of this work was completed.

The weather during the first four months of 1943 was unsuitable for the prosecution of construction work. Flood waters partially covered the project three times.

During the first flood period observations were made between Sta. 254+ 0 and Sta. 261+0 where the east half of the pavement slab was in place. Numerous air bubbles were in evidence at the transverse joints and along the edges of the pavement slab. The number gradually decreased until they had entirely disappeared. Subsequent floods failed to cause a repetition of this condition. This may be due to the fact that the first flood exceeded subsequent ones by approximately four feet.

TABLE 1. SUMMARY OF TESTS OF BORROW PIT SOILS FOR S.N.F.A.P. No. 626-H(1) S.N.F.A.P. No. 240-A(3)

Proj. Sample No.	Depth Sampled in Inches	Liquid Limit	Plasticity Index	Proctor Compaction: Max. Dry Wt. lbs. per Cu. Ft.	Proctor Compaction: Optimum Moisture Content	Mechanical Analysis, Percentage of: Sand +0.05 mm	Mechanical Analysis, Percentage of: Silt 0.05-0.005 mm	Mechanical Analysis, Percentage of: Clay -0.005 mm
S-1	6 to 48	20.1	2.7	115.2	13.6	19.6	39.9	13.2
S-2	48 to 96	24.2	6.6	113.0	14.2	19.2	47.5	21.7
S-3	6 to 24	24.4	6.0	107.0	15.8	36.0	37.2	26.8
S-4	24 to 72	29.7	7.7	101.5	20.2	5.6	61.6	32.8
S-14	18 to 30	23.8	3.4	107.9	15.8	11.0	66.8	22.2
S-15	30 to 36	24.7	3.5	107.2	16.4	15.0	66.2	18.8
S-16	132 to 144	19.6	Non Plastic	100.8	16.8	63.0	26.4	10.6
S-17	96 to 108	21.1	Non Plastic	105.9	14.6	70.0	20.8	9.2
S-18	48 to 60	21.0	4.5	110.1	15.2	50.0	30.8	19.2
S-22	12 to 96	26.7	2.4	100.9	17.6	1.4	85.4	13.2
S-23	12 to 120	21.6	5.1	116.8	13.2	40.0	40.2	19.8
S-25	12 to 36	21.5	2.9	114.8	12.8	29.2	55.0	15.8
S-26	12 to 36	26.0	9.4	111.1	15.8	19.8	47.2	33.0
S-27	12 to 36	31.3	13.3	107.8	15.4	9.5	45.3	44.6
S-28	12 to 36	20.7	3.5	117.6	11.6	43.0	39.6	17.4

Studies show that a considerable portion of this project was subjected to at least partial flooding during the early part of 1943.

Grading operations were resumed on June 1, 1943 and pavement placing followed in approximately two weeks. Except for ordinary delays caused by weather, equipment failures, etc., the work continued until the completion of paving operations on November 12, 1943.

SLAB PUMPING OBSERVATIONS

After the pavement had been opened to traffic for a period of between two and three months the first evidences of pumping became apparent. During this time the weather had been relatively mild and exceedingly dry. Within 30 days after the first pumping was observed this action had increased at such an alarming rate that it was decided to determine the extent to which it had progressed and to investigate any and all contributing factors.

In order to compare the action of pavement of this design with that built with reinforcing steel and load transfer bars at all joints the investigation was made to include Federal Aid Grade Crossing Project No. 240-A(2) which adjoins this project and which was placed during the late fall of 1941.

For purposes of easy identification the originally described project will be referred to as the "Plain-slab" project.

The investigation was begun by locating all transverse joints on each project. This work was done on March 27 and 28, 1944 after a series of rains made pumping joints easily identifiable. While it might seem that results observed at this time would represent extreme conditions subsequent observations indicate that average conditions prevailed. Each joint was listed as to type and condition and the results plotted. Figure 1 shows a typical section with legend markings.

As a general statement it may be said that pumping was much more extensive and severe where the pavement slab was placed on soil subgrade. It was noted that where pumping did occur over classified embankment areas the large portion of this action was confined to the low side of superelevated curves on which edge curbs were used. The water was carried to sod gutters spaced at frequent intervals and directed across the berm. During the construction of this project the sod gutters were placed one inch below the pavement edge. It was thought that this would be sufficient to allow for ordinary growth and fluffing of the sod and still have drainage away from the pavement. It was observed, however, that the sod built itself up to such an extent that, combined with an accumulation of ice control material, drainage was impeded so that water was ponded along the gutter line. This ponding with resultant splashing under traffic could have set up a condition which caused the pumping to develop.

Table 2 shows the results of this investigation. It will be observed that on the plain slab project the highest percentage of pumping occurred at the contraction joints regardless of the type of subgrade material. Pumping on the reinforced slab project is generally more severe at all types of joints. Following the usual Department policy some latitude was allowed in the choice of the type of transverse joint. Figure 2 shows the types of joints that were used on each project. Although surface sealing was not required State Maintenance forces went over the plain slab project during the fall of 1943 and performed this operation along the longitudinal center joint and along the transverse construction and expansion joints beginning at the south end of the project and extending to Sta. 244+0. When, on March 27 and 28, 1944,

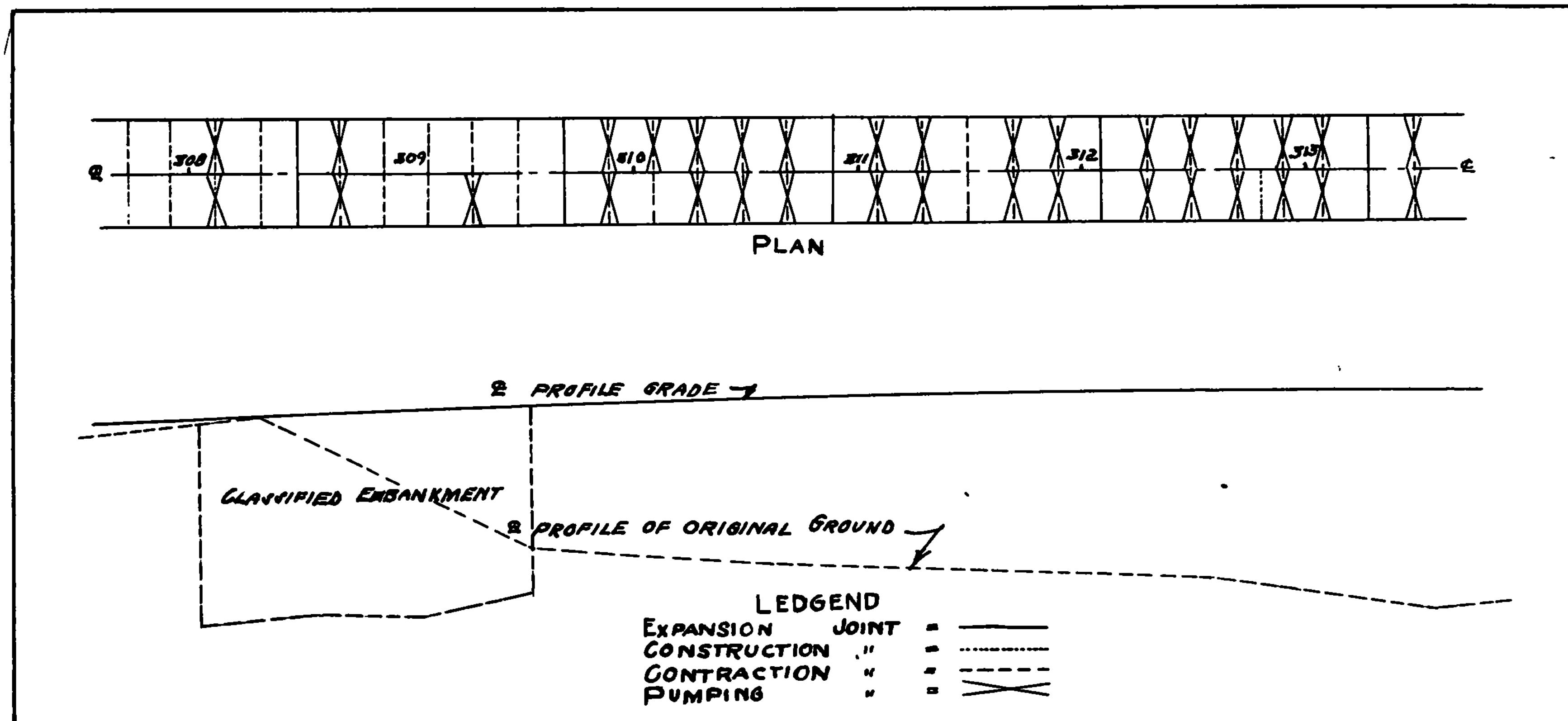


FIG. 1. SHOWING METHOD OF PLOTTING TRANSVERSE JOINTS

the location of the transverse joints was determined, it was observed that the surface seal was, generally, in good condition and showed evidence of having been well applied. Inquiry developed that asphaltic filler having a softening point of 75+ and a penetration range of 30 to 45 was used and applied when heated to a temperature that permitted easy flow of the material.

SLAB DEFLECTION OBSERVATIONS

On April 20, 21 and 26, 1944 a series of observations were made to determine the deflection of the pavement slab under traffic. The device used consisted of a metal stake and bracket to which two Ames gauges were attached. The metal stake was driven at such locations that one of the gauges was in contact with each of the pavement slabs adjacent to the transverse joint.. At each joint where observations were made the gauges were placed near the pavement edge (See Fig. 3) and near the junction with the center line joint.

An attempt was made, at first, to observe the slab deflections resulting from the passage of each vehicle over the joint. It soon became apparent that the movement under passenger cars and light trucks was so slight that it could not be measured on the dial of the Ames gauge. Observations were, therefore, confined to the action resulting from the passage of medium and heavy trucks and buses at normal operating speed of approximately 35 miles per hour. In certain instances trucks were stopped and asked to proceed at very slow speed, and it was observed that greater deflections resulted than when normal speeds were allowed.

In making these tests all traffic was routed over the half of the pavement under observation and dial readings were recorded on both the forward and rear slabs as determined by the direction in which the vehicle is moving. Table 3 shows the summary of pavement deflections. While the amount of the individual deflection is small there is every reason to believe that over a period of time serious trouble will develop. This is substantiated by observations on the reinforced slab project. In the two years that this pavement has been in place serious spalling has occurred at some transverse joints. Reference to Table 3 will show that the movement here is comparable to that on the plain slab project. That the repetition of these small deflections has affected the relative positions of the individual slabs was demonstrated as follows:

It was known that at the time work was completed on the plain slab project all pavement irregularities in excess of specification limit (1/4 inch in 10 ft.) had been removed. On May 1, 1944 the smoothness of the pavement was again tested by means of a hand profilometer. It was found that irregularities in excess of the specification limit existed at 111 transverse joints. This represents approximately 10 per cent of the total number of transverse joints on the project. The distribution of these irregular joints was fairly even throughout the length of the project.

In order to obtain specific subgrade information, samples were taken one foot from the pavement edge where slab deflections had been measured. The results of tests of these samples are shown in the last column of Table 3. It is interesting to note that at Sta. 417+68 where the most deflection was recorded and where very bad pumping was observed the silt content was lower than at transverse joints where less movement and pumping were in evidence.

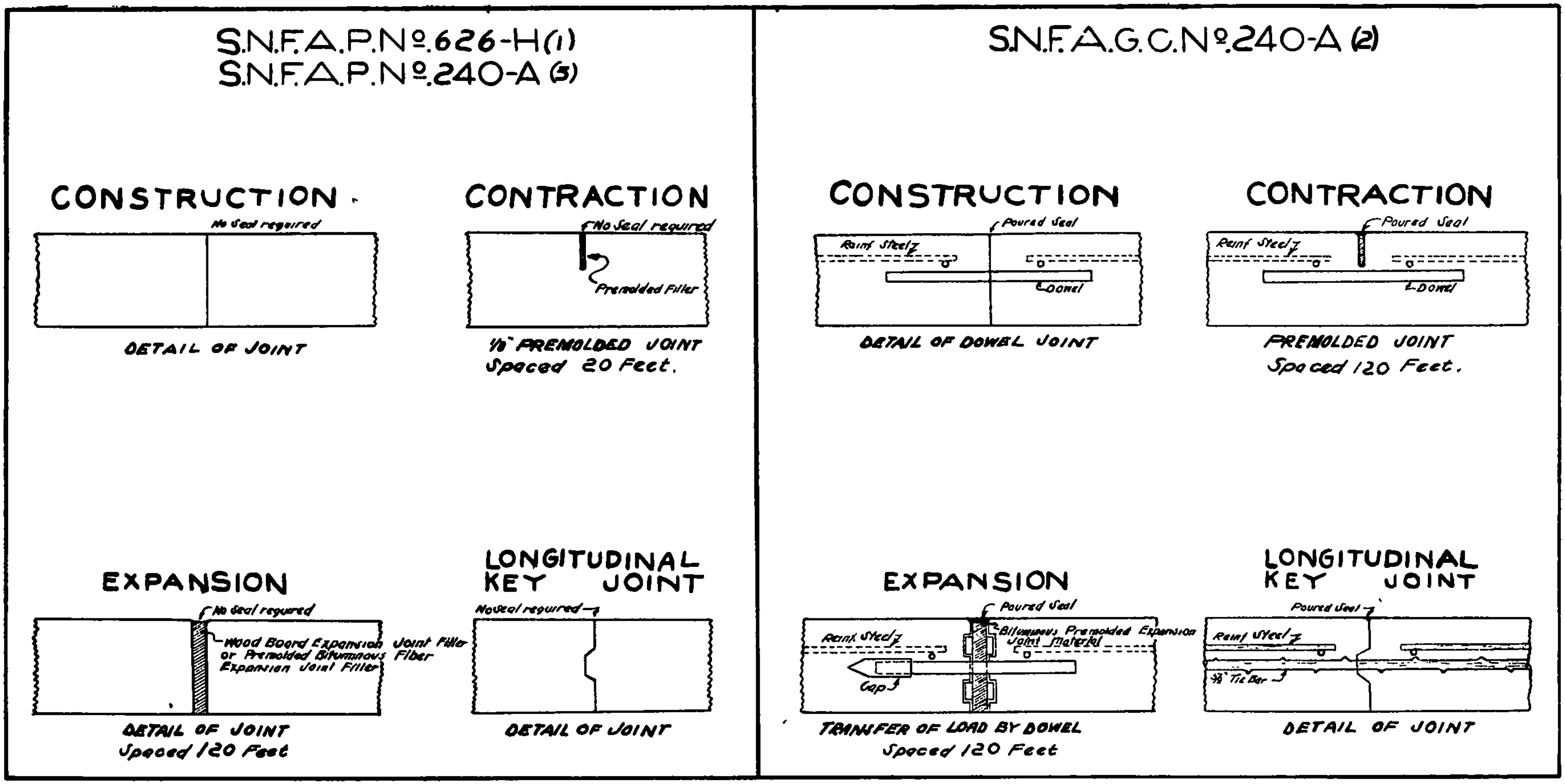


FIG. 2. PAVEMENT JOINTS

TABLE 2. SUMMARY OF TRANSVERSE JOINT INVESTIGATION FOR S.N.F.A.P. No. 626-H(1)
S.N.F.A.P. No. 240-A(3)
F.A.G.C.P. No. 240-A(2)

EXPANSION JOINT			CONSTRUCTION JOINT			CONTRACTION JOINT		
Total No.	No. Pumping	% Pumping	Total No.	No. Pumping	% Pumping	Total No.	No. Pumping	% Pumping
PLAIN CONCRETE SLAB PROJECT SOIL SUBGRADE								
130	3	2.31	29	7	24.14	622	325	52.25
PLAIN CONCRETE SLAB PROJECT CLASSIFIED EMBANKMENT SUBGRADE								
67	1	1.49	9	0	0.0	335	52	15.52
REINFORCED CONCRETE SLAB PROJECT SOIL SUBGRADE								
21	15	71.43	1	1	100.00	24	22	91.25

TABLE 3. SUMMARY OF PAVEMENT DEFLECTION UNDERMOVING WHEEL LOADS S.N.F.A.P. No. 626-H(1)
F.A.G.C.P. No. 240-A(2)
S.N.F.A.P. No. 240-A(3)

Station	Location	Number of Vehicles	Average Deflection in Inches: Forward Slab	Average Deflection in Inches: Rear Slab	Maximum Deflection in Inches: Forward Slab	Maximum Deflection in Inches: Rear Slab	Condition of Joint	Subgrade Material
313+68	6" from W. Edge	14	.0039	.0039	.011	.010	Medium Pumping	P.R.A. Class A-4 S.H.T.L. " 9
313+68	6" W. of ℄	7	.0093	.0077	.019	.015	" "	With 48.6% Silt
319+08	6" from E. Edge	8	.0043	.0055	.017	.014	Bad "	P.R.A. Class A-4 S.H.T.L. " 8
319+08	6" E. of ℄	12	.0074	.0106	.015	.021	" "	With 54.9% Silt
350+88	6" from E. Edge	7	.0004	.0007	.001	.001	No "	P.R.A. Class A-5 S.H.T.L. " 12
350+88	6" E. of ℄	10	.0005	.0004	.001	.001	" "	With 49.8% Silt
417+68	6" from E. Edge	10	.0092	.0095	.025	.025	Very Bad "	P.R.A. Class A-7 S.H.T.L. " 16
417+68	6" E. of ℄	12	.0015	.0024	.004	.010	" " "	With 33.2% Silt
435+06	6" from E. Edge	7	.0003	.0003	.002	.002	No "	S.S. 112 Grading 2
435+06	6" E. of ℄	8	.0010	.0008	.003	.003	" "	With 5.2% Passing #200 Sieve
478+96	6" from E. Edge	8	.0011	.0009	.003	.002	Bad "	P.R.A. Class A-7 S.H.T.L. " 15
478+96	6" E. of ℄	8	.0051	.0055	.011	.012	" "	With 48.3% Silt

SUBGRADE INVESTIGATIONS

The plans for the plain slab project contained the following note: "Excavated material, of which the grain size is of 50 per cent or more between 0.05 mm. and 0.005 mm. (S.H.T.L. Classification 8), shall be placed at least three feet below the pavement when used in embankment". The Soil Profile indicated that a number of areas from which roadway excavation material was to be obtained contained more than the maximum allowable amount of S.H.T.L. Classification 8. Table 1 shows the amounts of this type of material contained in borrow pit soils.

Because of the limitations set forth by the note quoted above and the presence of S.H.T.L. Classification 8 material in both the excavation and borrow material it was decided to sample the subgrade soil of the completed project. The results of tests of these samples are summarized in Table 4. While a number of the samples (Nos. 9, 13, 24) indicate an excess of undesirable material in embankment areas, the amount of pumping does not seem to be particularly affected thereby. This may be due to the relatively high percentages of compaction that were obtained on this project. An average of 99.75 per cent compaction was obtained from 299 tests completed during the construction operations. The three other samples (Nos. 6, 17, 18) which contain an excess of S.H.T.L. Classification 8 material seem to bear out the statement that the amount of pumping is not particularly affected by its presence. These latter samples are all from within excavation areas.

The limitations for fineness of classified embankment material is that not more than 15 per cent shall pass a No. 200 opening. Table 5, which is a summary of tests of samples of classified embankment in place, indicates that in a number of places this limitation is exceeded. In all probability this is caused, for the most part, by the mixing of some of the berm material in the sample and to the unavoidable combining of earth with the classified embankment material during construction operations. These contentions are further borne out by Table 6 which shows the mechanical analysis of samples taken from the pit which was the source of the classified embankment material. Sample numbers 1, S-24 and 34 were taken before and during construction operations and sample numbers 40 and 41 were taken recently. The gradation of the material represented by the two sets of samples are closely comparable. But regardless of the cause of the presence of an excessive amount of fine material in the classified embankment material, the amount of pumping does not seem to be affected thereby.

The limitations of the General Specifications provided the control of the type of material and its manipulation in the embankment on the reinforced slab project. Since practically all of this project consisted of embankment to be obtained from borrow pits a soil profile was not needed. Table 7 shows a summary of the tests of material obtained from these pits. Subgrade samples taken during April 1944 conform generally to that shown in the Table. It is interesting to note that only one of the borrow pit samples and none of the subgrade samples indicate the presence of more than 50 per cent silt. From 311 tests completed during the construction of this project an average of 102.4 per cent compaction was indicated. However, pumping is prevalent throughout the project to a very high degree. The presence of edge curbs and the steepness of the grade which caused considerable longitudinal flow of water over the pavement slab may be a contributing factor to the pumping especially since the bituminous seal at the transverse joints was rather poorly maintained.

TABLE 4. SUMMARY OF TESTS OF * SUBGRADE SOILS AS BUILT FOR SN.F.A.P. No. 626-H(1) SN.F.A.P. No. 240-A(3)

Proj. Sample No.	Location	Depth Taken in Inches	Liquid Limit	Plasticity Index	Mechanical Analysis Percentage of Sand +0.05mm	Silt 0.05-0.005mm	Clay -0.005mm	Classification P.R.A.	S.H.T.L.
1	1' Lt. Pavt. Sta. 232+85	12 to 36 Fill	21.5	4.9	32.2	29.5	9.1	A-4	Class 9
2	1' Rt. " " 237+41	" " " "	24.6	6.5	23.0	46.7	26.9	A-4	" 9
3	1' Rt. " " 246+60	" " " "	22.9	4.4	33.6	42.6	23.8	A-4	" 9
4	1' Lt. " " 246+00	" " " "	21.9	4.2	33.0	43.2	23.8	A-4	" 9
5	1' Lt. " " 254+60	" " " "	27.9	8.2	20.8	46.4	32.8	A-4	" 9
6	1' Rt. " " 263+40	" " " Cut	32.4	9.8	6.1	56.4	34.8	A-4	" 11
9	1' Lt. " " 275+20	" " " Fill	36.9	13.0	7.4	53.2	39.1	A-4	" 11
13	1' Lt. " " 287+60	" " " "	31.9	9.4	6.7	57.7	35.3	A-4	" 11
17	1' Rt. " " 302+12	" " " Grade	26.3	7.2	13.3	50.4	34.0	A-4	" 8
18	1' Lt. " " 321+48	" " " Cut	26.3	7.5	16.2	55.5	27.1	A-4	" 8
24	1' Rt. " " 352+28	" " " Fill	38.7	12.2	4.0	53.9	41.0	A-4	" 8
25	1' Lt. " " 326+68	" " " "	35.5	11.8	4.6	47.6	47.8	A-7	" 15
26	1' Rt. " " 372+08	" " " Grade	35.4	13.0	1.6	45.4	52.8	A-7	" 15
31	1' Lt. " " 414+88	" " " "	45.1	19.5	3.2	30.0	66.8	A-7	" 16
32	1' Lt. " " 425+08	" " " Fill	36.3	13.1	6.1	43.5	50.0	A-7	" 15
36	1' Lt. " " 457+42	" " " "	30.2	11.4	16.1	45.3	34.1	A-4	" 11
42	1' Rt. " " 456+00	" " " "	25.7	8.3	19.3	49.1	25.7	A-4	" 9

*These Tests represent Subgrade exclusive of areas where Classified Embankment Material was used. See Table 5 for this summary.

TABLE 5. SUMMARY OF TESTS OF CLASSIFIED EMBANKMENT MATERIAL IN PLACE FOR SN.F.A.P. No. 626 H(1)
SN.F.A.P. No. 240 A(3)

Proj. Sample No.	Location	Depth Taken in Inches	Liquid Limit	Plastic Index	Mechanical Analysis Total % Passing 2"	1"	1/2"	#10	#200
7	6" Lt. Pavt. Sta. 268+00	12 to 42	14.9	3.1	100.0	100.0	92.1	52.3	15.3
8	" " " " 270+80	" " "	16.9	3.2	100.0	100.0	93.6	56.9	14.7
10	" " " " 281+60	" " "	15.6	2.1	100.0	100.0	94.2	54.8	12.9
11	" Rt. " " 283+60	" " "	15.4	2.5	100.0	100.0	91.5	47.1	10.0
12	" Lt. " " 284+60	" " "	19.4	5.2	100.0	100.0	93.8	56.5	22.2
14	" " " " 387+80	" " "	19.4	4.2	100.0	100.0	99.6	63.5	26.6
15	" Rt. " " 291+00	" " "	17.1	3.5	100.0	100.0	85.5	26.3	8.5
16	" Lt. " " 293+40	" " "	17.9	3.5	100.0	100.0	98.0	64.8	22.2
19	" " " " 320+88	" " "	16.9	4.2	100.0	100.0	93.6	56.8	14.9
20	" Rt. " " 324+68	" " "	18.8	4.8	100.0	100.0	98.3	73.3	37.9
21	" Lt. " " 330+00	" " "	18.5	5.2	100.0	100.0	97.4	64.9	27.7
22	" " " " 338+00	" " "	18.5	4.4	100.0	100.0	93.9	57.9	20.4
23	" Rt. " " 342+28	" " "	18.5	4.7	100.0	100.0	90.2	55.2	20.1
27	" " " " 377+00	" " "	13.2	2.0	100.0	100.0	88.2	53.1	12.7
28	" " " " 390+00	" " "	14.6	2.6	100.0	100.0	95.2	52.7	8.9
29	" " " " 400+00	" " "	17.7	3.9	100.0	100.0	95.7	56.3	14.6
30	" Lt. " " 407+00	" " "	Non Plastic		100.0	100.0	98.0	51.8	4.5
33	" Rt. " " 431+08	" " "	17.4	4.2	100.0	100.0	94.9	56.9	4.2
34	12"Lt. " " 440+08	" " "	16.4	4.8	100.0	100.0	93.0	55.2	12.9
35	6" Rt. " " 444+28	" " "	16.4	2.9	100.0	100.0	95.1	54.5	8.8

TABLE 6. SUMMARY OF TESTS OF CLASSIFIED EMBANKMENT MATERIAL FOR SN.F.A.P. No. 626-H(1)
SN.F.A.P. No. 240-A(3)

Proj. Sample No.	Location	Depth Taken in Inches	Liquid Limit	Plastic Index	Mechanical Analysis Total % Passing 2"	1"	1/2"	#10	#200
1	Borrow Pit	12 to 42	Non Plastic		100.0	100.0	95.3	47.2	1.5
S-24	" "	" " "	"	"	100.0	100.0	89.7	42.5	1.0
34	" "	" " "	"	"	100.0	100.0	92.8	43.9	3.2
40	" "	" " "	"	"	100.0	100.0	96.2	54.4	5.4
41	" "	" " "	"	"	100.0	94.4	86.4	48.0	3.9

TABLE 7. SUMMARY OF TESTS OF BORROW PIT SOILS FOR SN.F.A.G.C.P. No. 240-A(2)

Proj. Sample No.	Depth Sampled in Inches	Liquid Limit	Plasticity Index	Proctor Compaction Max. Dry Wt. Lbs. Per Cu. Ft.	Optimum Moisture Content	Mechanical Analysis Percentage of Sand +0.05 mm	Silt 0.05-0.005 mm	Clay -0.005 mm
S-1	6 to 40	29.2	8.5	111.0	16.4	18.9	48.6	32.3
S-2	6 to 40	32.0	10.6	107.6	17.6	9.8	46.8	43.4
S-3	6 " 120	35.4	12.5	105.8	17.2	11.6	39.1	43.5
S-4	6 " 120	30.0	9.3	107.1	18.2	16.4	50.4	33.2
S-5	6 " 48	34.2	12.7	103.4	20.6	12.2	48.6	39.2
S-6	6 " 48	36.3	12.4	107.5	18.0	12.7	31.2	36.8
S-7	6 " 48	36.3	10.6	104.1	21.4	13.5	31.1	36.8
S-8	6 " 48	33.4	18.8	108.3	17.8	15.0	42.6	36.2
S-9	6 " 48	33.0	12.2	107.6	16.4	15.0	41.6	38.2
S-10	6 " 48	40.3	16.6	102.9	21.8	14.9	32.8	43.4
S-11	6 " 48	34.3	12.0	114.9	14.8	25.8	23.7	41.4
S-12	6 " 84	38.9	16.4	101.0	20.4	4.0	39.6	56.4
S-13	6 " 48	35.2	13.7	105.8	18.0	11.2	38.4	50.4
S-14	6 " 48	37.8	14.3	103.0	20.4	10.4	27.1	45.7

BERM AND UNDERDRAINAGE INVESTIGATIONS

On the left of Sta. 294+50 is located a sod gutter for disposal of pavement surface water. In an effort to check the movement of water through the subgrade one-half of this sod gutter was dug out for the width of the berm and to a depth of the thickness of the pavement slab. It was not possible to check the movement of the water, but it was observed that the bottom of the trench was damp one week after it was opened.

A trench of similar depth was dug through the berm on the left of Sta. 314+25 at the end of a contraction joint. Clear water ran from beneath the pavement slab when the trench was opened. The bottom of the trench was damp one week later.

On the left of Sta. 417+68 where a trench was cut through the berm, water was being worked from beneath a contraction joint by traffic one week later. Figure 4 shows this trench. This location is at the transverse joint where the greatest deflection was observed as shown by Table 3.

In order to check the functioning of the No. 46 size (3/4 in. to No. 4) porous backfill material that was used over the longitudinal roadway drainage pipe placed adjacent to classified embankment areas, the material was removed at one location down to the top of the pipe and more or less vertical faces of undisturbed material were exposed. Figure 5 shows one such vertical face on the left of Sta. 337+10. At this location it was observed that eight inches of heavily silted aggregate was in place immediately above the top of the pipe. Above this was approximately 20 in. of aggregate with a very small amount of silt. Above this was a layer, approximately 12 in. thick, composed of predominately silty material. The flow line of the ditch has been raised approximately six inches by silt depositing in it during the five month period that has elapsed since the contract work was completed on this project. While the porous backfill is apparently functioning satisfactorily at the present time, it is doubtful whether it will do so indefinitely with the continuance of the depositing of silt.

Excavation on the left of Sta. 290+20 showed the same general characteristics with regard to the silt deposited in and above No. 46 size porous backfill aggregate.

On the left of the center line between Sta. 431+0 and Sta. 445+0 it was planned to install roadway drainage pipe with No. 46 size porous backfill material adjacent to a classified embankment area. During the course of construction operations it became necessary to change the design to the following: The open joints in the line of pipe were wrapped with burlap and No. 6 size porous backfill aggregate was placed around the pipe and up to a point 6 in. above the top of the pipe. From here to the flow line of the ditch classified embankment material was used. An examination was made of one section, and it was found to be functioning satisfactorily with very little silt having filtered into the porous material. It was observed, however, that the flow line of the ditch was built up with deposited silt to a greater extent than in the two previous places that were examined where No. 46 size aggregate was used.

An examination was made of the outfall end of all pipe underdrains on this project, and they were all apparently doing the job for which they were intended after having been in place from seven to twelve months.

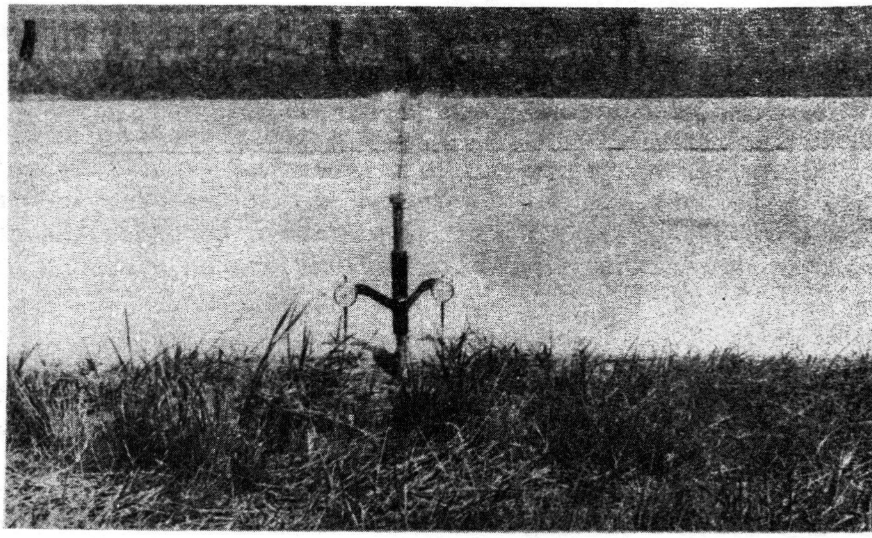


FIGURE 3. AMES DIAL GUAGE IN POSITION TO MEASURE SLAB DEFLECTIONS.

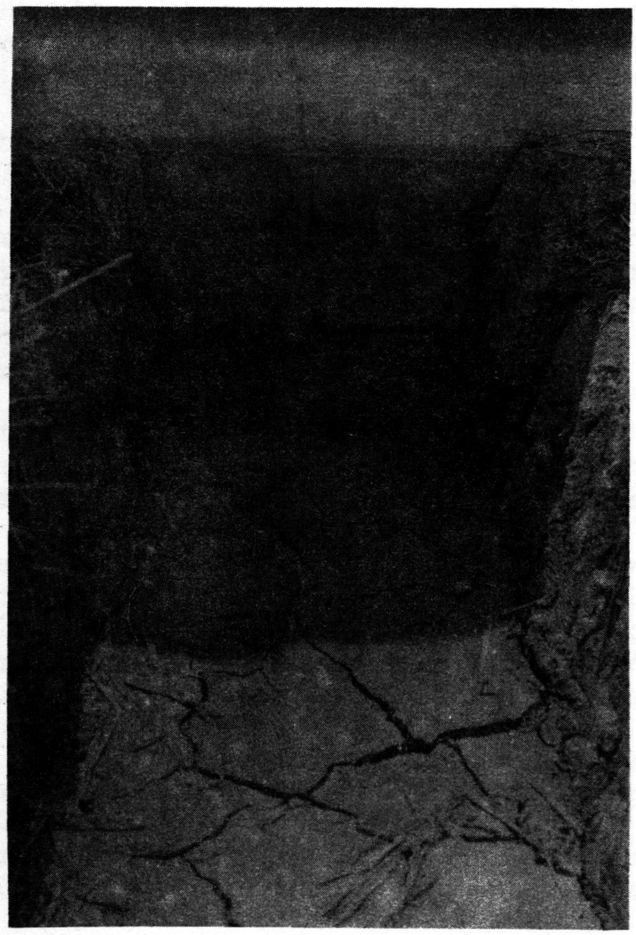


FIGURE 4. TRENCH CUT THROUGH BERM FROM PAVEMENT EDGE. WATER RAN FROM BENEATH THE PAVEMENT JOINT WHEN TRENCH WAS OPENED.

FIGURE 5. VERTICAL FACE OF #46 POROUS BACKFILL OVER PIPE. NOTE HEAVY SILT CONTENT FOR APPROXIMATELY 8" OVER PIPE.

CONCLUSIONS

As a result of the previously described tests and observations the following conclusions are reached and recommendations are made. It should be borne in mind that conditions were observed and tests made on only the two projects and the conclusions and recommendations may not be universally applicable. They will be presented in the reverse order from that in which the description was written.

When using gravel for porous backfill over pipe underdrainage, it is recommended that the size be not larger than No. 6 (3/8 in. to No. 8). Despite the high coefficient of permeability (187.5 ft. per day) that was obtained from tests of the No. 46 porous backfill material taken from immediately above the underdrainage pipe, heavy silting has occurred and will eventually reach a point where the functioning of the system will be greatly impaired if not entirely invalidated. It is further recommended that the open joints of the underdrainage pipe be wrapped with burlap to prevent the finer particles of the porous backfill material from being carried through these joints.

The plans for the plain slab project provided that the porous backfill material extend upward to the flow line of the ditch indicating that it was the intention to dispose of the surface run-off through the porous backfill material. Heavy silt deposits have, at least partially, sealed the flow line so that the surface water must travel longitudinally along the ditches for some distance to the nearest inlet or cross road culvert. The gradients of the ditches are not sufficient to prevent further depositing of silt during the longitudinal flow of the water. The build-up in the elevation of the flow line of the ditches previously described has resulted.

It is recommended that, especially where S.H.T.L. Classification 8 material is encountered, the top of the porous backfill material be at a point approximately six inches below the flow line of the ditch and that some type of impervious material be used above this point. The gradient of the ditch should be not less than 0.5 per cent and suitable vegetation should be provided to prevent erosion.

The advisability of using classified embankment material for subgrade treatment has been clearly demonstrated by the prevalence of pumping over areas where it was not used and the comparative freedom from pumping in areas so treated. Reference to Table 3, particularly to the last column, would seem to indicate that classified embankment material should be used in all roadway excavation areas where silt in excess of 25 per cent or more is encountered at subgrade elevation. The limitations regarding the use of excavated materials provided by the plans for the plain slab project should be altered to read "Excavated material, of which the grain size is 25 per cent or more between 0.05 mm. and 0.005 mm. shall be placed at least three feet below the pavement when used in embankment".

Based on the successful functioning of the classified embankment on this project up to the present time and on the mechanical analysis of samples taken from the project, it would appear that the fineness limitations of the specifications might be exceeded without causing harmful results. This statement should be further qualified by application only when bank run aggregate is used. No recommendation is made as to the ultimate fineness of material that might be used since opportunity was not afforded for any tests on which to base a statement.

The drainage difficulties caused by the fluffing and natural growth of the sod indicate, that on sections where edge curb is used, some other method of disposing of the water that accumulates on the surface of the pavement should be used in place of sod gutters. Because of the prevalence of pumping on the reinforced slab project where catch basins were spaced at approximately 350 ft. it is recommended that disposal units be placed at not to exceed 250 ft. to prevent a detrimental accumulation of water on the pavement surface.

By far, most of the pumping occurred at the contraction joints where the premolded or "ribbon" type was used. On the assumption that a more positive seal can be maintained it is recommended that the alternate provided by most plans, the impressed joint, be used. It is also recommended that a surface seal be required for all types of joints, since, regardless of the amount of care that is exercised during construction some variation will be obtained in the depth of the top of the joint material beneath the pavement surface.

The roughness of the plain slab pavement project at the present time proves the advisability of providing dowels and tie bars at transverse and longitudinal joints respectively.

It is hoped that the foregoing conclusions and recommendations may prove to be beneficial in avoiding on future projects the difficulties that were encountered on the projects under observation.

Very helpful assistance and cooperation was extended by The Highway Testing Laboratory during these investigations and appreciation is herewith expressed.

DISCUSSION ON INVESTIGATION OF CONCRETE PAVEMENT PUMPING

MR. H. E. MARSHALL, OHIO DEPARTMENT OF HIGHWAYS: The author's careful observation and analysis of the performance of the two adjoining sections of new pavement on U.S. Route 52 east of Portsmouth, Ohio are particularly valuable since they represent a study of a project made by the same individuals who were responsible for its construction. The intimate knowledge of the project which is gained by the engineer in charge of its construction can rarely be entirely built up by the outside investigator who attempts the study of the performance of a project at some time more or less remote from its completion.

A number of very important conclusions have been drawn by the author from his observation of this project of which the most gratifying is the marked improvement in the condition of the pavement constructed on a granular subbase over that constructed on the raw soil subgrade. The author's principal conclusions appear to be entirely justified on the basis of observations made on these two projects; however, there are several points which he makes that are not borne out by experience on other projects throughout Ohio. These points are briefly discussed below.

A note on the plans for this project provided that "Excavated material of which the grain size of 50 per cent or more is between 0.05 and 0.005 mm. (silt) shall be placed at least 3 ft. below the pavement when used in embankment." This note has been used on the plans for a number of projects in Ohio where a considerable quantity of material high in silt was likely to be encountered in excavation. The objection to the presence of these soils closer to the pavement than about 3 ft. is that they are frequently elastic and rubbery due to the preponderance of the uniform sized silt particles and that they are very difficult to compact. Further these soils are very susceptible to frost heaving. The author proposes that the restriction on the soil used in the upper 3 ft. of embankment be extended to include all materials which contain more than 25 per cent silt. Study of Table 4 indicates that this restriction would exclude all materials similar to those taken from the soil subgrade on this project from the upper 3 ft. of embankment. Such a note would restrict the upper portion of all fills to either materials so granular or so high in clay as to have less than 25 per cent silt. This restriction in the writer's opinion would be neither justifiable nor desirable. Present day traffic does not warrant the use of selected predominantly granular material in depths as great as 3 ft. on the usual soil types encountered in Ohio and the use of non-granular soil so high in clay as to contain less than 25 per cent silt would afford a subbase no better and very probably somewhat worse than would be obtained if soils high in silt were used.

Referring now to the author's comments concerning the permissible quantity of fines in the classified embankment material, he states that "it would appear that the fineness limitation of the specification can be exceeded without causing harmful results. This statement should be further qualified by application only when bank run aggregate is used." The grading specification to which he refers is as follows:

Sieve Size	Per cent Passing		
	Grading 1	Grading 2	Grading 3
3 in.	100	100	
2 in.			100
1 in.	30-70	75-100	
½ in.		50- 90	
No. 10	0-25	25- 70	50-100
No. 200		0- 15	0- 15

Although the observations made on this project apparently justify the above general statement concerning the effect of soil fines slightly in excess of the 15 per cent limitation of the specification, this has not been found to be true throughout the State. It has been frequently observed that materials which contain the maximum or slightly in excess of the maximum permissible quantity of passing the No. 200 mesh sieve material are by no means as free draining as material for this course should be. In a number of instances free water has been observed ponded on top of these materials after they have been spread and compacted on the subgrade and prior to the placement of the pavement. Low permeability of the subbase is also indicated by the seepage of water from joints on the low sides of superelevated curves which the author reports on this project and has also been observed on a number of other projects where the classified embankment material was known to contain a considerable fraction of passing the No. 200 mesh soil fines. One of the most important functions of this type of subbase material is the speedy removal of water entering it either from the surface through joints and cracks or from the subgrade and the quantity of passing the No. 200 mesh soil fines has a very important effect on the permeability of the material. Recognizing this fact the Ohio Department of Highways in November of 1944 revised the specification for classified embankment material.[/1] The revision consisted principally of decreasing the quantity of fine sand and soil fines permitted in the course.

/1 - See Table 5, page 62 of this publication.

THE PUMPING OF CONCRETE PAVEMENTS IN NEW JERSEY, CORRECTIVE MEASURES EMPLOYED, AND FUTURE DESIGNS

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SYNOPSIS

Pumping at joints in concrete pavement slabs was first observed in New Jersey in 1930. It occurred on all pavement of standard design in which dowels 3/4-in. in diameter were used for load transfer. A 100 per cent increase in the number of dowels did not eliminate the trouble. The use of crushed stone drains along the edge of the pavement was only partially effective in stopping pumping.

In 1932 a test road was built over a silty-clay soil. One joint with no load transfer device, two with six 3/4-in. round dowels in the 10-foot width of pavement, two with twelve 3/4-in. dowels, and several with various combinations of heavy rectangular and channel type dowels were placed in the slab. Continuous applications of heavy loads under adverse moisture conditions indicated that the use of a load transfer device composed of 2-in. channel-dowels was necessary to prevent faulting and subsequent pumping.

A recent survey of 60,000 channel-dowel joints on heavy duty highways disclosed only three failures that were caused by pumping. No faulting was found at these joints and the failures had occurred by sagging of the pavement. The stone drains along the edge of the pavement were partially clogged with subgrade soil.

A study of pavements laid on sub-bases composed of granular materials lead to the conclusions that their use minimized pumping, reduced damage due to frost action and increased load bearing capacity. All pavements built since 1939 are supported on a layer of bank-run sand, gravel or cinders 8 inches in thickness. To date, where granular materials have been used in conjunction with channel-dowel joints, pavements have remained true to grade, cracks are few and far between, and there have been no indications of pumping, even under heavy truck traffic.

A study of wood for use in expansion joints shows that for most varieties, loads in excess of 500 and less than 1000 pounds per square inch will be required to cause compression of the fibers. If loading is continued, a point is reached where no further compression is obtained. Some varieties may be compressed to as much as 50 per cent of the original thickness. If dry wood is compressed to 50 per cent of its thickness, it will recover to about 65 per cent and remain at that thickness as long as it remains dry. Soaking in water will cause the wood to swell to 94 per cent, and for some varieties more than 100 per cent, of its original thickness. Repeated compression, drying and soaking will result in a permanent reduction in thickness. These tests indicate that wood as an expansion joint filler will have the following merits:

1. Unlike the conventional bituminous fillers, wood will not extrude, regardless of the extent of joint closure or infiltration. (This applies only to wood with the grain direction installed vertically).

2. Unlike other fillers, the wood is expected to retain sufficient swelling capacity and resiliency to prevent the detrimental accumulation and distribution of infiltrated material in the joint spaces which, in many locations, has caused rupturing of the concrete.

Due to the limited time permitted, the following material is perhaps not presented in strict accordance with the "Outline of Proposed Studies" agreed upon by the Committee. However, it is submitted at this time with the hope that a general description of pumping and faulting as observed in New Jersey, together with comments concerning the corrective measures employed, their effectiveness, and future designs, will further the work of the Committee. Because pumping and joint faulting have been so commonly associated, much of what follows necessarily concerns joints and their behavior. For reasons of continuity and completeness, it has been found necessary in frequent instances to include information already presented in a paper prepared by Mr. Harold W. Giffin, Engineer of Survey and Plans, New Jersey State Highway Department entitled "Transverse Joints in the Design of Heavy Duty Pavements". Mr. Giffin's paper which has been published in the 1943 Proceedings of the Highway Research Board is recommended as essential reading to those concerned with the design of concrete pavements.

Pumping and Faulting in New Jersey

Pumping and joint faulting first occurred in New Jersey sometime preceding the winter of 1930-31 on U.S. Route 1 and U.S. Route 130, the principal routes between New York and Philadelphia. By the spring of 1931 it had become severe on those routes and had extended to certain other routes that were carrying an increasing volume of heavy truck traffic. Its occurrence in New Jersey at that early date resulted from the rapid increase in heavy trucking in the immediately preceding years, especially in those locations such as New Jersey where the short distances between centers of large population favored the use of trucks. In consequence, and as a matter of sheer necessity, important revisions in design were made 12 years ago, and from time to time thereafter, for the specific purpose of preventing pumping and faulting.

Type of Pavement and Subgrade Involved

In 1931, practically all of the pavements that were pumping, and had faulted joints, had been constructed as follows:

Thickness: 9 in. uniform.
Slab length: Mostly 35 ft. Some variables to maximum of 68-2/3 ft.
Width: 10 ft. minimum - 14 1/2 ft. maximum.
Reinforcement: Single mat 2 in. below top surface. 3/8 in. longitudinal bars 7 1/2 in. c. to c.
Corner Reinforcement: 1/2 in. hairpin bars, all corners, 2 in. below top surface.
Load Transfer: Six 3/4 in. round dowels, 20 in. long, 3 at each end of joint 12 in. c. to c.
Joint Filler: 1/2 in. Thickness Premoulded Bituminous. Poured bitumen at top.

Practically all of these pavements had been laid directly upon whatever native subgrade soil was encountered. Except in very infrequent locations where the soil was found to be exceedingly unstable the subgrades had received no treatment other than shaping to grade and rolling. Inasmuch as subgrades varied from practically clean sand and gravel to silts and clays considerable variation in behavior resulted under identical conditions of pavement design and traffic. Many long stretches of pavement laid on inferior subgrades were carrying heavy truck traffic shortly after completion and in some locations serious pumping and faulting had developed after but three years of service.

Conditions Noted in 1931

Detailed investigations were made early in 1931 to determine the causes and extent of pumping and faulting. Conditions noted at that time appear to have been more or less typical of the pumping and faulting occurring in other parts of the country from time to time, especially recently as a result of heavy, wartime hauling. These conditions have been so adequately described by numerous observers in recent years that no general description appears necessary. However, some comments concerning certain conditions noted in 1931 may be appropriate.

1. The direction of the fault, step, or offset in the pavement surface was invariably such that as the wheels left one slab end ("leaving end") and crossed the joint space they dropped down onto the depressed slab end ("receiving end"). Faulting of as much as 3/4 in. was frequent.

2. The "leaving ends" were not depressed. (With but very few exceptions the "leaving ends" are still not depressed - have never been mud jacked even where immediately adjacent to "receiving ends" that have required mudjacking several times.)

3. Only minor pumping was occurring at the cracks. No cracks had faulted even in areas where joint pumping and faulting were severe. (In general, this condition still obtains today. Compared to the number of joints that have pumped, faulted, and required mudjacking, and repetitions of mudjacking, the number of pavement failures at cracks has been insignificant. Within recent years, however, some cracks have faulted. These instances will be discussed later.)

4. The water contributing to pumping was found to be almost entirely surface water that had infiltrated to the subgrade through open joints, and at points along the shoulder line. (This was substantiated by the fact that pavements on high fills and in cuts behaved more or less alike with respect to how soon they started to pump after rain began to fall, and with respect to the severity of pumping and faulting.)

5. The joint fillers or sealers did not exclude surface water. The independent movement of the slab ends had done much to destroy their sealing value, especially during cold weather when the bituminous sealing materials were brittle. It was noted that considerable quantities of water were swept into open transverse joint spaces by traffic.

In March, 1931, a section of the depressed end of a slab at a joint which had faulted 1 in. was removed for examination. The pavement was on a 2 foot fill. Heavy rains had fallen two days previously. The following note was made:

"The subgrade under the portion of the slab removed is clay - covered with a layer of water 1/8 in. to 1/4 in. deep over an area 5 ft. wide parallel to the joint. Apparently this subgrade is impervious to water. Unless pumped out, the water seems to have no means of escape other than by evaporation or slight seepage. A hole drilled through the pavement 14 ft. from the joint showed comparatively dry, firmly compacted subgrade." Soil sample No. 354253 shown in Table 2 represents the subgrade soil in this area.

The 3/4 in. round dowels were found very much out of parallel and the concrete around them was porous. The midsection of each dowel was bent for a length of about 4 in. in the shape of a flat "S", and the dowel holes had been enlarged by a crushing or wearing action of the dowels on the concrete for about 2 in. inward from the faces of the joint. These conditions are probably typical of most faulted joints installed prior to 1931.

In 1932, as a temporary measure, the number of 3/4 in. round dowels per joint was increased from six to twelve, and various devices were tried to maintain them in proper alinement. Periodic inspections made of these joints disclosed that the additional number of dowels probably retarded the rate of failure to some extent but that serious faulting developed nevertheless.

In addition to increasing the number of dowels, shallow crushed stone drains of various description were constructed immediately adjacent to the edge of the pavement in some locations, and a special effort was made to construct them in a manner most likely to carry off any free water that might tend to collect under the pavement. Drains of this type were installed in 1933 in some sections of U.S. Route 1, immediately adjacent to a heavy trucking lane laid in 1932 with joints having twelve 3/4 in. round dowels. Their effectiveness is difficult to evaluate, however, inasmuch as the joints adjacent to the drains are now faulted just about as much as those where the drains were omitted. There is at present an average faulting of about 1/4 in. - maximum 1/2 in. ± and there are some indications of pumping. Probably, due to progressive pumping of subgrade material into them, these drains have become less effective from year to year. The average daily truck traffic on this lane (1940 count) is about 1300, and many of the trucks are heavily laden truck-trailer units. The subgrade soil is essentially clayey and corresponds more or less to Sample No. 354,234. (Table 2).

Test Road

A circular concrete test road was constructed in 1932 on silty-clay soil, primarily to make an accelerated test of various joint types, and to further observe the process of pumping. Some of the joint types tested were:

One plain joint - no dowels - no load transfer - consisting simply of a 1/2 in. width space filled with premoulded bituminous filler.

Two joints with six 3/4 in. round dowels in each (same design as used prior to 1932), 1/2 in. premoulded bituminous filler.

Two joints with twelve 3/4 in. round dowels (same as temporarily adopted in 1932), 1/2 in. premoulded bituminous filler.

Several other joints having various combinations of heavy rectangular dowels, plus bearing angles for the dowels, plus sheet metal flashing.

Traffic consisted of one Mack truck pulling a loaded trailer at approximately 9 miles per hour. The maximum axle loads were:

Mack Truck		Trailer	
Front	- 5,500 lbs.	Front	- 13,400 lbs.
Rear	- 17,300 lbs.	Rear	- 32,800 lbs.

As concerns this test, the following remarks seem appropriate:

1. From August 1 to 18, during dry weather, the equipment completed approximately 1300 trips around the track at various speeds while numerous deflection readings were made. No visible or measurable deterioration of any kind was noted during this period except slight pumping at the plain joint during a shower on the afternoon of August 3.

2. During the morning of August 18 water was applied by means of sprinkling wagons, and in the afternoon and evening heavy rains fell. Shortly after the application of water the pavement at the plain joint started to pump noticeably and pumping to a lesser degree started at most of the other joints. By 8:00 P.M., after 840 trips that day, the plain joint had faulted 3/16 in. By midnight, 1120 trips, it had faulted 1/2 in. and a crack appeared $5\frac{1}{2}$ ft. from the depressed end. Running on the track ceased at midnight.

3. The following morning an examination disclosed no faulting at any other joints, nor any cracking, except at the plain joint as noted above.

4. On August 22, after 4270 trips since August 18, more cracks were evident adjacent to the plain joint which had become faulted $1\frac{1}{2}$ in. None of the joints with dowels had as yet faulted, and no cracking had developed in their vicinity. Sometime after the plain joint had faulted 1 in. or more the "leaving" slab end at this joint also became depressed, accompanied by cracking.

5. On August 23, after 5310 trips since August 18, cracks developed in the pavement adjacent to both of the joints with six 3/4 in. round dowels. No measurable faulting was evident. But due to the erosion of the subgrade and the formation of cracks near these joints, the pavement at the joints had sagged somewhat. Eventually these joints did fault slightly. On this date, no faulting or cracking was visible at any other joints that had more or heavier dowels.

It is necessary to mention here that the conditions of test were much more severe than normally occur in service. After August 18, the equipment was kept running continuously each day for 12 hrs. until the pavement at the plain joint became practically impassable. Prior to August 18, dikes had been constructed along the edges of the pavement to prevent the escape of surface water; in fact for a time the entire roadway was flooded and the slabs were more or less under water. The supposition was that those joints that best survived the ordeal would probably be most satisfactory in actual service. In consequence, there was a distinct difference in the behavior of the joimts in the test road as compared to joints in service. No joints, other than those with six 3/4 in. round dowels faulted measurably. The typical failure of the joints with various types of load transfer consisted of a pumping out of subgrade soil from under the pavement at the joints followed by a sagging of the pavement at the joints - both adjacent slab ends depressing the same amount. As the test progressed, the cracks also pumped violently - but none faulted. The absence of appreciable faulting, even at the joints with the weakest load transferring devices, appears to be explained by the flooded conditions, magnitude of load, and the lack of sufficient repetitions of load to permit the dowels to progressively crush, chew out, or wear away enough concrete to appreciably decrease their effectiveness. In short, the attempt to accelerate the faulting of the joints by the application of a limited number of heavy loads under extremely adverse subgrade conditions was, for the most part unsuccessful.

Although all of the joints with sturdy load transferring devices eventually pumped and sagged, to date, with but very few exceptions, no comparable behavior has been observed at similar types of joints that have been in service on heavy trucking routes for from 6 to 10 years. Considering the very rapid rate of faulting and pavement failure at the plain joint as compared to the joints with load transfer, this test at least indicated that load transfer at all points of interruption in the continuity of the pavement, such as at joints and cracks, is definitely beneficial.

Revisions in Design

The use of 3/4 in. round dowels was abandoned in 1933 and a much stronger joint structure was designed and adopted the same year. Except for occasional minor revisions, the joint so designed was used in conjunction with all concrete pavements constructed during the period 1933-1942. Load transfer was furnished by 2 in. channels, 20 in. long, an average of 12 in. apart. Bearing angles were provided above and below the dowels adjacent to the joint faces to prevent the dowels from crushing or wearing out the concrete at points of high bearing pressure. Sheet metal flashings were provided to exclude as much surface water as possible.

Effectiveness of Heavy Joint Design

More than 60,000 channel-dowel joints were installed during the period 1933-1942. Many are in service in pavement laid directly on soil susceptible to pumping which also carries extremely heavy truck traffic. To date, failure at these joints due to pumping has been negligible. A recent survey of the heavy trucking routes involving an examination of several thousand channel-dowel joints disclosed pavement failure due to pumping at only three joints. However, in these instances, no faulting had occurred. Instead, the pavement immediately at the joints had sagged to a maximum of 5/8 in. These joints are all in a heavy trucking lane (laid 1936) within a few hundred feet of each other in an area where subgrade and drainage conditions are particularly adverse. Also, in this area, appreciable sagging of the pavement was found at several cracks, and one crack had faulted ½ in. Pumping in varying degrees is general in this vicinity and, unless prevented, the pavement is expected to become progressively depressed at many other joints and cracks. The stone drains constructed immediately adjacent to this lane were found partially clogged by pumped-out subgrade material. The average daily truck traffic over these joints (1940 count) is 1050. The subgrade is represented by Samples No. 354,253 and 354,254. In considering the performance of these joints it is desirable to add that joints in an adjacent roadway constructed in 1928 with six 3/4 in. round dowels pumped badly and faulted as much as 1 in. within three years after construction, under less severe traffic conditions.

Some 20 miles south of this pumping area faulting of 1/4 in. and 7/16 in., respectively, was found at two channel-dowel joints in pavement constructed in 1934. However, faulting at these joints was not due primarily to structural failure as a result of heavy traffic. Instead, the effectiveness of the dowels was found to have been destroyed by the infiltration of practically incompressible silty, sandy material which, having accumulated in the lower portion of the joint spaces and displaced the filler, offered sufficient localized resistance to closure of the joints during pavement expansion as to rupture the concrete supporting the dowels. This having occurred, the possibility of infiltrated material eventually destroying the effectiveness of load-transferring mechanisms is a phase of joint design which merits serious consideration. The destructive effects of infiltration are discussed at length in Mr. Giffin's paper.

On other routes, no faulting or defects due to pumping are as yet apparent at the channel-dowel joints, even in lanes constructed directly on native, clayey soil which have been carrying in excess of 3000 trucks per day for more than six years. Many of these joints are immediately adjacent to 3/4 in. round dowel joints in adjoining older lanes which have faulted badly.

In general, the performance of the channel-dowel joints from the standpoint of preventing faulting and diminishing pumping has been satisfactory. Very likely, where truck traffic is heavy and subgrades are clayey, there is some slight pumping and, in consequence, it is expected that eventually the pavement at the joints will become depressed. On the other hand, except where infiltration destroys the effectiveness of the dowels, no faulting is anticipated.

In comparison with the unsatisfactory performance of weaker joints, the benefits derived from the use of these joints are beyond question. Actually, the pavement in the immediate vicinity of most channel-dowel joints in heavy duty pavements appears to be in as good if not better condition than elsewhere because of more apparent deterioration at intermediate cracks.

In view of this, it would seem that especially in those locations where the cost of granular or other suitable subbase material is prohibitive, and where successful treatment of the native soil is in doubt, the possibility of at least minimizing pumping and at the same time completely preventing faulting by the utilization of sturdy joint structures, properly installed, is certainly worthy of serious consideration.

Drains

Because the test road demonstrated that sturdy joints would only retard and not completely prevent pumping under extremely severe conditions, longitudinal crushed stone drains, constructed immediately adjacent to the pavement edge, were specified in most contracts during the period 1934-1939. The effectiveness of these drains is not known due to the indeterminate influence of the improved joint structures and because even where drains were not included pumping at present appears to be no more pronounced than where they were. The stone drains were expensive and their use was abandoned in favor of subbase material.

Subbase

In view of the generally recognized benefits to be derived from the use of granular subbase material with respect to minimizing or preventing pumping, reducing damage due to frost action, and increasing the load bearing capacity of the pavement, practically all pavements constructed since 1939 are supported on a layer of bank-run sand, gravel, or cinders, at least 8 in. thick. Insufficient time has elapsed to justify any predictions as to the effectiveness of these materials on a long-range basis. But to date, where granular subbase materials have been used in conjunction with channel-dowel joints, the pavement has remained practically true to grade, cracks are few and far between, and there have been no indications of pumping, even under the most severe conditions imposed by heavy trucking. Although the use of properly graded subbase material may prevent pumping there is considerable doubt as to whether, in the absence of adequate load transfer at joints and cracks, it will also prevent faulting. Some of the older pavements in New Jersey that have carried heavy truck traffic for years were laid on old macadam roads that were scarified, regraded, and rolled. Although the joints in these pavements lacked sturdy load-transferring mechanisms they have not faulted appreciably. But this type of construction is, of course, exceptional. Where, in one location, the pavement constructed in 1923 was laid on a layer of granular materials the joints in one area have faulted to a maximum of 9/16 in. Whether faulting was due to pumping, further compaction, or progressive rearrangement of the underlying material is not known. Subbase Sample No. 354,111 was removed from under a joint in this area which had faulted 9/16 in. Another section of the same job on a different kind of subbase has no faulted joints. The subbase in this section is represented by Sample No. 354,110. This pavement was laid on a regraded gravel road which was about one year old, and the subbase materials are presumably the original gravel surfacing, scarified, regraded, and rolled. This too is an exceptional type of construction.

Unfortunately, there are no known locations in New Jersey where a layer of subbase material as it is usually thought of was utilized in conjunction with weak joint structures on roads which have been carrying heavy truck traffic. Consequently, it cannot be stated definitely whether the use of subbase material without the assistance of adequate load transfer at joints and cracks will or will not prevent faulting. However, if the performance of pavements constructed on native granular soils is any criterion, the chances are that subbase alone is very likely to prove inadequate. In New Jersey, the degree of faulting on granular soils has been very variable. On one route, short stretches laid on native somewhat coarse, sandy soil did not fault, and perhaps did not pump, even under extremely heavy wartime hauling of sufficient intensity to cause complete failure in other areas laid on clayey soil. But faulting of more than 1/2 in. occurred in other sandy areas on the same route. The best that can be said at this time is that faulting has, and has not, occurred on granular soils, all depending upon the influence of many factors. So far as is known, the presence of pumping is not necessarily essential to the development of faulting if the underlying materials are susceptible to further compaction or rearrangement.

Present Views Concerning Subgrades and the Process of Pumping

Until very recently, no attempt has been made to classify pumping and non-pumping types of soil on the basis of detailed physical and chemical analysis. The practice, rather, has been to judge subgrade soils primarily in terms of their permeability and susceptibility to erosion. Wherever pumping has occurred in New Jersey the subgrade soil and the adjacent shoulders are more or less impervious. In addition, the subgrade soil is susceptible to erosion.

The process appears to take place essentially as follows: Surface water infiltrates to the subgrade through leaking joints, and at other points affording entrance, and collects in whatever vacancies may exist between the bottom of the pavement and the subgrade surface. Apparently very little water is absorbed by the subgrade. The only part of the subgrade that seems to combine with the water to any extent and become susceptible to pumping is confined to a thin film on the upper surface of the subgrade, 1/4 in. or possibly less in thickness. The thin film of water-saturated subgrade frequently has the consistency of thick paint. If the film is scraped off, apparently stable unaffected subgrade is exposed. With the agitation of the entrapped water some of the finer particles in the water-saturated film mix with the water and are carried away. As the particles are progressively carried off new surfaces of unaffected subgrade are exposed which in turn are softened and carried away.

There appears to be little evidence to indicate that, in general, pumping is the result of a softening of the subgrade for any appreciable depth or that large masses of subgrade are expelled at any one time. If a considerable depth of soft, water-saturated subgrade were involved failure would, it seems, occur much more rapidly than has been observed. Therefore, for the time being at least, it is assumed that any soil that becomes practically impervious when compacted but which, nevertheless, remains susceptible to erosion when in contact with violently agitated water is, in all probability, a pumping type of soil. The suitability of subbase materials is also judged in certain respects from the same point of view.

At the present time, numerous samples of soil from pumping and non-pumping areas are being analyzed and it is admitted that the results may modify the foregoing views.

Some years ago an effort was made to devise an erosion test, primarily to determine the most erosion-resistant types of subbase, but the results were erratic and inconclusive. However, this test did tend to substantiate the opinion that all subgrade materials that are essentially impervious are susceptible to erosion, unless the grains are adequately cemented together by some agent.

Comments on Subbase

If the use of subbase is indicated, the ideal type of construction might be thought to be slabs resting upon a layer of porous granular material which in turn rests upon a more or less impervious subgrade. Theoretically, the infiltrated surface water will drain through the porous subbase to the subgrade surface and thence, percolating through the subbase, flow along the surface of the subgrade to an outlet of some sort. Apparently the desirable functioning of this type of construction would indicate that the subbase material be as "open" as possible and that the subgrade surface be true to grade so as to avoid the retention of water in depressions. But there appear to be limitations as to how near this ideal of "open" subbase may be approached.

In 1932, for a distance of several miles on one heavy trucking route, an attempt was made to end the pumping and faulting problem once and for all by constructing the pavement on 9 in. of 2½ in. crushed stone. No fine materials were used in combimation with the stone to fill the voids. Not long after being in service the pavement settled considerably in many places. An examination disclosed that the crushed stone had been pressed into the clayey subgrade during wet or thawing weather and that the clay in turn had worked up into the voids in the stone. After 12 years of service, the pavement is in much poorer condition than adjacent sections laid on native, pumping-type soil.

Sometime later, tests were made to determine whether or not, under conditions of pressure, moisture, or softening, clayey or silty subgrades would work up into the voids of granular materials consisting of relatively small particles, such as sand or sand gravel mixtures. Attempts were made to combine adjacent layers of soft clay and coarse building sand by the application of pressure and agitation in the presence of free water—without success. The most severe test made was performed as follows: A cylindrical sheet metal container 14 in. in diameter, 8 in. high, was filled to a depth of 4 in. with silty-clay soil which had been mixed with water to a soft, almost sloppy, buttery consistency. Four inches of fairly coarse, washed, white sand was then carefully placed on top of the clay and covered with a metal disk 13-3/4 in. in diameter to which pressure could be applied by means of a lever system. The container was then placed completely under water and kept submerged for two weeks. During this period a sudden load of not less than 2000 lbs. was applied to the disk approximately 12,000 times. The person who operated the lever was instructed to make every effort to combine the two materials. These efforts were unsuccessful. At the conclusion of the test a perfectly clean line of separation was found between the materials. There were no indications whatsoever that the clay had entered the voids in the sand. In spite of having been submerged for two weeks the clay was much more stable than before. The pressure had squeezed considerable water out of the clay and stabilized it. Similar tests with crushed stone, and open gravel, on the same consistency of clay resulted in complete combination of the materials under a single application of load.

Efforts were made recently to combine adjacent layers of soft, sloppy, clayey soil and sand by repeated applications of a pressure of 30,000 lbs. per sq. ft. without success.

In view of the foregoing it is believed that no subbase material containing large voids should be used unless particular care is exercised to completely fill the voids with sandy material even though this results in an appreciable lowering in the rate of percolation.

If sand or similar subbases are used it appears probable that:

1. No matter how much water is present, it is unlikely that subbase and subgrade will combine because of pressure alone.

2. If the subgrade should become soft or sloppy, from whatever cause, but has no lateral means of escape, it may possibly be stabilized by heavy wheel loads squeezing some of the water out of the subgrade into the subbase.

3. Even if there is free water in the lower portions of the subbase layer, but no free water at the upper surface, no pumping will occur.

4. The subbase acts as a protective coating to the subgrade tending to keep it stabilized, and preventing its erosion even though there may be free water on the surface of the subgrade.

This probably accounts in large measure for the success of pavements laid on a layer of sand or well-graded porous, granular material.

Although bank-run sands and gravels are usually thought to be more or less permeable numerous tests have shown that many of the bank-run materials available from many sources in New Jersey may be compacted into a practically impervious mass. Usually the impermeability of these materials is due to the presence of a very small percentage of clay or materials removable by elutriation. Future studies may indicate the desirability of processing most subbase materials.

At present very little is known as to the effect of the subbase materials used. It is very likely that their effectiveness is susceptible to considerable variation inasmuch as they vary from practically clean sand to bank-run gravel containing a fairly high percentage of clay. Some of the bank-run gravels are capable of being compacted into a very firm mass and their use perhaps increases the bearing capacity of the pavement considerably more than sand. On the other hand, material of that kind may be open to the objection that since it is likely to be impervious it may permit the accumulation of free water immediately under the pavement and thus eventually lead to failure by erosion. With this thought in mind, perhaps the most suitable subbase material may not necessarily be that which results in the greatest increase in bearing capacity but rather a material that has, in addition to sufficient bearing value, just enough porosity to prevent the accumulation of free water. The determination of the most suitable subbase materials to be used in conjunction with the construction of concrete pavements is regarded as one of the most important subjects for future study.

Wood Joint Filler

When the causes of pumping and faulting were investigated in 1931 it became apparent that the conventional bituminous fillers were unsatisfactory, in fact it was concluded that because of certain fundamental characteristics bituminous fillers could not satisfactorily exclude water and foreign material. Since then considerable study has been devoted to other materials and methods of sealing joints. Having as an objective the discovery or development of a filler that would fill the entire joint space at all times tests were made several years ago on various kinds of wood to determime the extent of recovery after compression. It was found that most woods after being compressed considerably will remain in a state of compression indefinitely, if kept dry, but that if soaked in water will swell a great deal. The possibility of using precompressed wood as a joint filler became apparent and, in consequence, many tests have been made, and are still in progress, to determine its behavior over a period of years. In addition, a few joints in service have been filled with precompressed wood.

The primary purpose of these tests has been to determine how long various kinds of wood retain their capacity to swell, under what conditions they might lose that capacity, and at what rate.

Since the efficacy of joint filling materials must of necessity be judged on the basis of long-range performance, and since there appears to be no reliable way to accelerate tests of this kind, nor even to subject the materials in the laboratory to conditions comparable to actual service, much still remains unanswered. However, information concerning the manner of testing and the behavior of the test specimens and installations to date may at least serve to stimulate interest in this material.

General Behavior of Wood

Most woods resist appreciable compression at pressures less than 500 lbs. per sq. in. At pressures somewhat greater, but usually less than 1000 lbs. per sq. in., depending upon the kind of wood and its grain structure, the elastic limit is exceeded and appreciable compression occurs with little additional pressure. For example, the general behavior of white pine is that its thickness is decreased only 5 per cent by a pressure of 700 lbs. per sq. in. but decreased 25 per cent at 1000 lbs. per sq. in. and 50 per cent at 1600 lbs. per sq. in. (All percentages are in terms of the initial uncompressed thickness of the wood.) If compression is continued, so-called "hard-bottom" is reached eventually at which point little additional compression occurs even under very high load. The amount of compression that occurs before hard-bottom is reached varies considerably with different kinds of woods. In general, the lower the specific gravity of the wood the more it may be compressed. White Cedar, for example, reached hard-bottom after being compressed about 65 per cent whereas Cypress may reach the same point at 50 per cent. (These percentages vary considerably for even the same kinds of wood, depending primarily upon the specific gravity and the grain direction.) Water-soaked wood is somewhat easier to compress.

In general, if dry wood is compressed to 50 per cent of its initial thickness, in a device that prevents transverse spreading, it will recover to about 65 per cent and remain practically at that thickness indefinitely, if kept dry. Humid atmosphere does not cause appreciable further recovery. The first time it is soaked in water it will swell to within at least 94 per cent of its initial thickness, and some woods swell to more than 100 per cent. If recompressed in a clamp to 50 per cent, and kept clamped at 50 per cent until dry, it will shrink to about 48 per cent - that is, it will eventually become loose in the clamp. If again soaked, it will not swell quite as much as it did the first time. Repetitions of clamping at 50 per cent, drying while clamped, followed by soaking, cause a further reduction in "swelled thickness". This progressive loss has been termed "compression-shrinkage". The rate of loss is not constant, however, inasmuch as it diminishes as the process is repeated, until, at normal temperatures, a point is reached where the "swelled thickness" remains more or less stable. The conditions under which the wood is kept clamped have a marked effect upon the rate of loss. Oven-drying causes the highest rate of loss, and the lowest rate occurs if the wood is kept wet at all times. Loss due to air-drying at normal temperatures is intermediate between oven-drying and constant soaking.

Most of the tests were made as described in the foregoing, and some of the results are shown in Table 1. For example, Cypress Specimen 165D was compressed to 50 per cent of its initial thickness on September 1, 1942 (3220 lbs. per sq. in. pressure). It recovered to 63.3 per cent, and had swelled to 94.7 per cent during soaking for 24 hours. After one cycle of being clamped at 50 per cent and oven-dried it swelled to 87 per cent, and after 5 cycles it swelled to 81 per cent. As will be noted, its least "swelled to" thickness was 65.3 per cent on May 1, 1943, after 71 cycles. After May 1, 1943, oven-drying was discontinued and the specimen was air-dried. From then on the "swelled to" thickness increased somewhat and appears now to have become stabilized at about 70 per cent. During the 27 additional cycles between August 1, 1943 and August 1, 1944 no further loss in swelled thickness occurred. Cypress Specimen 165W (part of the same piece of lumber as 165D) has been kept under water continually and, as may be noted, its "swelled to" thickness exceeds that of Specimen 165D. Both of these specimens, and those that are described in the following, have lost an indefinite amount of thickness due to some progressive transverse spreading in the clamps.

The relative behavior of various kinds of wood is indicated by Specimens No. 272, 273, 274, and 275. All of these specimens were subjected to identical treatment. During the first 12 cycles they were oven-dried at 100°F, air-dried thereafter. As indicated, Redwood suffered the greatest loss.

It is desirable to mention here that the "swelling range" of the wood is considered to be the difference between the thickness shown in the "Swelled to" columns and the "Recovered to" column. It is assumed that the wood will be installed at its "Recovered to" thickness and that consequently the greater the difference between its "Recovered to" thickness and its "Swelled to" thickness the more likely it will be to always fill the joint space. Therefore, while the "Swelled to" thickness of White Pine Specimen No. 275 is 86.5 per cent as compared to 77 per cent for Cypress Specimen No. 272 its actual swelling range, due to its greater "recovered thickness", is only 1.8 per cent greater.

Other specimens are being tested to determine the loss in swelling capacity due to being kept in a protracted state of compression such as would likely occur in service during the summer when the wood may be under considerable compression for several months. These specimens are Nos. 172B, 173, and 174B. They have been stored in damp sand continually, clamped at one-half their initial thickness, since September 1942 except that every three months they are released and soaked for 24 hours, measured, reclamped, and returned to storage. As will be noted, these specimens now swell less than any others. Whether or not they will eventually fail to swell at all is not known. It seems problematical as to whether these protracted compression tests are indicative of what may be expected in service inasmuch as daily as well as seasonal variations in joint width will provide some relief, and may tend to keep the wood "alive", so to speak, whereas long-continued compression at a constant dimension may cause the wood to acquire a so-called "set".

Cypress Specimen 172A has been kept clamped at one-half thickness and stored continually out-of-doors exposed to sun, rain, freezing, and thawing. Every three months it is released and its swelling recorded.

Due to restrictions imposed by the war only a few joints in actual service have been filled with precompressed wood. These installations were made more than two years ago and the performance to date has been satisfactory. During dry weather some shrinkage apparently occurs in the wood near the pavement surface permitting the infiltration of fine material to a depth of one or two inches. Below this depth, however, the wood appears to remain damp and in a swelled condition, and has so far been found to completely fill the joint space at all times.

Until the merits of precompressed wood are more definitely determined, and until the manufacturing process is worked out, the intention is to use uncompressed, clear, heartwood Cypress specially fabricated so that the grain direction of the main body of the wood is vertical in the joint space. Because this ordinary wood is not expected to completely fill the joint space in the winter time an extra strip has been attached to the bottom to exclude as much surface water as possible. As mentioned in Mr. Giffin's paper, it is supposed that infiltrated material will progressively accumulate in whatever vacancies occur and eventually compress the wood causing it to function in time somewhat like precompressed wood.

Two of the important merits of wood joint filler appear to be:

1. Unlike the conventional bituminous fillers, wood will not extrude, regardless of the extent of joint closure or infiltration. This applies only to wood with the grain direction installed vertically and only if restraining means such as dowels are provided to avoid lateral extrusion at the edges of the pavement.

2. Unlike other fillers, the wood is expected to retain sufficient swelling capacity and resiliency to prevent the detrimental accumulation and distribution of infiltrated material in the joint spaces which, in many locations, has caused rupturing of the concrete.

Recent Widening and Faulting of Cracks

As mentioned previously, no faulted cracks were found in 1931, in fact not until very recently has the faulting of cracks been of any consequence in New Jersey. Early in 1945, however, investigations were started to determine the cause of wide cracks which were recently observed to be increasing in number in a few locations in New Jersey. Due to a lack of time and sufficient personnel these investigations are not as complete as desired and, of necessity, the causes in certain instances are still not conclusive. However, the findings to date are believed to be of sufficient importance to be brought to the attention of the Committee, and to merit the attention of all others who are concerned with the design of joints, for the following reasons:

Number	Species	Manner of Testing	Compression Data: Uncomp Thick	Spec Grav	Date	Compressed To: Meas.	%	Load P.S.I.	Recovered To: Meas	%	Swelled To: Meas.	%	Date First Clamped	Clamped Thickness: Meas	%	1 Cycle Swell: Meas	%	5 Cycle Swell: Meas	%	Nov. 1, 1942: No of Cycles	Swelled to: Meas	%	Feb 1, 1943: No of Cycles	Swelled to: Meas	%
165 D	Cypress	Wet & Dry Cycles	1.50"		Sept 1,1942	0.75"	50	3220	0.95"	63.3	1.42"	94.7	Sept 3,1942	0.75"	50	1.30	87	1.22	81	23	1.03"	68.7	50	0.99"	66.0
165 W	Cypress	Wet Cycles	1.50"		Sept 1,1942	0.75"	50	3220	0.95"	63.3	1.42"	94.7	Sept 9,1942	0.75"	50	1.38	92	1.30	87	23	1.26"	84.0	49	1.20"	80.0
172 A	Cypress	Weathering	1.50"		Sept 1,1942	0.75"	50	2740	0.95"	63.3			Sept.28 1942	0.75"	50					1	1.34"	89.4	2	1.25"	83.3
172 B	Cypress	Damp Sand	1.50"		Sept 1,1942	0.75"	50	2740	0.95"	63.3			Sept 28,1942	0.75"	50					1	1.31"	87.3	2	1.22"	81.4
173	Douglas Fir	Damp Sand	1.50"		Sept 1,1942	0.75"	50	3720	0.91"	60.7			Sept 28,1942	0.75"	50					1	1.19"	79.4	2	1.14"	76.0
174 B	White Cedar	Damp Sand	1.50"		Sept.28,1942	0.750"	50	1300	0.95"	63.3			Sept 28,1942	0.75"	50					1	1.11"	74.0	2	1.00"	66.7
272	Cypress (Heartwood)	Wet & Dry Cycles	1.47"	.595	Sept.30,1942	0.735"	50	5780	0.952"	64.6	1.43"	97.0	Oct 4,1943	0.75"	50.8	1.25	85								
273	Douglas Fir	Wet & Dry Cycles	1.49"	.566	Sept 30,1942	0.744"	49.7	4880	0.942"	63.4	1.46"	98.0	Oct 4,1943	0.75"	50.5	1.28	86								
274	Redwood	Wet & Dry Cycles	1.49"	.405	Sept.30,1942	0.745"	50	1846	0.931"	62.4	1.40"	94.0	Oct 4,1943	0.75"	50.2	1.07	72								
275	White Pine	Wet & Dry Cycles	1.48"	.413	Sept 30,1942	0.740"	50	1580	1.07"	72.3	1.52"	103.0	Oct 4, 1943	0.75"	50.7	1.37	93								

Number	Species	Manner of Testing	May 1, 1943: No of Cycles	Swelled to: Meas	%	Aug 1, 1943: No. of Cycles	Swelled to: Meas.	%	Nov. 1, 1943: No. of Cycles	Swelled to: Meas	%	Feb 1, 1944: No. of Cycles	Swelled to: Meas	%	Mar 1, 1944: No of Cycles	Swelled to: Meas.	%	May 1, 1944: No of Cycles	Swelled to: Meas	%	Aug 1, 1944: No. of Cycles	Swelled to: Meas	%
165 D	Cypress	Wet & Dry Cycles	71	0.98"	65.3	77	1.08"	72.0	85	1.06"	71.6	94	1.06"	71.6	96	1.05	70.0	101	1.05"	70.0	104	1.09"	73.0
165 W	Cypress	Wet Cycles	76	1.19"	79.4	86	1.19"	79.4	95	1.14"	76.0	105	1.16"	77.3	107	1.15	76.8	113	1.17"	78.0	118	1.15"	77.0
172 A	Cypress	Weathering	3	1.25"	83.3	4	1.19"	79.7	5	1.07"	71.7	6	1.09"	73.0				7	1.12"	75.0	8	1.15"	77.0
172 B	Cypress	Damp Sand	3	1.15"	76.6	4	1.14	76.0	5	1.04"	69.4	6	1.02"	68.0				7	1.02"	68.3	8	0.99"	66.0
173	Douglas Fir	Damp Sand	3	0.98"	65.4	4	0.97	64.7	5	0.88"	58.6	6	0.87"	58.0				7	0.88"	58.5	8	0.87"	58.0
174 B	White Cedar	Damp Sand	3	0.95"	63.4	4	0.96	64.0	5	0.88"	58.6	6	0.88"	58.6				7	0.89"	59.0	8	0.88"	58.5
272	Cypress (Heartwood)	Wet & Dry Cycles							6	1.13"	76.6	15	1.08"	73.4	17	1.08	73.4	22	1.08"	73.7	23	1.13"	77.0
273	Douglas Fir	Wet & Dry Cycles							6	1.15"	77.2	15	1.09"	73.2	17	1.10	74.0	22	1.09"	73.0	23	1.13"	76.0
274	Redwood	Wet & Dry Cycles							6	0.99"	66.3	15	0.97"	64.6	17	0.97	64.6	22	0.96"	64.5	23	1.00"	67.0
275	White Pine	Wet & Dry Cycles							6	1.29"	87.1	15	1.21"	81.7	17	1.21	81.7	22	1.20"	81.0	23	1.28"	86.5

Note – Percentages shown are per cent of Uncompressed Thickness

TABLE 1. PRECOMPRESSED WOOD TESTS

1. Highway engineers in general should be informed as to the conditions under which these wide cracks occurred in order that adequate provisions may be made to avoid their occurrence in future work.

2. In an effort to prevent the faulting of joints in future work it appears likely that many engineers may be planning to install larger dowels, or a greater number of dowels, without giving due recognition to the influence of certain important factors (notably the effects of rusting of the dowels or other metallic load-transferring mechanisms) which if ignored may soon lead to the development of serious defects.

The following is not to be regarded as an exhaustive report on the causes of wide cracks but, instead, primarily as a word of caution to those whose responsibility it may be to evaluate the merits of various types of joints and to decide upon which shall be used.

Apparently most of the cracks which have recently widened in heavy duty pavements in New Jersey occurred during early life, due to differential frost-heaving or subgrade settlement. For some years the reinforcing steel evidently was capable of maintaining the cracks at hair-line width. In recent years, however, some of these cracks have widened considerably, present width averages about 3/8 in. — maximum width 7/8 in. Where there is heavy trucking the wider cracks have faulted, some as much as 5/8 in. In practically all instances where the crack width is 1/4 in. or more the longitudinal reinforcing steel has broken.

These conditions were noted in the fall of 1944 on U.S. Route 22 in the vicinity of North Plainfield. This route carries heavy truck traffic. In this location 9 faulted cracks, ranging from 1/4 in. to 7/8 in. in width, were found in a stretch of pavement 2000 ft. long. All of these cracks are located within the middle third portion of the slabs. No cracks of any width were found at or within less than 9 ft. from the transverse joints. At all of these 9 cracks the reinforcing steel had failed. The pavement was constructed during the summer of 1938, as follows:

Thickness: 10 in. Uniform. Width of Slab: 10 ft. Length of Slab: 53 ft. Longitudinal Reinforcement: Sixteen 3/8 in. Diameter bars. Dummy or Contraction Joints: None. Subgrade: Essentially silty clay — no subbase. Load transfer at Expansion Joints: Twelve 2 in. channel dowels, (7/8 in. flange and 1/2 in. web). Prior to installation the dowels were given two coats of paint — white lead followed by red lead. Immediately prior to embedment in the concrete the dowels were given an additional coating of mineral oil. The coatings were applied the entire length of the dowel.

These particular coatings were used because comprehensive tests made in 1935 indicated that the bituminous coatings previously used on 3/4 in. round and 2 in. channel dowels did not prevent excessive resistance to sliding. These tests indicated that the white lead, red lead, and oil coatings, if applied in accordance with the specifications, would facilitate free sliding of a 2 in. channel dowel (10 in. embedment) at considerably less than 1000 lbs. These coatings were applied primarily to facilitate free sliding during early life inasmuch as it was expected that if freedom of slippage had been established initially the subsequent opening and closing of the joints would cause a continuing decrease in sliding resistance. In the light of present knowledge, these expectations were erroneous.

Inasmuch as measurements of the periodic variations in width at various types of experimental joints installed twelve years ago in another route indicated that for some unknown reason the sliding resistance of dowels might possibly increase materially in time it was decided to install a series of measuring plugs adjacent to all joints and cracks in this 2000-ft. stretch of pavement to determine whether the joints were opening and closing normally, and to measure the variations in width occurring at the cracks. Measurements taken immediately before and after a considerable drop in pavement temperature indicated that practically none of the joints in this location opened to any extent and that contraction of the pavement was accounted for principally by an appreciable opening of the cracks at which the reinforcing steel had failed. As a result of these measurements, and on the basis of an assumed subgrade resistance, it was estimated that the sliding resistance per dowel was at least 3000 lbs. This was later confirmed by removing the concrete on one side of a so-called "frozen" joint and, with a special dowel-pulling device, ascertaining the force required to cause slippage. The sliding resistance of 5 dowels was determined. The average resistance was found to be approximately 18,000 lbs. per dowel. Of these five, the dowel which offered the greatest resistance remained practically immovable at 15,000 lbs. When subjected to a constant pull of 24,500 lbs., it slid at the rate of .0075 in. in 7 minutes. More or less "free" sliding occurred at 25,000 lbs.

It is evident from the foregoing that, contrary to what might be supposed, the sliding resistance of these dowels has increased very materially within the past 6½ years.

Three of these dowels were entirely withdrawn from the concrete for detailed inspection. Considerable rusting of the dowel surfaces was found, especially for several inches each side of the joint space. (See Figure 1). The remaining 6 in. or so in the vicinity of the ends of the dowels was comparatively free from rust although some rust was found along the entire bottom surface of the channels and along the bottom surface of the top flange. The dowels were surrounded by very dense, well consolidated concrete. The indications are that moisture entered at the joint faces and progressed inwardly for several inches. As the dowels were withdrawn, pieces of concrete at the joint face which evidently adhered to the rusted portions of the dowels were broken out.

In so far as this particular joint is concerned there appears to be no doubt that corrosion of the dowel metal is the primary cause of the resistance to sliding. And there appears to be no reason to doubt that corrosion has similarly affected the rest of the joints in this and many other locations. That corrosion is the primary cause of restraint appears to be substantiated by (a) appreciable rusting and pitting of the dowel surfaces and (b) the fact that as soon as the pulling had advanced to the point where the rusted portion of the dowels had been withdrawn from the concrete the force required to continue sliding was considerably less.

So far as is known, corrosion causes restraint to sliding for the following reasons:

1. When steel corrodes the volume of the products of corrosion is several times the volume of the metal destroyed.

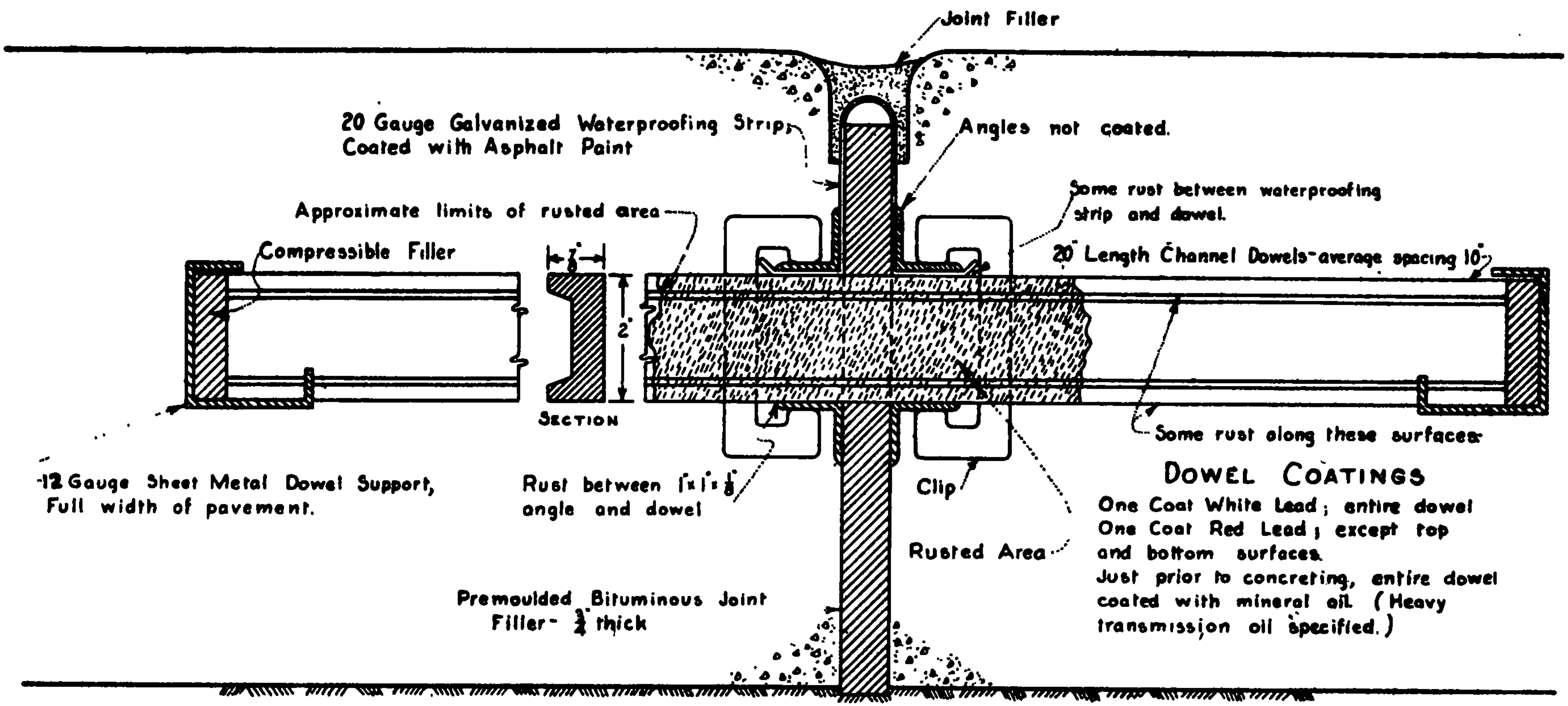


FIGURE 1. SECTION THROUGH CHANNEL-DOWEL JOINT SHOWING RUSTED AREAS.

SCALE: 1/2" = 1"

2. If corrosion occurs in a state of confinement the products of corrosion exert a considerable expansive effort which, under certain conditions, may be as much as 2000 lbs., per sq. in.

3. If corrosion occurs in immediate association with concrete an exceptionally voluminous form of rust may occur.

(The foregoing information regarding the behavior of the corrosion products is according to Ulick R. Evans, M.A., Sc. D., in his comprehensive treatise "Metallic Corrosion Passivity and Protection", Edward Arnold & Co., London.)

It appears, therefore, that as the rust forms its tendency to expand imposes a progressively increasing squeezing or gripping action on the dowels which gradually builds up sliding restraint. Apparently sliding of the dowels back and forth in the concrete as normally occurs initially is insufficient to counteract the gripping tendency induced by corrosion.

Evidently the magnitude of the restraint is a function of the area of the corroded surface and that for equal percentages of corrosion the larger the surface area of the dowel the greater the magnitude of the restraint. This is not to imply, however, that corrosion is of no consequence in the case of small dowels. As will be brought out later, the first wide, faulted cracks to be observed in New Jersey occurred two or three years ago on U.S. Route 1 in pavements in which joints with twelve 3/4 in. round dowels were installed. However, this has not as yet been definitely determined to be due to the same cause.

In any event, there is no question but that any material restraint to dowel sliding, from whatever cause, will sooner or later prove detrimental, especially to those pavements which carry heavy truck traffic, mainly for the following reasons:

1. During periods when the pavement temperature is decreasing the sliding resistance of the dowels induces tension in the pavement by tending to prevent its contraction. The tension thus induced is additive to whatever tension may be present due to other causes.

2. The tension induced by the sliding resistance of the dowels imposes an additional strain on the reinforcing steel spanning transverse cracks and thus tends to elongate the steel and widen the cracks.

Where cracks have recently widened in heavy duty pavements excessive tension was apparently responsible for starting the process of widening. As soon as a small amount of widening had taken place, however, the influence of heavy trucking appears to have accelerated the process. As mentioned previously, many of these cracks apparently occurred during the early life of the pavement due to causes other than contraction. Evidently for some years, during which time dowel restraint was perhaps more or less negligible, they remained at practically hair-line width. In later years, however, as dowel corrosion progressed and sliding resistance increased, the additional tension caused some further widening. So long as the reinforcing steel maintained the cracks at hair-line width it is probable that efficient load transfer was effected through aggregate interlock. With further widening, however, the effectiveness of the interlocking action was correspondingly decreased and some independent vertical movement of the abutting pavement sections at the cracks became possible. When sufficient independence of movement had thus been permitted the reforcing steel became subject to some flexing with each passage of load. As a result of thousands of flexings under heavy traffic within a relatively short time the steel became fatigued and failed, apparently while the cracks were still quite narrow. The cracks probably did not exceed 1/8 in. in width at the time the steel failed. After the steel failed, however, the rate of widening probably increased.

That the process of crack widening on Route 22 took place as described appears to be substantiated by the following observations:

1. No definite necking down of the fractured reinforcing steel was found. What appears to be necking down, however, has been noted at wide cracks in some very light traffic pavements in which 2 in. channel dowels were installed; apparently, in these instances, due to direct tension resulting from excessive dowel restraint.

2. In some of the cracks which are badly ravelled in the vicinity of the pavement surface, pieces of broken longitudinal reinforcing steel about 1¼ in. long have been found in the broken up concrete in the crack space.

It appears, therefore, that in so far as heavy duty pavements are concerned there is a critical crack width which if exceeded will permit excessive independent vertical movement of the abutting pavement sections. If the independence of movement is excessive in conjunction with frequent repetitions of load the reinforcing steel may soon fail due to bending fatigue while the cracks are apparently of harmless width, even though the ultimate strength of the steel in direct tension may have at no time been approached. The minimum amount of steel necessary to prevent cracks from widening in excess of a critical maximum under any given conditions, even with complete absence of restraint to joint opening, is as yet uncertain. But inasmuch as the faulting of cracks appears to be by no means infrequent in other locations as well as in New Jersey the relationship of critical crack width to traffic intensity is a phase of design which evidently merits further study.

As mentioned previously, a number of wide, faulted cracks have occurred in a section of U.S. Route 1. This section is between Trenton and Penns Neck and was constructed in 1932, as follows:

Thickness: 9 in. Uniform. Width of Slab: 10 ft. 6 in. Length of Slab: Approximately 69 ft. Longitudinal Reinforcement: Sixteen 3/8 in. Diameter bars. Dummy or Contraction Joints: None. Subgrade: Essentially silty clay — no subbase. Load Transfer at Expansion Joints: Twelve 3/4 in. round dowels, 20 in. long, sliding ends coated with cut-back tar, grade U.C., or K.P., 4 in. or 6 in. length cardboard sleeves with compression caps at ends.

Wide, faulted cracks in this pavement were first noted about two years ago and they have become more numerous since then. Most of the wide cracks are located approximately midway between joints, but a few are within 12 to 15 ft. from the joints. At the present time it is not possible to make any statement as to the cause of these wide cracks, for the following reasons:

1. None of the dowels has as yet been removed to determine whether there is any corrosion present.

2. Practically all of the joints have faulted from 1/4 to 1/2 in. and it appears entirely possible that even though corrosion may exist its effects at present may be substantially counteracted by the faulting. With faulting there is inevitably some bending of the dowels and enlargement of the dowel holes in the vicinity of the joint space. Furthermore, at faulted joints, independent slab

action under passage of load probably induces a small amount of dowel sliding, and the whole process of faulting involves a general loosening of the dowels. Whether or not there was a time prior to the faulting of the joints that the dowels, due to corrosion, offered appreciable restraint to sliding and thus induced the initial widening of the cracks, and heavy truck traffic then completed the process, is not known. But whatever the cause, this instance is mentioned simply to point out that wide cracks have occurred in conjunction with 3/4 in. round dowels as well as in conjunction with 2 in. channel dowels and that corrosion might have been influential in this instance as well.

It appears desirable to add that an indication of the possibility of a progressive increase in dowel sliding resistance was found 2½ years ago. In 1932, for experimental purposes, various types of joints were installed in series in some sections of a widening lane then under construction on U.S. Route 1. The dowels in some of these joints are 3 in. I-Beams (4, and in some cases 2, per joint, average length 21 in.) painted with cut-back tar. During construction Monel metal gauge plugs were installed each side of all the joints in the experimental sections in order that subsequent variations in joint width might be accurately determined. In 1942 it was noted that most of the joints with four I-Beams had considerably decreased in width during the preceding 10 years whereas those joints that had no dowels or mechanisms to restrain movement had compensated for this decrease by opening a corresponding amount. This is also true, but to a lesser degree, in the case of some of the joints with two I-Beams. Measurements taken in December, 1932 indicate that all of the I-Beam joints opened more or less normally (slightly less than completely unrestrained joints) during the first winter following construction. Recent measurements indicate, however, that the maximum annual variation in width of a number of these I-Beam joints is less than .03 in. whereas the immediately adjacent unrestrained joints have an average annual variation of about .45 in.

When this was first observed it was concluded that for some unknown reason the sliding resistance had progressively increased during the preceding ten years — but the cause was only surmised. However, the possibility that restraint might be due to corrosion of the dowels was suspected, but not investigated — in fact no investigations have as yet been made to definitely determine why no slippage takes place at these I-Beam joints.

Prior to recently determining that corrosion quite definitely can be the cause of excessive dowel restraint it was the intention to specify 1 3/4 in. x 1 in. x 20 in. length hot-rolled steel dowels spaced 12 in. c. to c., in future work and to facilitate their free sliding by greasing and encasing the sliding ends in sheet metal sleeves. However, in the light of recent investigations the efficacy of these measures in permanently facilitating free sliding is indeed questionable. Sleeves, due to some looseness of fit, might actually permit freer access of moisture to the dowel surfaces than coatings. There are also serious doubts, considering past experiences, as to whether coatings of any kind such as oils, greases, paints, bituminous compounds, rust-inhibiting preparations, synthetic resin finishes, galvanizing, cadmium plating, or any purely surface treatment can be relied upon to provide any more than temporary protection against corrosion. Without any question whatever, white lead, red lead, and oil coatings applied to ordinary hot-rolled steel dowels cannot be depended upon to prevent corrosion and eventual excessive restraint. This may hold true for any paint or dip coat.

For the foregoing reasons consideration is being given to the possible use of dowels made of materials that are corrosion-resistant. At present intensive studies are in progress to determine the relative corrosion-resistance of various kinds of metals, including several grades of stainless steel, under conditions tending to promote rapid corrosion. Inasmuch as metals that are corrosion-resistant are generally more expensive than ordinary steel studies are being made to determine the minimum dowel size, in terms of weight of material utilized, that tests indicate will meet requirements. Heretofore there has been some hesitancy about using structural shapes such as small I-Beams, mainly because of the possibility that their relatively thin webs or flanges might rust out completely within a few years. If, however, the dowel material is such that it may be relied upon to resist corrosion indefinitely the dowels can probably be of the most efficient shape such as to provide maximum strength, in conjunction with sufficient area of bearing surface, per pound of metal utilized. Some further decrease in the size of section may also be possible if the dowel material has a higher elastic limit than materials heretofore employed.

Present Views on Faulting, and Future Joint Design

In terms of past experience in New Jersey it appears essential that some substantial form of connection be provided at all points of interruption in the continuity of the pavement in order to counteract the persistent tendency of heavy truck traffic to cause faulting. At cracks this is presumably accomplished by the interlocking of the irregular surfaces of fracture, or so-called aggregate interlock, provided the amount of longitudinal reinforcing steel is sufficient to maintain the cracks at hair-line width. At joints various kinds of connections or means to counteract faulting are of course available. But, in so far as New Jersey is concerned, the employment of a series of substantial dowels has proved to be the most direct, dependable means of accomplishing this purpose. In view of this, and notwithstanding the difficulties that have recently become apparent in some locations due to dowel corrosion, it is the present intention to continue the utilization of substantial dowels. This intention is believed to be justified for the following reasons:

1. Three-fourths inch round dowels spaced 10 in. apart have proved to be incapable of preventing serious joint faulting in pavements carrying heavy truck traffic. The faulting of joints with 3/4 in. round dowels is due primarily to the fact that these dowels are deficient in stiffness. This deficiency in stiffness permits the development of excessive bearing pressures of the dowels on the concrete in the immediate vicinity of the joint space. These excessive pressures repeated hundreds of thousands and even millions of times in the course of a few years on heavy trucking routes cause a progressive crushing or wearing away of the concrete supporting the midsection of the dowels. This action necessarily enlarges the dowel holes for some distance each side of the joint space and soon renders the dowels non-effective.

2. In the case of dowels with greater bearing area and stiffness there is a correspondingly lesser tendency for the bearing capacity of the concrete to be exceeded. Increased bearing area and increased stiffness each contribute to a lessening of the unit pressures on the concrete. Tests now in progress have as one objective the determination of the most efficient dowel section and size, in terms of bearing area and bending resistance, consistent with the holding capacity of the concrete.

3. The more efficiently the load-transferring device prevents independent vertical movement of the adjacent slab ends the lower will be the magnitude of the forces and actions tending to cause pumping and faulting.

4. Notwithstanding the recognized value of granular subbase materials, and procedures having as an objective the stabilization of subgrades, the construction of subgrades and subbases has not yet reached the degree of dependability necessary to guarantee the prevention of faulting at joints which lack substantial load-transferring devices. Considering the numerous complex factors that tend to promote faulting, the magnitude of the forces involved, and the degree of stability it is necessary to provide, it appears to be uneconomical, and less dependable, to endeavor to counteract faulting by omitting load-transferring devices and resorting to special methods of subgrade stabilization alone. Instead, in the interests of economy and dependability, it appears preferable to utilize substantial load-transferring devices in conjunction with ordinary methods of subgrade stabilization.

5. Due to the absence of adequate load-transfer, should faulting occur on granular subbase because of greater compaction or rearrangement of the materials under the "receiving ends", not only will the pavement suffer but mudjacking might impair the functioning of the subbase.

6. During 1942, and since, several miles of pavement have been constructed in which no dowels or load-transferring devices were installed at the joints. Instead, the slab ends are supported on 4 in. thickness plain concrete sills (5 ft. wide across practically the entire width of the slabs). Even though these sill joints are in service in comparatively light traffic roads their performance to date has not been entirely satisfactory. There has been some cracking of the pavement at the edges of the sills. During the past winter differential heaving of the adjacent slab ends occurred at many of these joints even though these pavements were laid on 12 in. of apparently good quality granular subbase material. At a number of joints the differential amounted to 1/4 in., the maximum observed being 1/2 in. At present (April, 1945) some of these raised ends have not returned to their normal position. Where frost heaving occurs sills are apparently unsuitable.

7. In New Jersey, no evidence has been found to indicate that strong dowels materially restrain warping, or cause warping cracks.

Subject to minor modifications, New Jersey's heavy-duty concrete pavements to be constructed during the immediate post-war period are being planned essentially as follows:

Pavement Thickness: 9 or 10 in., uniform. Slab Length: Approximately 60 ft. Dummy or Contraction Joints: None. Joint Filler: Cypress, 1 1/8 in. thick, fabricated, grain direction vertical, sealing strip at bottom. Longitudinal Reinforcement: Single or double line of 3/8 in. diameter bars 7 1/2 in. c. to c., or equivalent in welded mats. Longitudinal Joints: Tongue and groove, or tie bars. Subbase: Sand, sand-gravel, stone-sand, or cinders — minimum thickness 8 in. under all pavements on impervious, erodible, or frost-susceptible soils. Load Transfer at Joints: Dowels of rectangular solid bars, or of structural shape, which have a bending resistance within the elastic limit of not less than 7500 inch pounds — 12 in. center to center. The dowels probably will consist of corrosion-resistant materials, or the sliding ends will be provided with positive means to prevent corrosion and to guarantee continued free slippage.

On the basis of present knowledge, the dowel problem appears to be essentially as follows:

1. If the dowels are relatively small the sliding restraint due to corrosion may be inconsequential. However, small members not only lack sufficient strength to transfer heavy truck loads and counteract faulting but are also susceptible to rusting out completely.

2. If, on the other hand, the dowels are of adequate size to fulfill their intended function the effects of corrosion with respect to materially restricting their freedom to slide may have serious consequences.

It is evident, therefore, that to efficiently counteract the faulting of joints in pavements which will be subject to heavy trucking the dowels must be substantially large. But it is also evident, particularly in conjunction with dowels that have a large surface area, that positive means must be provided to insure free slippage indefinitely. To this may be added that it is probable that the more efficiently the joint structure transfers load the more susceptible it is to being restrained by corrosion — due to its greater rigidity and resistance to a loosening effect under traffic.

War-Time Hauling

From December 1943 to August 1944 extremely heavy war-time hauling was done on 21 miles of concrete pavement between Perth Amboy and Colts Neck. The average weight of the loaded trucks was approximately 60,000 lbs. Based on counts, the estimated average daily number of these units passing over the pavement during this period was 212. No serious damage seems to have occurred anywhere during the winter months. But, during a rainy, thawing period early in the spring of 1944, serious failure occurred in several areas where the pavement was laid on silty-clay subgrade. Pumping was extremely violent in these areas, and the slabs soon became cracked at very close intervals. Failure rapidly progressed to such an extent that the pavement in some locations had to be reconstructed. The destruction of the pavements on adverse subgrades continued until the latter part of June when, due to the setting in of a long period of drought, pumping diminished and finally ceased. On the other hand, similar pavements on granular soils were affected much less severely, and none needed reconstruction. Faulting in varying degrees was common at all joints with six 3/4 in. round dowels except in some comparatively short stretches where the native granular soil appears to be of exceptionally good quality. In these exceptional locations, no defects of any kind are apparent.

Samples of the soils supporting these pavements were taken at various places along the trucking route, and the general condition of the pavement was noted. Information relative to the kind of joints, year constructed, condition of the pavement, and the characteristics of the soils in various locations is shown in Table 2. These locations are shown in miles from Perth Amboy.

It is desirable to point out that the pavement on Route 4 and 35, and Route 4, has channel-dowel joints. Route 4 and 35 was laid on native sandy soil, and Route 4 was laid on 8 in. of granular subbase. As noted under "Comments", the pavement on Route 4 and 35 is cracked immediately adjacent to some joints due to excessive sliding resistance of the dowels, but no cracking due to traffic is apparent. On Route 4, of the 225 slabs involved, only 4 have visible cracks, and these are at transitions from cut to fill. The pavement in this 3 mile stretch of roadway does not appear to have been damaged in any way by the heavy trucking.

TABLE 3. NEW JERSEY – 1940 TRAFFIC COUNT – INCLUDING WEIGHTS OF TRUCKS

Route	Location	Type	Average Daily Traffic	% of trucks in daily traffic	Average Daily Truck Traffic	Number of Trucks of Various Weights – Daily Average						
						Less than 5 tons	5 to 10 tons	10 to 15 tons	15 to 20 tons	20 to 25 tons	25 to 30 tons	30 to 35 tons
35	South Amboy	Dual	7599 (South-bound)	9.2	699 (South-bound)	438	164	60	22	9	6	
4	From Route 35 Southerly	Dual	5537 (South-bound)	7.2	398 (South-bound)	250	93	34	13	5	3	
4	North of Matawan	2 Lanes	3627 (South-bound)	7.2	261 (South-bound)	165	61	22	8	3	2	
34	South of Matawan	2 Lanes	2475 (South-bound)	9.2	228 (South-bound)	143	53	20	7	3	2	
25	North of Camden	Dual	9148 (North-bound)	15.6	1427 (North bound	488	404	200	208	116	7	4
25	South of Bordentown	Dual	5667 (North-bound)	18.5	1048 (North bound	259	299	163	202	89	31	5
25	South of New Brunswick	Dual	15,077 (Total)	20.7	3120 (Total)	917	977	568	353	215	84	6
25	North of New Brunswick	Dual	19,843 (Total)	20.7	4107 (Total)	1207	1286	747	464	284	111	8
25	At Newark Airport	4 Lanes	62,184 (Total)	16.0	9949 (Total)	2756	2845	1751	1671	667	189	70
26	North of Trenton	Dual	12,477 (Total)	20.7	2582 (Total)	759	808	470	292	178	70	5
29	Somerville	3 Lanes	11,344 (Total)	17.6	1997 (Total)	581	607	268	318	177	42	4
6	East of Hackettstown	2 Lanes	6000 (Total)	9.3	558 (Total)	155	160	98	93	37	11	4

With respect to the grading and characteristics of the various soils shown in the tabulation, it is desirable to add that most of these samples were taken in cuts and that every effort was made to obtain samples truly representative of the soil supporting the pavement. However, due to the extremely variable types of soil encountered, and particularly to the presence of a thick undulating layer of heavy, black, greasy, silty-clay soil which was frequently found under only parts of a slab, the described condition of the pavement in some locations may appear, and perhaps is, inconsistent with what might be inferred from the characteristics of the samples.

As a matter of general information, a tabulation (lower part of Table 2) has been added showing the characteristics of some of the soils which have contributed to severe pumping and faulting on other routes. Two of the most unsatisfactory subgrade soils in New Jersey, with respect to pumping and faulting, are represented by Samples No. 354,381 and 354,253.

Traffic

The Table 3 shows traffic counts taken in 1940 at points on several trucking routes, and the various weights of the trucks.

PUMPING OF CONCRETE PAVEMENTS IN TENNESSEE

Cooperative Study
By
Tennessee Department of Highways
and
Portland Cement Association

SYNOPSIS

A cooperative study was made during March and April, 1944 to determine the nature, extent, cause and remedy of pumping in Tennessee. Approximately 290 miles of pavement on the main traffic routes connecting the principal cities were studied in the survey. The projects studied were representative of the range of traffic, loadings, pavement designs and subgrade soils encountered over the State.

Reconnaissance surveys were made to obtain a quantitative measure of pumping of pavements under different traffic loads on various soil types. Three stages or classes of pumping were recognized; slab ends where pumping occurred without accompanying faulting; pumping accompanied by faulting; and pumping and faulting accompanied by a broken slab end.

The reconnaissance surveys were followed by detail studies of traffic, pavement design and soils on individual projects. Sections of individual projects were mapped and sampled. Mapping included sketches showing pavement jointing, cracking, pumping, typical soil section, location of sampling, with notes on soil condition and pertinent construction data. Undisturbed as well as disturbed soil samples and water content samples were taken from under both pumping and non-pumping pavement joints and cracks. Density tests, routine soil-constant tests, mechanical analysis, compaction tests, consolidation tests and triaxial compression tests were made to determine the relationships between soil types, soil condition and pumping.

Some of the concrete pavements on the more heavily traveled main highways in Tennessee began to pump at transverse joints and cracks under the heavy axle loads which the roads carried immediately prior to and since the beginning of the war.

This report gives the results of a cooperative study made by the Tennessee Department of Highways, and the Portland Cement Association to determine the causes of pumping and its extent in Tennessee. The laboratory tests of soils was conducted by the Public Roads Administration.

The investigation was limited to the study of factors related to the pumping action under repeated heavy loads causing mud to be ejected at cracks and joints and resulting in eventual loss of subgrade support and breaking of the pavement slab. The ejection of water, where not detrimental, or the faulting of slab ends due to permanent compression of coarse grained soils and not accompanied by removal of soil were not included in the studies.

NATURE AND SCOPE OF STUDIES

The survey was planned to study pumping under the ranges of traffic loadings and soil types which exist on the principal traffic routes. A major portion of the concrete pavement projects on the seven most heavily traveled routes representing the major soil groups and carrying the largest number of heavy trucks were selected for study. Selected projects on two routes carrying medium to light truck traffic also were included in the survey. The heavy traffic routes were as follows:

1. Memphis north on U. S. Route 51.
2. Memphis northeast to Nashville on U. S. Route 70 with some added projects on State Routes 1 and 76 in the vicinity of Milan and Humboldt.
3. Nashville northwest to Clarksville on U. S. Route 41W.
4. Nashville southeast to Chattanooga on U. S. Route 41.
5. Chattanooga northeast to Knoxville on U. S. Routes 27 and 70.
6. Chattanooga northeast toward Knoxville on U. S. Route 11.
7. Knoxville east on U. S. Routes 70 and 11E.

Projects on State Route 76 near Paris and U.S. Route 70N between Cookeville and Crossville which routes carried less traffic than the seven mentioned above were included in the survey.

Reconnaissance Survey of Pumping

A method of classifying pumping slab ends in terms of the progressive stages of development was developed prior to the beginning of the reconnaissance survey. The use of this classification made it possible, by counting the number of pumping slab ends of the various classes, to express the stage of development of pumping, and assess the extent of damage to the pavements on various subgrade soils and under different conditions of traffic.

Pumping was grouped into three classes as follows:

Class 1. Pumping of slab ends at joints and cracks with no evidence of faulting at slab ends or breaking of slabs due to pumping.

Class 2. Pumping accompanied by faulting with no evidence of breaking of slabs due to pumping.

Class 3. Pumping accompanied by faulting and breaking as a result of loss of subgrade support due to pumping.

The reconnaissance survey included a count of the number of expansion joints, contraction joints and cracks for quantitative analysis of pumping. All pumping cracks and joints were classified and counted to determine the number and percentage of pumping slab ends of each of the three classes. Counting was done from an auto driven very slowly over the road. One observer recorded the number of pavement joints, cracks, corner breaks and related pavement items. A second recorder classified and recorded pumping. Where practical, each project was separated into sections according to soil series and horizon, and on some projects between cut and fill and the pumping tabulated for each section. Sections of each project on which pumping and no pumping had developed were noted during the reconnaissance survey for later detailed study.

Detail Survey and Sampling of Selected Sections

Upon completion of the reconnaissance surveys of a number of contiguous projects, the sections noted for detail study were re-examined. Those having the most nearly uniform soil type and soil conditions throughout their length were selected for further study and sampling to determine the reason for the absence or presence of pumping.

The detail survey of selected sections consisted of the following:

1. Mapping the location of all joints and cracks.
2. Describing and classifying pumping and mapping all locations which were pumping.
3. Making soil borings and sketching the soil section represented in the soil profile.
4. Determining the pedological soil series classification.
5. Taking undisturbed soil samples, disturbed soil samples and samples for water content determination.
6. Observing surface drainage condition of shoulders.

An example of the data obtained in the mapping of the detail study sections is shown in Figure 1.

Samples for water content determination were taken at depth of 0 to 1, 2 to 4 and 5 to 8 in. below the pavement. Exceptions were made where pumping had left a cavity, and mud existed in the cavity. There the first sample was taken from the uppermost firm soil beneath the liquid mud. Samples were taken at distances of from two to three ft. in from the edge of the slab. Samples were placed in 6 or 8 ounce metal containers and sealed to prevent loss of moisture.

Undisturbed samples were taken by excavating in from the edge of the slab a distance of two to three feet and removing an undisturbed column of soil as shown in Figure 2. Upon removal the column was trimmed to a cylindrical shape 6 in. in diameter and 8 to 10 in. in height, coated with alternate layers of paraffin and cloth, and packed in sawdust for shipment. Disturbed samples consisted of 50 to 75 lb. soil set aside during removal and trimming of the undisturbed soil cylinder.

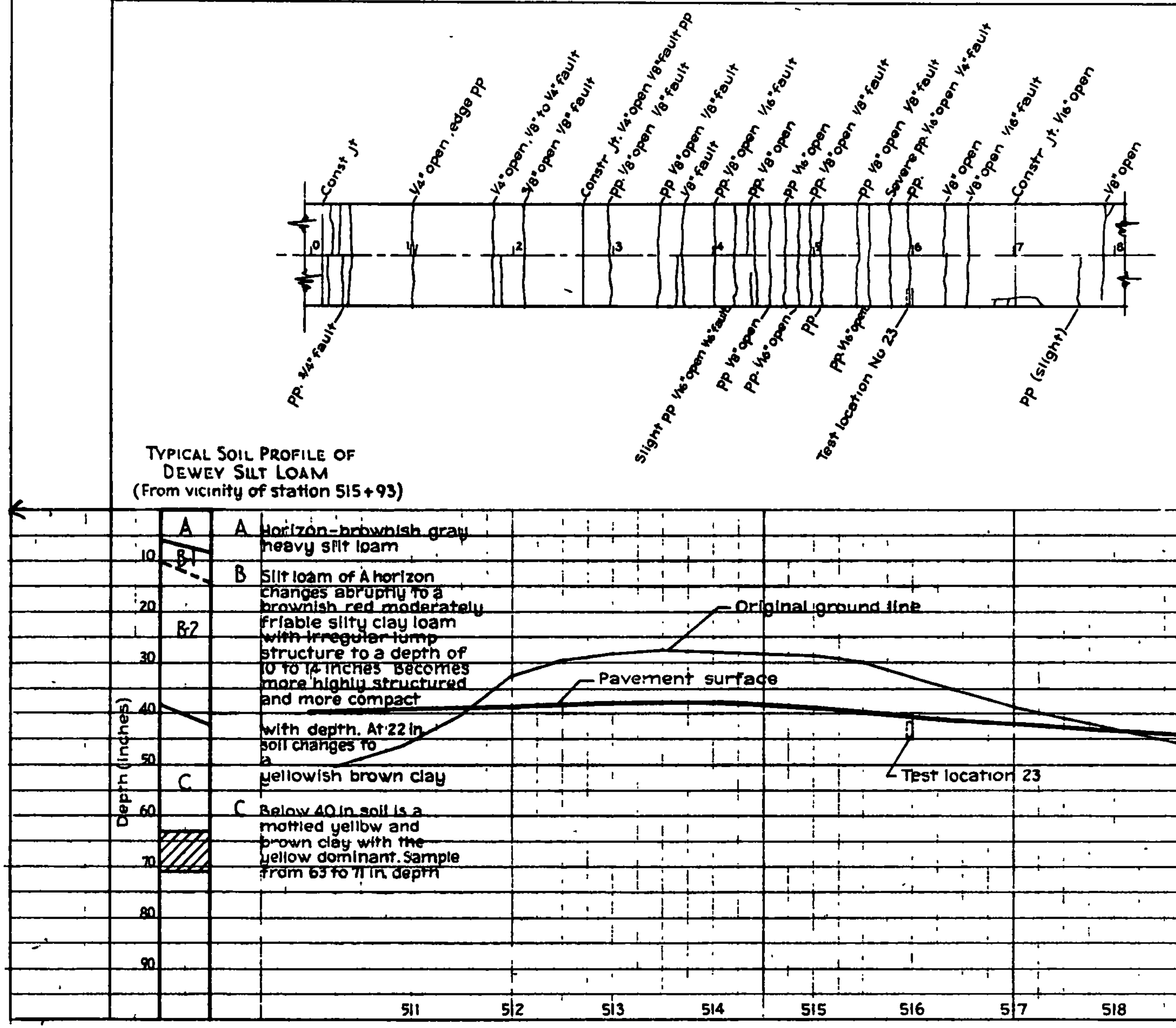
Testing of Soils

Samples for water content determination were tested in the central soils laboratory of the Department of Highways at Nashville. Large undisturbed and disturbed samples were shipped by the Tennessee Department of Highways to the Public Roads Administration for test. The following tests were conducted by the Public Roads Administration Laboratory.

1. Liquid limit, plastic limit, field moisture equivalent, centrifuge moisture equivalent, and shrinkage limit.
2. Mechanical analysis.
3. Standard compaction.
4. Consolidation.
5. Triaxial compression.
6. Water content and density on specimens prepared from undisturbed samples for the consolidation and triaxial compression tests.

Traffic Studies

The earliest comprehensive traffic data available were obtained during the 1937 traffic surveys. Additional traffic data were available from the 1942 and 1943 surveys. In some instances the 1942 count and weighing of vehicles showed a greater number of heavy axle loads than did the 1943 survey. However, since the 1943 survey represented the count most nearly applicable to traffic conditions existing at



Construction Data

Location

State *Tennessee* County *McMinn*

US Route *11* Project *FAP 60 AS*

Location No. *23* Station *510+10 518+10 (515+93)*

Length (Mi) *7.020* Year Built *1926*

Materials

C A *Gravel-Tennessee River-Lenoir City* F A *Sand-Tennessee River-Lenoir City*

Cement *Signal Mt* CF *1:2·3½*

Pavement Design Details

Cross Section *9-6-9 x 18 & 9-6-9-6-9 x 18* Long Jt *Metal plate except at thickened center where butt joint was used*

Tie Bars *½"x4'@5'c.c., except none at thickened center sections.*

Exp. Jt Spacing *None* Width – Filler –

Contr. Jt Spacing *None* Type – Depth Groove –

Load Transfer *None*

Soil Type

Soil Series & Hor. *Dewey C-Horizon*

P.R.A. Group *A-7* Type *Clay* PI *45*

% S & Gr. *18* % Silt *11* % Clay *71*

Soil Condition

Water Content – 0-1" *41* 2"-4" *39* 5"-8" *40* At

Density (PCF) *No test made*

Remarks

Severe edge pumping and several class 3 broken slabs as a result of pumping. Cavity under pavement ⅛" to ¼" deep and 1ft long at station 515+93. Shoulders were high from material recently graded from side ditch and left in a windrow on shoulder. Side ditch was shallow.

Fig I Detail Study Location No 23

the time of the study of pumping it was decided to use the 1943 count. Very little pumping was observed in Tennessee prior to 1937. For these reasons, the data on traffic were compiled for each project for both 1937 and 1943 to show the change in traffic loadings which occurred during the incidence of pumping.

ANALYSIS OF OBSERVATIONS AND DATA

In some instances it was found difficult to obtain a true appraisal of pumping during the reconnaissance survey. It has been mentioned that the progress of pumping from incidence to loss of subgrade support and the breaking of the slab was divided into three stages and classification of pumping made on that basis. During the detail survey when each joint and crack was examined closely it was found that in some locations pumping had begun but had not yet become visible on the surface of the pavement. In the earliest stage, a thin film of mud was found under the slab end on the far side of the joint in the direction of traffic. As the action continued, the mud gradually worked up along the edge of the slab and into the joint or crack opening until it reached the surface. Therefore, the reconnaissance data include only pumping which was visible at the surface while the detail study sections include pumping in its earlier stages where there was unquestioned evidence that a mud had formed and that the mud was progressing toward the surface.

Pumping was found to be a progressive action and its occurrence depended upon so many factors that close correlation between traffic and any other single factor of pavement design feature or subgrade soil type with definite amounts and degrees of pumping was not found. In some instances the maintenance of joints and crack openings in the pavement had been timely and good and had prevented or greatly reduced pumping. In other instances on similar soils and traffic but with less effective maintenance, pumping had become severe and considerable damage done to the pavement.

The correlation between pumping and the various factors which may affect pumping differed with the various factors of subgrade soil, pavement design and traffic. The relationships brought out by the data are given in the paragraphs which follow.

Relation Between Traffic and Pumping

No pumping of serious consequence was reported in Tennessee prior to 1937 when the first comprehensive traffic survey was made. Although no survey of pumping had been made at that time it is safe to assume that the number of truck axle loads at that time did not cause pumping except at locations of poor drainage or as a secondary effect where pavements had become damaged. Traffic counts and weighings were made again during 1942 and 1943. Since the 1943 count was made only a few months before this survey of pumping, the 1943 traffic data have been used. Table 1 shows the comparative 1937 and 1943 traffic count and axle weights for four projects included in the pumping survey. The projects were located near weighing stations used in the traffic survey.

In 1937 traffic carried only a small number of axle loads in excess of 14,000 lb. with few to none in excess of 16,000 lb. The 1943 count showed a large increase in axle weights in excess of 14,000 lb. a large proportion of which exceeded 16,000 lb. in weight. The pavements for which the data are shown are 8 in. by 6 in. by 8 in. parabolic cross section as are many of the pavements examined in this survey.

TABLE I

1937 AND 1943 TRAFFIC COUNTS AND AXLE WEIGHTS ON FOUR PROJECTS
(Located Near 1937 and 1943 Weighing Stations)

Project	Year Built	Location	Year of Traffic Survey	24 Hour Total Traffic (Number Vehicles)	24 Hour Truck Traffic (Number of trucks)	24 Hour Truck Axle Loads in Kips (Cumulative) Under 8	Over 8	Over 10	Over 12	Over 14	Over 16	Over 18	Over 22	Total Number Axles	Remarks
FAP R-8A(4) (Reop. & Ext.)	1936	3 miles north of Memphis on US Route 51	1937	1580	356	586	148	56	33	19	9	-	-	734	
			1943	5740	1383	2479	522	372	189	102	48	9	-	3001	No pumping (Loess Soils)
FAP 36DS	1929	2 miles east of Memphis on US Route 70	1937	3200	547	845	304	182	60	11	-	-	-	1149	
			1943	3740	748	1302	575	517	367	196	130	40	8	1877	30% of joints and cracks pumping (Loess Soils)
FAP 231AS	1930	10 miles north of Chatta-nooga on US Rt. 27	1937	2560	351	477	224	183	131	23	-	-	-	702	Clay soils derived from dolomitic and cherty limestones.
			1943	3780	692	1124	530	457	359	262	131	25	-	1654	17% of joints and cracks pumping.
NRH 269B	1934	17 miles east of Chatta-nooga on US Routes 41 & 64	1937	1180	313	517	150	95	60	9	4	-	-	667	Clay soils derived from cherty limestones.
			1943	2975	690	1054	574	481	398	282	116	23	-	1628	26% of joints and cracks pumping.

It appears from the data shown in the table that pumping developed from the increase in number of axle loads of the 14,000 lb. to 18,000 lb. range. The axle loads necessary to start pumping undoubtedly depended on the type and condition of the subgrade soil.

A correlation between traffic and pumping was found for the uniform loess soils in western Tennessee. Table 2 shows the soil analyses, axle loads and pumping found on 11 projects built on loess soils. It is evident that the loess soils on the various scattered projects are very nearly similar in characteristics. Under average maintenance conditions the pavement built on loess soils of average water content and density did not pump until the number of 14,000 lb. wheel loads exceeded 50 per day on an 8 in. by 6 in. by 8 in. pavement. A study of the projects on which slight to moderate pumping occurred showed that such pumping was associated largely with open cracks or poor surface drainage of the pavements. On project 36DS (7.97 miles in length) 122 of the 135 pumping slab ends were at cracks. The higher water content of the soil on that project is indicated in Table 2.

An over-all study of pavements built on all soil types failed to show any direct relation between the amount of pumping and the number of axle weights in the 12,000 to 18,000 lb. range.

No quantitative data were obtained on the relation between speed of traffic and pumping. It was observed, however that many of the long mountain grades in eastern and southeastern Tennessee showed a marked difference in surface condition and pumping on the uphill traffic lanes compared to the downhill traffic lanes. The very marked difference in the density of the oil streak (indicative of slow speed and tractive effort) found on many long grades is shown in Figure 3. A second example is shown in Figure 4. Here severe pumping occurred on the uphill lane while the downhill lane showed little pumping.

The influence of traffic on pumping also was evident on 4 lane roads where nearly all pumping occurred in the outside lane which carries nearly all truck loads. Almost no pumping occurred in the inner passing lane of 4 lane divided or undivided highways.

Relation Between Roadbed Design and Pumping

A tabulation of pumping slabs on multi-lane divided and undivided highways showed the undivided highway had a tendency to pump somewhat less than the divided type.

No effort was made to determine the relative effect of lip curb on pumping during the reconnaissance survey. General observations were that its use did not increase or decrease pumping to an extent which was significant.

On four lane divided and undivided highways both traffic lanes drained toward the shoulder. There was no evidence that such drainage resulted in more pumping than occurred on many two lane roads.

Relation Between Pavement Cross Section and Pumping

Of 36 projects on which pumping was studied during the survey, 26 were of 8 in. by 6 in. by 8 in. cross section. Ten of the 36 projects were 18 ft. wide, 18 were 20 ft. wide and 3 were 22 ft. wide. Two projects were of 7 in. uniform thickness and 4 were of 9 in. by 7 in. by 9 in. by 20 ft. cross section.

TABLE 2

SUBGRADE SOILS DATA, AXLE LOADS AND PUMPING ON LOESS SOILS

County	Project	Sample Location No.	L.L.	P.L.	P.I.	F.M.E.	C.M.E.	S.L.	P.R.A. Soil Group	Mechanical Analysis: Gravel & Sand	Mechanical Analysis: Silt	Mechanical Analysis: Clay	Mechanical Analysis: Colloids	Textural Soil Type	Soil Water Content (Per Cent): 0-1"	Soil Water Content (Per Cent): 2"-4"	Soil Water Content (Per Cent): 5"-8"	Soil Density (p.c.f.)	24 Hr. Truck Axle Loads in Kips (Cumulative): Over 12	Over 14	Over 16	Over 18	Remarks on Pumping
Dyer	39AS	13C	40	22	18	32	25	20	A-7	10	61	29	16	SiCL	-	-	-	-	54	29	14	3	None
Henry	280F	37	37	19	18	26	22	18	A-7	18	52	30	14	SiC	18	18	18	-	54	29	19	6	None
Shelby	31CS	12b	32	21	11	27	19	22	A-4	25	61	14	7	SiL	-	-	-	-	57	31	15	3	None
Gibson	147CS-DS	14C	35	18	17	24	22	18	A-7-4	19	61	20	8	SiCL	-	-	-	-	92	49	23	4	None
Madison	51CS	11	39	19	20	28	24	17	A-7-6	22	53	25	10	SiCL	21	24	22	101-108	93	50	33	10	None
Shelby	R-8-A IV Reop. Ext.	10C	36	21	15	28	25	22	A-4	10	67	23	10	SiCL	-	-	-	-	189	102	48	9	None
Fayette	36AS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	227	121	80	24	None
Shelby	36DS	9	45	23	22	32	29	19	A-7	11	61	28	12	SiCL	23	26	27	93-96	367	196	130	40	None (for detail Study Section) only
Gibson	147CS-DS	15b	42	18	24	33	26	20	A-7	10	56	34	19	SiC	-	-	-	-	92	49	23	4	Slight Pumping (No Count Made)
Shelby	R-8-A III	11C	40	21	19	29	27	20	A-7	11	65	24	10	SiCL	-	-	-	-	140	76	36	7	Moderate Pumping (No Count Made)
Shelby	36BS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	174	93	62	19	Moderate Pumping (3% of Joints and Cracks Pumping)
Shelby	36CS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	199	106	70	21	Moderate Pumping (3% of Joints and Cracks Pumping)
Shelby	36DS	10	46	23	23	35	30	20	A-7	9	60	31	6	SiC	25	26	26	96	367	196	130	40	Severe Pumping (30% of Joints and Cracks Pumping)
Shelby	36GS	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	475	253	168	51	Slight Pumping (0.5% of Joints and Cracks Pumping)

Samples 13C, 14C, 9, 15b, 11C and 10 were classified as Loring Series; Samples 12b, 11, and 10C as Memphis Series and Sample 37 as Lexington Series. All samples not followed by the letter "C" were from the B Horizon.

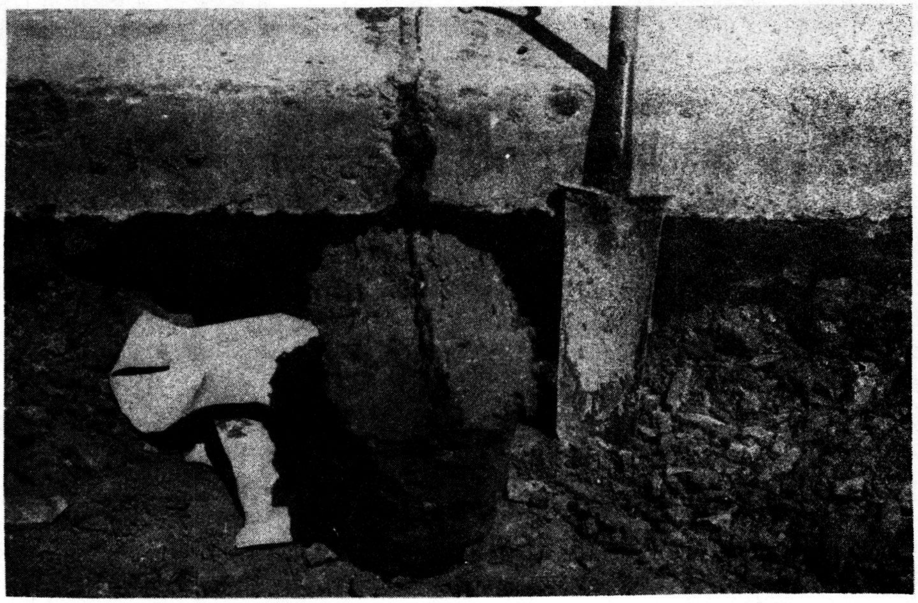


FIGURE 2. UNDISTURBED COLUMN OF SOIL REMOVED FROM UNDER A PUMPING EXPANSION JOINT. NOTE THAT A SOFT MUD HAD FORMED ONLY UNDER THE FAR SIDE OF THE JOINT IN THE DIRECTION OF TRAFFIC.

FIGURE 3. NOTE THE DIFFERENCE IN THE OIL STREAKS RESULTING FROM DIFFERENCE IN UP-HILL AND DOWNHILL SPEEDS AND TRACTIVE RESISTANCE OF VEHICLES ON A LONG GRADE. FAP 486 A (2) STA. 380 COFFEE COUNTY (4-6-44)

FIGURE 4. ANOTHER EXAMPLE SHOWING THE DIFFERENCE IN THE OIL STREAKS IN THE UP-HILL AND DOWNHILL LANES DUE TO SPEED OF TRAFFIC ON A LONG 6% GRADE. NOTE THE PUMPING AND DAMAGE TO PAVEMENT IN THE UPHILL TRAFFIC LAND. FAP 269 B MARION COUNTY (3-22-44)

Pumping occurred on all pavement cross sections where other conditions conducive to pumping were present. The 9 in. by 7 in. by 9 in. thickness showed an average of approximately 1/3 as much pumping as did the 8 in. by 6 in. by 8 in. pavements, but the former carried about two thirds as many heavy axle loads as did the 8 in. by 6 in. by 8 in. pavements.

Relation Between Pumping and Joint Type and Spacing

The counting of the total number of joints and cracks made it possible to determine the joint and crack interval for various expansion joint spacings and combinations of expansion joint and contraction joint spacing. Pavement with longer expansion joint spacings had longer joint and crack intervals. A trend in reduction of pumping is indicated for pavements with expansion joints at 300 ft. intervals. For pavements with the 500 ft. expansion joint spacing there was a considerable reduction in the amount of pumping.

Pumping was classified and recorded separately at joints and cracks on 33 of 41 projects studied. The 33 projects represent a total of 229 miles of pavement and cover the range of soils encountered in Tennessee. A summary of observations on 41 projects is given in Table 3. In the average, the amount of pumping at joints differed but little from the pumping at transverse cracks. However, very few individual projects showed the same amount of pumping at joints and cracks, the amount being influenced by the occurrence of cracks, the spacing of expansion joints and resulting width of crack opening, the relative maintenance condition of the sealing material at joints as compared to cracks, and apparently, whether the dowels at joints permitted free movement of the slab ends. The following examples illustrate the wide difference in pumping at joints as related to pumping at cracks on individual projects: On one 4-lane divided highway, having no transverse cracks, 6 per cent of the joints pumped. On another project pumping occurred on 1 per cent of the joints compared to pumping at 21 per cent of the transverse cracks. The expansion joints were uniformly of the width constructed but cracks had opened from 1/8 to 3/8 in. indicating definitely that the dowels were restricting normal slab movements on that project.

Table 3 shows that in the average the amount of pumping of transverse joints (expansion and contraction joints) was about the same as at transverse cracks, (18 and 21 per cent of the joints and cracks respectively).

It was difficult to differentiate between expansion joints and contraction joints in the reconnaissance survey on many of the projects investigated. Therefore, observations on the relative pumping at expansion and contraction joints were limited to six projects having a combined length of approximately 62 miles. It may be seen from the average values in Table 4 that pumping occurred in almost equal amounts at expansion joints, contraction joints and cracks.

Considerable pumping occurred along the longitudinal center joint near transverse joints and cracks. A deformed metal plate was used to form the center longitudinal joint on most projects. Likewise, pumping occurred along the outside edge of the pavement. All pumping at the edge and at the center joints was credited to the nearest transverse crack or joint. In only one instance was pumping found along the edge of uncracked slabs of 20 to 30 ft. in length and that was on the inside of a superelevated curve in a deep cut near the top of a long steep grade.

Table 3

SUMMARY OF OBSERVATIONS OF PUMPING ON 41 PROJECTS IN TENNESSEE

(Located on Principal Traffic Routes)

Project	Location: Length (Miles)	Location: County	Location: Route	Pavement Design: Pavement Cross-Section	Pavement Design: Joint Spacing Exp.Jt.	Pavement Design: Joint Spacing Contr.Jt.	Pavement Design: Load Transfer	Joint and Crack Count: Total No. Joints	Joint and Crack Count: Total No. Cracks	Joint and Crack Count: Joints and Cracks	Pumping Joints (Per Cent): Class 1	Pumping Joints (Per Cent): Class 2	Pumping Joints (Per Cent): Class 3	Pumping Joints (Per Cent): Total	Pumping Cracks (Per Cent): Class 1	Pumping Cracks (Per Cent): Class 2	Pumping Cracks (Per Cent): Class 3	Pumping Cracks (Per Cent): Total	Pumping Joints & Cracks (Per Cent): Class 1	Pumping Joints & Cracks (Per Cent): Class 2	Pumping Joints & Cracks (Per Cent): Class 3	Pumping Joints & Cracks (Per Cent): Total	Miles of 2 lane Width of Pumping Pavement
35	7.650	Hamilton	SR 29	6-7½-6x16'	--	—	—	921	353	1274	1.3	0.7	2.9	4.9	6.8	2.5	4.8	14.2	2.8	1.2	3.5	7.5	0.49
36 AS	7.882	Fayette	US 70	8-6-8x18'	35	--	None	1252	785	2037	—	—	—	—	—	—	—	—	—	—	—	—	—
36 BS	2.138	Shelby	US 70	8-6-8x18'	500	—	None	20	472	492	—	—	—	—	1.9	0.4	0.4	2.8	1.8	0.4	0.4	2.6	0.06
36 CS	8.624	Shelby	US 70	8-6-8x18'	500	—	None	1090	1912	3002	0.6	—	—	0.6	2.4	1.0	0.4	3.8	1.8	0.6	0.3	2.7	0.23
36 DS	7.970	Shelby	US 70	8-6-8x18'																			
				8-6-8-6-8x36'	500	—	None	52	396	448	11.5	5.8	7.7	25.0	22.2	7.8	0.8	30.8	21.0	7.6	1.6	30.1	2.40
36 GS	1.820*	Shelby	US 70	8-6-8-6-8x36'	500	--	None	34	565	599	--	—	—	—	0.5	—	—	0.5	0.5	—	--	0.5	0.02
41AB(4)(5)	2.264*	Knox	US 70	7" unif.x22'	90	30	(Special (Base Plate	786		786	6.4	—	—	6.4	—	—	—	—	6.4	—	--	6.4	0.58
41AB(Reop.)	7.291*	(Knox- (London	US 70	7" unif.x22'	90	30	(Univ. or (Nat'l.	2549	11	2560	38.0	0.2	—	38.2	27.3	—	—	27.3	38.0	0.2	—	38.2	5.57
51AS(& Ext)	10.697	Carroll	US 70	8-6-8x18'	40	—	¾"x4' at 3'c.c.	1401	1033	2434	—	—	—	—	—	—	—	—	—	—	—	—	—
55AS	9.689	Anderson	US 25W	8-6-8x18'	40	—	¾"x4' at 3'c.c.	1279	248	1527	—	—	—	—	—	—	—	—	—	—	—	—	—
56A(2)(3)	5.231*	Davidson	US 41	(8-6-8-6-8x22' (& 7"unif.x22'	90	30	¾"x2' at 12"c.c.	1027	192	1219	26.9	1.2	—	28.2	12.0	11.55	0.5	24.0	24.5	2.8	0.1	27.4	2.88
58AS	10.646	Bradley	US 11	8-6-8x18'	40	—	None	1391	105	1496	3.5	3.5	0.4	7.3	4.8	—	—	4.8	3.5	3.3	0.3	7.2	0.72
60AS	7.020	McMinn	US 11	(9-6-9x18' & (9-6-9-6-9x18'	—	—	--	75	678	753	9.3	8.0	2.7	20.0	9.6	6.0	1.8	17.4	9.6	6.2	1.9	17.6	1.76
78 AS	9.263	Bradley	US 11	? x18'	35	—	None	1365	357	1722	2.9	1.3	0.9	5.1	3.1	—	—	3.1	3.0	1.0	0.7	4.7	0.47
78 BS	7.110	Hamilton	US 11	8-6-8x20'	40	—	¾"x4' at 3'c.c.	935	424	1359	1.8	0.3	—	2.1	4.0	1.2	—	5.2	2.5	0.6	—	3.1	0.31
208 G	8.901	Rutherford	US 41	8-6-8x20'	90	30	¾"x2' at 12"c.c.	1573	630	2203	44.0	9.8	2.2	55.9	39.3	49.3	9.5	98.1	42.6	21.1	4.3	68.0	6.19
208 H	3.181	Coffee	US 41	8-6-8x20'	90	30	¾"x2' at 12"c.c.	552	169	721	15.4	2.4	0.4	18.1	19.5	37.3	10.0	66.8	16.3	10.5	2.6	29.5	0.94
208 I (2) 486 A (2)	5.963	Coffee	US 41	8-6-8-6-8x22'	90	30	¾"x2' at 12"c.c.	1022	415	1437	1.2	0.1	—	1.3	9.2	10.8	1.0	21.0	3.5	3.1	0.4	7.0	0.42
208 J	7.316	Rutherford	US 41	8-6-8x22'	90	30	¾"x2' at 12"c.c.	1320	1369	2689	6.0	0.2	—	6.1	0.4	0.3	—	0.7	3.2	0.2	—	3.4	0.34
228 B	7.211	Knox	US 70	9-7-9x20'	300	50	None	708	1144	1852	19.4	4.9	0.6	25.0	20.5	8.1	1.2	29.8	20.0	6.9	1.0	28.0	2.01
231 A	4.046	Hamilton	US 27	8-6-8x20'	500	--	None	71	344	415	50.7	7.0	—	57.7	8.7	—	—	8.7	15.9	1.2	—	17.1	0.69
231 B	4.355	Roane	US 27	8-6-8x20'	300	50	None	464	62	526	3.9	1.7	—	5.6	1.6	1.6	--	3.2	3.6	1.7	--	5.3	0.23
231 D	3.587	Hamilton	US 27	9-7-9x20'	300	—	None	64	150	214	—	1.6	1.6	3.1	2.0	—	--	—	1.4	0.5	0.5	2.3	0.08
35 A (3) 231 D (2)	1.353	Hamilton	US 27	8-6-8-6-8x22'	90	30	¾"x2' at 12"c.c.	234	20	254	—	—	—	—	—	—	—	—	—	—	—	—	--
231 E	8.792	(Hamilton- (Rhea	US 27	8-6-8x20'	500	—	None	146	649	795	3.4	1.3	—	4.8	10.0	1.8	—	11.9	8.8	1.8	—	10.6	0.93
231 GPC	25.215	Rhea	US 27	9-7-9x20'	300	50	None	2651	772	3423	2.5	0.2	0.1	2.8	2.1	0.4	—	2.5	2.4	0.2	--	2.7	0.68
239 A	6.574	(Cheatham- (Robertson	US 41-W	8-6-8x20'	90	30	¾"x2' at 12"c.c.	1144	69	1213				**				**	20.0	13.4	—	33.4	2.20
239 B	12.887	Montgomery	US 41-WJ	8-6-8x20'	90	30	¾"x2' at 12"c.c.	2302	385	2687				**				**	5.6	1.5	0.2	7.3	0.94
239 C	7.942	Cheatham	US 41-W	8-6-8x20'	90	30	¾"x2' at 12"c.c.	1402	28	1430				**				**	16.6	6.9	—	23.5	1.87
240 A	10.493	Marion	SR-2	8-6-8x20'	500	—	None	143	1180	1323	9.8	5.6	5.6	21.0	4.1	1.3	6.4	11.8	4.7	1.7	6.3	12.7	1.28
267 A	8.361	Davidson	SR 24	9-7-9x20'	320	40	None	1104	426	1530	—	—	—	—	0.2	—	—	0.2	0.1	—	—	0.1	0.05
269 B	5.627	Marion	US 41	8-6-8x20'	320	40	None	735	314	1049	5.9	12.2	5.7	23.8	18.5	11.8	1.3	31.5	9.6	12.1	4.4	26.1	1.47
269 C	5.238	Marion	US 41	8-6-8x20'	90	30	¾"x2' at 12"c.c.	857	254	1111	8.3	1.3	4.3	13.9	21.7	15.0	2.4	39.0	11.3	4.4	3.9	19.6	1.03
269 D	5.586	(Marion- (Hamilton	SR-2	8-6-8x20'	320	40	None	709	144	853	24.5	18.1	3.7	46.3	36.8	11.1	3.5	51.4	26.6	16.9	3.6	47.1	2.63
269 H	3.851	Marion	US 41	8-6-8x20'	90	30	¾"x2' at 12"cc.	633	215	848	17.2	1.9	1.7	20.9	13.0	23.3	2.8	39.1	16.2	7.3	2.0	25.5	0.98
280 F	5.145	Henry	SR-76	8-6-8x20'	90	30	J-bars at 20"c.c.	893	415	1308	—	—	—	—	—	—	—	—	—	—	—	—	—
313 AS	4.105	Louden	SR-1	8-6-8x18'	40	—	None	546	142	688	27.5	18.5	9.0	54.9	26.1	9.9	5.6	41.6	27.2	16.7	8.3	52.2	2.14
322 BS	2.943	McMinn	US 11	8-6-8x18'	500	—	None	55	191	246	3.6	12.7	12.7	29.1	3.1	8.4	12.6	24.1	3.3	9.3	12.6	25.2	0.74
339 CS	6.498	Hamilton	US 11	8-6-8x18'	40	—	¾"x4' at 3'cc.	808	619	1427	5.2	1.2	0.4	6.8	8.1	3.4	1.0	12.5	6.4	2.2	0.6	9.2	0.60
379 AS	3.110	Knox	US-25W70	8-6-8x18'	500	—	None	53	385	438	30.2	7.5	1.9	39.6	21.6	9.9	3.4	34.8	22.6	9.6	3.2	35.4	1.10
379 BS	3.003	Sevier	US-25W70	8-6-8x18'	500	—	None	51	449	500	5.9	—	—	5.9	15.6	3.6	0.2	19.4	14.6	3.2	0.2	18.0	0.54
							Average (Pumping Projects)				12	4	2	18	12	7	2	21	12	5	2	19	
Total	291.4																						42.8

** Pumping Joints and Cracks not recorded separately in reconnaissance survey.

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It should be noted here that distributed reinforcing was used on only a part of one of the projects studied. Also that the data available did not permit a study on the influence of expansion joint fillers. Construction records showed that nearly all projects were built with premolded or poured bituminous fillers.

Load Transfer Devices and Their Relation to Pumping

Twelve projects on which load transfer devices were used and 30 projects without load transfer devices at joints were investigated in the reconnaissance survey. Four of the projects on which no load transfer devices were used were located on soils of loess origin. Inasmuch as the other projects were on clay and cherty clay soils having generally similar characteristics, the four projects on loess soils were left out in assembling the average values shown in Table 5. Inclusion of pumping data from the four projects on loess soils would result in only very small changes in some of the values for pumping shown in the table.

The total per cent of pumping joints was only slightly greater for the projects without load transfer devices. However, the truck traffic on the projects involving the use of load transfer devices was much heavier than on the older pavements built without load transfer devices. The load transfer devices did appear to reduce to marked degree the per cent of classes 2 and 3 pumping joints which involve faulting and breakage. However, it should be noted that more severe pumping of classes 1 and 2 was found at intermediate transverse cracks where the pavements were built with load transfer devices at joints.

Close inspection of transverse joints on two of the projects on which load transfer devices were used showed that the dowels did not allow free movement of the slabs when the slab expanded or contracted on those projects. The total effect of restricted movement due to frozen dowels could not be determined for all projects, but it may account for the more severe pumping generally indicated at transverse cracks in pavements having doweled joints.

The Effect of Shoulder Drainage on Pumping

Pumping occurred under conditions of good surface drainage as well as poor surface drainage. Severe pumping was observed on grades and on heavy clay soils at many locations where shoulders were lower than the pavement edge, had slopes in excess of one inch per foot and where runoff was not impeded by vegetation. Contrariwise, pumping occurred on some projects having loess subgrade soils only where poor surface drainage existed.

The Subgrade and Its Relation to Pumping, Soil Parent Material and Soil Series

The reconnaissance and detail study surveys were planned for the principal traffic routes traversing the major representative soil groups of Tennessee. Beginning at Memphis and proceeding eastward the survey covered the following soil areas in the order given:

1. The yellow silty clay loams and silty clay soils of western Tennessee which originate from deep loess to the soils derived from shallow loess underlain by coastal plains materials.

TABLE 4
COMPARISON OF PUMPING AT EXPANSION JOINTS, CONTRACTION JOINTS AND CRACKS
(Six Projects totaling 62 Miles in Length)

Projects	Pumping Exp. Joints (Per Cent) Class 1	2	3	Total	Pumping Contr. Joints (Per Cent) Class 1	2	3	Total	Pumping Cracks (Per Cent) Class 1	2	3	Total	Pumping Joints & Cracks (Per Cent) Class 1	2	3	Total	Remarks
41AB (4)(5)	18.7	-	-	18.7	9.7	-	-	9.7	-	-	-	-	12.7	-	-	12.7	2-22 ft. lanes - 8 ft. grass dividing strip. 7" unif. 30-90 ft. Jt. spacing. Edge curb along dividing strip. Univ. or Nat'l Load Transfer. Heavy clay soil.
41AB (Reop.)	40.4	0.2	0.1	40.7	36.8	0.1	-	36.9	27.3	-	-	27.3	38.0	0.2	-	38.2	2-22 ft. lanes - 4 ft. concrete dividing strip. 7" unif. 30-90 ft. Joint spacing Spec. base plate under Exp. Jts. Heavy clay soil with 2" loose sand mixed to depth of 6".
208I(2) 486A(2)	3.0	-	0.3	3.3	0.3	-	-	0.3	9.2	10.8	1.0	21.0	3.5	3.1	0.4	7.0	8-6-8-6-8x22' 30-90 ft. Joint spacing. $\frac{3}{4}$"x24" dowels at Exp. & Contr.Jts. at 12" c.c. A-4 clay soil with 2" to 3" loose sand mixed to depth of 6" to 8".
228BS	15.7	7.4	0.9	24.0	20.2	4.5	0.5	25.2	20.5	8.1	1.2	29.8	20.0	6.9	1.0	27.9	9-7-9x20' 50-300 ft. joint spacing. No load transfer. A-6-7 cherty clay soils.
231BS	-	-	-	-	4.7	2.1	-	6.8	1.6	1.6	-	3.2	3.6	1.7	-	5.3	8-6-8x20' 50-300 ft. joint spacing. No load transfer. A-6-7 cherty clay soils.
231GFC	2.0	0.2	0.2	2.4	2.6	0.2	-	2.8	2.1	0.4	-	2.5	2.4	0.2	0.1	2.7	9-7-9x20' 50-300 ft. joint spacing. No load transfer. Cherty clay soils.
Average	13.3	1.3	0.3	14.9	12.4	1.2	0.1	13.7	10.1	3.5	0.4	14.0	13.4	2.0	0.3	15.7	

Note: No cracks were observed on Project 41AB (4)(5)

TABLE 5

RELATION BETWEEN LOAD TRANSFER DEVICES, PUMPING AND TRAFFIC
(33 Projects Having Clay, Clay Loam and Cherty Clay Subgrades)

Number of Projects	Load Transfer	Pumping Joints (Per Cent) Class 1	2	3	Total	Pumping Intermediate Cracks (Per Cent) Class 1	2	3	Total	Pumping Joints & Cracks (Per Cent) Class 1	2	3	Total
21	None	13	6	3	22	12	5	3	20	11	6	3	20
12	3/4"x24" at 12"cc*	18	2	1	21	17	17	3	37	17	6	1	24

Number of Projects	Load Transfer	24 Hour Traffic: Total Vehicles	Total Trucks	Truck Axle Loads in Kips (Cumulative): Over 10	Over 12	Over 14	Over 16	Over 18
21	None	2,186	419	256	200	142	65	12
12	3/4"x24" at 12"cc*	3,144	690	441	362	258	109	21

*One project used a special type of load transfer device and a second project used 3/4 in. x 48 in. dowels spaced on 36 in. centers.

2. Sandy soils derived from coastal plains materials.

3. Cherty clay soils in west central Tennessee derived from cherty limestones.

4. Clay soils derived from limestones and shales and massive limestones, including phosphatic limestones.

5. Sandy and silty soils derived from sandstones and shales.

6. Gravelly clay soils derived from cherty limestones and limestones in east central and east Tennessee.

Soils which originate from the deeper loess formations were investigated on U.S. Route 51 north of Memphis in Shelby, Lauderdale and Dyer counties; on U.S. Route 70 in Shelby, Fayette, Madison and Carroll counties and on State Route 76 in Gibson county. Thin loess underlain by coastal plains deposits was investigated in Henry county on State Route 76.

The deep loess is classified pedoligically into three soils series as follows:

1. The Memphis series occupies the narrow highland ridges and forms on rolling to hilly land. It has a slightly to moderately plastic B horizon of yellowish brown to faintly reddish brown silty clay loam. Relatively good surface and subsurface drainage are characteristic of the Memphis series.

2. The Loring series occupies the nearly level to rolling uplands. It develops a more plastic and more compact B horizon which is characterized by slower drainage than that of the Memphis series. The brownish silty clay loam and clay B horizon is splotched with gray, rust-brown and yellow characteristic of less perfect drainage.

3. The Granada series consists of loess overlying coastal plains material at depths of 4 to 15 ft. It develops a semi-clay-pan B horizon which restricts drainage and occupies undulating to gently rolling topography ranging in slope from about 2 to 7 per cent.

A fourth series, the Lexington, consists of a few inches to about 2 ft. of loess over ferruginous sands and sandy clays of the coastal plains deposit. Roads on the Lexington soils lie mostly in the sandy materials although one location investigated in Henry county was in the B horizon of a soil classified as Lexington.

No pumping was found during this survey on loess soils of the Memphis group, all pumping being found on the B horizon of the Loring series which is a compact, imperfectly drained soil. Although no pumping was found on the more lightly traveled roads built on loess soils of the Granada series, it is believed that its compact and imperfectly drained B horizon needs attention to avoid pumping under a large volume of heavy axle loads.

A summary of soils test data, axle weights and pumping is given in Table 2 for all loess soils tested. Neither the soils constants nor the mechanical analysis indicate a wide difference in the physical characteristics of these soils.

It is evident from the test data that there is a very small difference between the loess soils where pumping has or has not developed. The difference is mainly one of compactness and movement of water in the soil profile.

No pumping occurred on the sandy soils of the coastal plains materials which are classified in the Ruston series and which form the C horizon of the Lexington series.

Pumping prevailed on pavements built on all cherty clay soils in west central Tennessee which were derived from cherty limestones. These soils were classified in the Baxter, Dickson and Bodine series. Pumping occurred in both the B and C horizons of all 3 series except where the sand and gravel content prevented its occurrence. The relation between sand and gravel content of a subgrade soil and pumping is discussed later in the report.

Pumping occurred throughout the Hagerstown, Maury and Mercer groups derived from limestones and also from gravelly clay soils derived from cherty and dolomitic limestones except for subgrades having high gravel content which will be discussed later. These gravelly clay soils are represented by the Clarksville, Fullerton and Dewey series.

No pumping occurred on the more sandy soils of the Muskingum and Hartsells series which were formed from sandstones and shales. However, the occurrence of strata of fine plastic clay in some areas of sandy soils created localized areas of poor drainage which resulted in severe pumping of the clay soils even under relatively light traffic.

PRA Soil Groups

Pumping was found on the A-7, A-6 , A-5-7 and A-4 soil groups. It was also found in the A-2-4 and A-2-6 borderline groups having near the minimum sand content in the soil mortar for those soils.

No pumping was encountered in soils of the A-2 and A-3 groups nor on many of the borderline A-2-4 and A-2-6 and A-2-7 groups containing near maximum sand and gravel content for those groups.

In many instances soil samples taken from under pumping and non-pumping slabs showed generally similar soil constants. This is true for some of the clay, silty clay and silty clay loam soils. It is difficult, for that reason, to make a positive statement regarding the limiting values of soil constants for pumping or non-pumping soils. It is of interest to note in Table 6, which shows soil constants of all soil samples taken from detail study sections that both the range of values and the average values of all soil constants are considerably lower for the soils of the clay, silty clay, silty clay loam and clay loam soils where no pumping occurred than for soils of similar textural types found under pumping slabs.

Consolidation and Triaxial Compression Characteristics

Tests were made to determine the relative consolidation characteristics of soil types and soil conditions associated with pumping as well as those found where no pumping occurred. Undisturbed samples were taken immediately under or adjacent to transverse joints and cracks. A portion of each sample was tested in the condition in which it was received at the laboratory. A test was made on another portion of the sample in an inundated condition.

Nine soils from under pumping slabs and five soils from under non-pumping slabs were tested for consolidation. The average values of per cent reduction in height for various loads and average per cent consolidation between load increases at various time periods for both pumping and non-pumping soils are given in Table 7.

The data, although representing relatively few samples, do indicate a trend of greater total and residual compression for pumping soils than for non-pumping soils. Soils from under pumping slabs resulted in greater consolidation under all loads in both the "existing" and "inundated" conditions and consolidated in a shorter time than did the samples from under non-pumping slabs. The data therefore indicate a trend of greater total and residual compression for the pumping than for the non-pumping soils.

A tabulation of per cent reduction in height and maximum difference between vertical and normal pressures obtained in the triaxial compression test for both pumping and non-pumping soils is given in Table 8. The values for per cent reduction in height follow a trend similar to that observed in the consolidation tests. The values for shearing stress computed from the maximum differences between vertical and lateral pressures shown in Table 8 are erratic and do not show a well defined trend.

Permeability tests were made on nine pumping soils and six non-pumping soils. All were fine grained silty clay loam to clay soils except one sand. No significant trends are evident from the data obtained.

TABLE 7

SUMMARY OF DATA FROM CONSOLIDATION TESTS

Method of Conducting Consolidation Tests	Per Cent Reduction in Height at Load of (Kips /.s.f.)								Remarks
	0.02	0.10	0.35	1	2	4	8	0.10	
Tested as Received	0.0	0.31	0.52	0.89	1.36	2.38	3.94	2.69	Pumping Slabs
Tested as Received	0.0	0.14	0.28	0.52	0.94	1.48	2.54	1.75	Non-Pumping Slabs
Inundated	0.0	0.11	0.01	0.40	1.10	2.13	3.87	2.18	Pumping Slabs
Inundated	0.0	0.0	0.10	0.36	0.78	1.56	2.72	1.43	Non-Pumping Slabs

Method of Conducting Consolidation Tests	Average Per Cent Consolidation at Time of (Minutes)								Remarks
	0.09	0.25	0.49	1	4	9	25	64	
Tested as Received	37	42	47	51	60	66	74	79	Pumping Slabs
Tested as Received	48	54	60	66	73	78	82	86	Non-Pumping Slabs
Inundated	44	52	56	62	71	78	83	88	Pumping Slabs
Inundated	51	57	62	68	77	81	86	90	Non-Pumping Slabs

Sample 38, a non-plastic sand, not included in above tabulation.

Soil Texture

Data obtained from sieve and hydrometer analysis on all samples of soil from detail study sections are shown in Table 6. The data obtained on soils from under pumping and non-pumping slabs show a definite relation between textural soil type and pumping, when the grading of the total subgrade soil is considered. The percentages shown in Table 6 as sand and gravel, comprise all sand and gravel in the total soil sample, i.e., all materials coarser than 0.05 mm. diameter (retained on No. 270 sieve). The relation between pumping and soil texture is illustrated in the triangular chart in Figure 5.

No pumping was encountered in Tennessee on soils having more than 55 per cent sand and gravel (coarser than 0.05 mm. diameter) in the total soil. It may be seen that pumping did and did not occur on some soils having a combined sand and gravel fraction of less than 55 per cent. Many of these non-pumping soils having less than 55 per cent sand and gravel, had relatively high silt content and low clay content. These soils permit more rapid movement of water away from leaky joints and

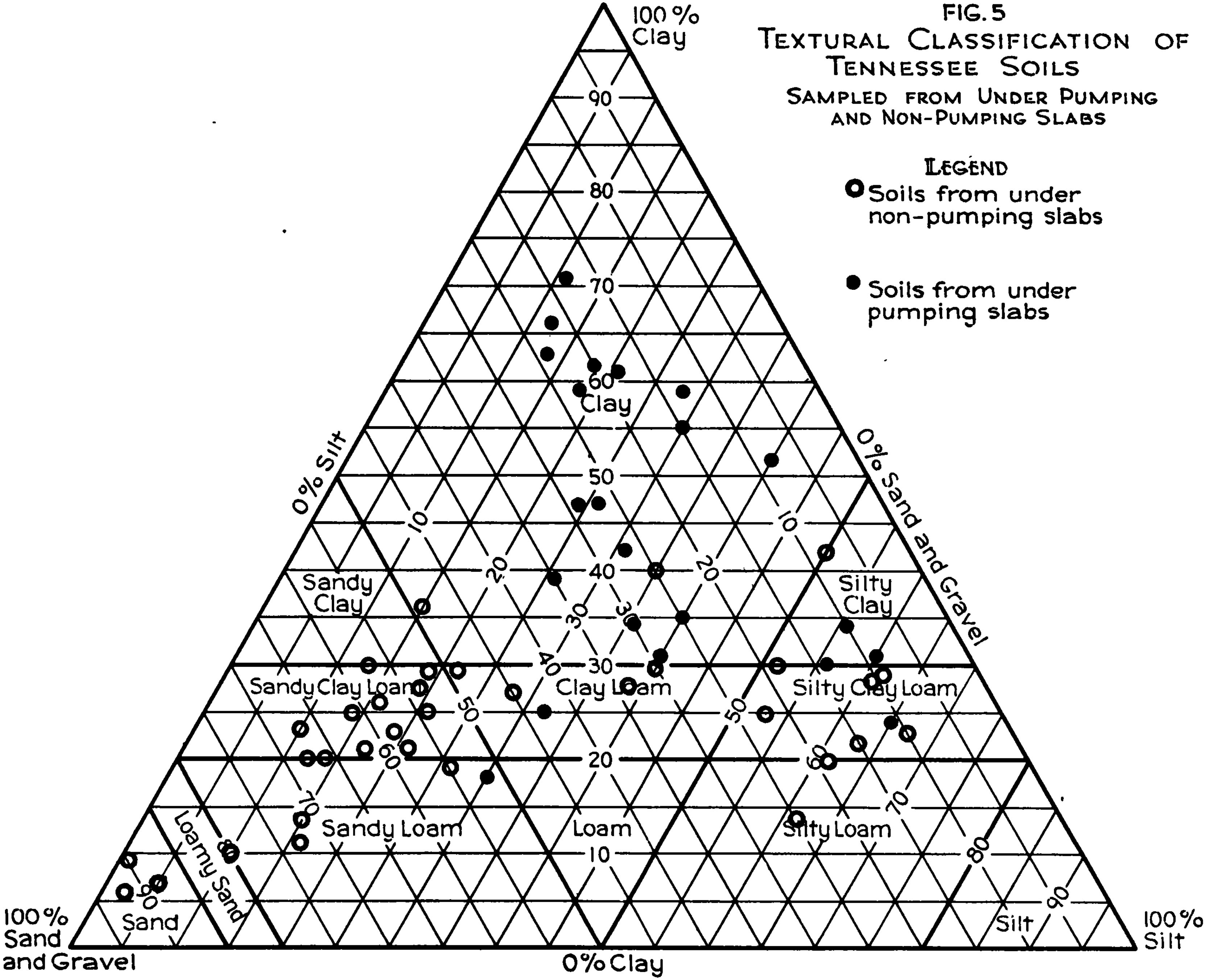

FIG. 5
TEXTURAL CLASSIFICATION OF
TENNESSEE SOILS
SAMPLED FROM UNDER PUMPING
AND NON-PUMPING SLABS

TABLE 8

Triaxial Compression Test Results

Loc. No.	Pumping: Max. P_v-P_h (Kips /.s.f.) P_h=0	P_h=2	Pumping: Red. in height (Per Cent) P_h=0	P_h=2	Pumping: ø (deg.)	Non-Pumping: Max. P_v-P_h (Kips /.s.f.) P_h=0	P_h=2	Non-Pumping: Red. in height (Per Cent) P_h=0	P_h=2	Non-Pumping: ø (deg.)
1	3.9	5.0	2.4	1.6	12					
5	4.4	4.4	6.0	4.8	0					
7						3.0	3.8	2.3	2.5	9
8						2.3	2.6	0.8	1.1	5
9						1.3	2.6	1.5	2.0	14
10	2.1	3.0	5.0	1.5	10.5					
11						4.0	4.0	0.5	0.6	0
12						8.8	-	1.0	-	-
14	1.0	-	0.9	-	-					
26	3.0	2.9	1.2	1.7	0					
27	1.0	1.0	3.0	4.0	0					
29	5.2	3.0	1.5	2.5	0					
30	2.0	3.3	4.5	4.5	14.5					
31	0.8	3.2	0.9	12.5	21.5					

cracks in the pavement and thus either prevent pumping or limit its occurrence to prolonged periods of wet weather. Such soils have been observed to be wet weather pumpers and do not pump during periods of infrequent rains as do the more plastic and more impervious clay soils.

Soil Water Content and Density

Soil water content determinations were made at 14 locations where pumping occurred and 15 where no pumping occurred. The results of soil water content determinations are shown in Table 6. Water contents of less than the plastic limit were found in only two of the 14 locations where pumping occurred. The major portion of the pumping locations had water contents of 3 to 5 percentage points greater than their corresponding plastic limits. The overall average water content exceeded the average plastic limit by 3.

Five of the eleven tests of plastic soil from under non-pumping slabs had water contents below their plastic limits and six had water contents 1 to 4 percentage points greater than their plastic limits. Average water content for plastic non-pumping soils immediately under the pavement was 19.4 per cent compared to an average plastic limit of 18.9 per cent.

Density and water content tests showed that in nearly every instance the soil was at or near saturation. Thus, the higher densities shown in Table 6 for non-pumping plastic soils limited the water holding capacity of the soils and prevented the absorption of sufficient water to soften the soil, reduce its bearing capacity, permit greater slab deflections and start pumping.

The variation in soil structure and compactness of different horizons in the soil profile had little or no influence on pumping in some soils and much in others. Pumping was often equally severe in the B and C horizons of soils derived from limestones and cherty limestones. However, in soils formed from loess, the more compact and less perfectly drained B horizon of the Loring series accounted for a major portion of the pumping found on those soils. The soils of the Muskingum and Hartsells series formed from sandstones and shales sometimes had compact clay strata underlying more pervious sandy soil. The less pervious compact clay strata restricted drainage, and caused the clay soils to become saturated and pump badly where the clay strata intersected the grade.

The Effect of Subgrade Treatments

Subgrade treatments consisting of 2 to 4 in. of loose sand mixed with the existing clay subgrades to 4 to 8 in. compacted depths were employed on three projects included in the survey. Detail study sections were included on 2 of the 3 projects. The natural subgrade soils on all three projects consisted of heavy clay soils derived from cherty limestones and are representative of the worst pumping soils in the State.

Some pumping occurred on all three projects. However, the soil samples taken during the detail study showed that where the sand and gravel content (portion having particles larger than 0.05 mm. diameter) in the treated subgrade soil was greater than 55 per cent, pumping did not occur. Two of these projects carry a moderate number of heavy axle loads. The third carried the greatest number of heavy axle loads encountered in the State.

There was evidence on some projects that the use of applications of sand-gravel to provide temporary surfacing for local traffic during the period between grading and paving had been effective in materially reducing pumping.

The Effect of Pavement and Shoulder Maintenance

Inasmuch as very little mudjacking as a maintenance operation to fill cavities under pumping slab ends had been done in Tennessee at the time of this survey observations could not be made regarding its effectiveness.

It has been mentioned previously that pumping on heavy clay soils occurred where shoulders were well sloped and permitted rapid runoff of surface water. However, it was observed that often the most severe pumping occurred in cuts and at the bottom of hills at low points in the grade where surface drainage was poor. Likewise, on borderline soils, that is, soils which pump only during prolonged periods of wet weather, pumping occurred only when associated with poor surface drainage due to high shoulders or to ruts in shoulders at the edge of the slab.

In summary, the observations showed that good maintenance of joints, cracks and surface drainage either prevented pumping of materially reduced the progress of pumping.

Summary and Correlation of Data

The data and observations indicate that methods and designs for preventing pumping on new construction can be grouped into two categories. First, a group of minor factors which tend to reduce or minimize pumping and second, a group of major factors to prevent the occurrence of pumping.

The first group includes the following factors:

1. The design of pavement and shoulders to obtain rapid runoff of surface water.

2. The use of minimum provisions for expansion or a maximum spacing of expansion joints of a given width. This tends to keep the pavement in restraint and reduces the amount of opening of intermediate contraction joints or cracks.

3. The use of a pavement thickness adequate for prevailing wheel loads.

4. The provision for load transferance at transverse joints to prevent faulting.

5. The control of water content and density of the subgrade during construction to obtain the greatest supporting value which the soil can be expected to maintain under service conditions.

6. Close attention to surface maintenance of the pavement and shoulders to reduce the entrance of water to the subgrade.

The data and observations pertaining to this first group are not conclusive but they do give positive evidence that all of the above factors will tend to reduce pumping. The observations on loess soils showed that pumping was often associated with poor surface drainage and that no pumping occurred on contiguous slabs on similar soils where the surface drainage was good.

The longest spacing of expansion joints was associated with less pumping. Pavements having load transferance at joints and carrying a substantially greater volume of heavy axle loads than did pavements with no load transfer devices showed approximately the same percentage of pumping joints as was found on pavements without provisions for load transfer. It is significant however, that the amount of Class 2 and Class 3 pumping at tranverse joints was less where load transferance was provided. Where "frozen" dowels restricted normal slab movements, transverse cracks showed greater pumping than found on pavements where the dowels functioned properly or on pavements built without dowels.

The results of observations on the beneficial effects of timely and good surface maintenance have been mentioned previously.

The close correlation between soil water content and pumping indicated that compaction with control of water content is of considerable value in reducing pumping. It is known that heavy clay soils having high volume change, although compacted to high densities, may swell and eventually absorb water in excess of their plastic limits. For such soils compaction with water content control may be of temporary value. However, for some soils having moderate volume change which will not swell and absorb water in excess of their plastic limits, the data indicate that compaction prevented the occurrence of pumping on those soils.

The second group includes the following two major factors which prevent the occurrence of pumping:

1. The use of a natural subgrade soil, an admixture of sand with natural soil or subbases of granular material which will not erode readily under normal expected movements of the slab.

2. The compaction of the better graded friable soils which do not erode readily to provide the maximum supporting value for the slab, which the soils can be expected to maintain.

Soils having more than 55 per cent retained on the No. 270 sieve have prevented pumping under the conditions found in Tennessee at the time of the survey. The above statement is based on normal conditions of drainage found on the projects studied and dees not pertain to locations where drainage conditions permit abnormal accumulation of water. The statement likewise pertains to the combinations of pavement thickness and traffic loads which prevailed.

It should be noted from the mechanical analysis of the soils that most of the sandy soils encountered were fairly well graded. It is possible that very poorly graded sandy soils having a large fine sand fraction might pump under heavy traffic and abnormal drainage conditions.

The data regarding thickness of subbases and subgrade treatments over pumping soils are not extensive. However, the thicknesses of treatment used, which consisted of mixing sand with the natural soils subgrades to form compacted depths of four to six in. (in one instance 8 in.) were effective in preventing pumping where the total soil mixture contained more than 55 per cent of material retained on the No. 270 sieve.

The data and observations indicated that adequate compaction was a major factor in preventing pumping on the better graded friable soils having relatively low volume change, by limiting the water holding capacity of those soils. Such soils were found to maintain good support at water contents near saturation.

APPENDIX

Wartime Road Problems

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No. 4

MAINTENANCE METHODS FOR PREVENTING AND CORRECTING THE PUMPING ACTION OF CONCRETE PAVEMENT SLABS

DEPARTMENT OF MAINTENANCE
W. H. ROOT, Chairman

Committee on Maintenance of Joints in Concrete Pavements as Related to the Pumping Action of the Slabs

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HIGHWAY RESEARCH BOARD
2101 Constitution Avenue, Washington, D. C.
October, 1942

MAINTENANCE METHODS FOR PREVENTING AND CORRECTING THE PUMPING ACTION OF CONCRETE PAVEMENT SLABS

What Is Pumping?

During the present emergency, Highway Maintenance Engineers are faced with the increasingly serious problem of maintaining existing pavements under a large increase in the number of heavy loads passing over them. The deflection of slab ends at joints and cracks under heavy loads after the accumulation of water in some subgrade soils causes a displacement and ejection of water carrying soil particles in suspension. This action is known as "pumping," and its continued repetition removes sufficient soil to result in lack of subgrade support.

Causes of Pumping

The pumping action of concrete slabs at joints, cracks and edges results from an unfavorable combination of soil type, water and amount and weight of traffic.

The subgrade soil type, where pumping occurs, consists of soils containing a predominance of silt and clay. Nonplastic granular soils such as sands and gravels are not susceptible to this action.

Free water must be present immediately under the joint, crack or edge of the pavement for pumping to occur. As a result, this action takes place during periods of prolonged rainfall such as may be expected normally in the spring and fall seasons. The source of free water is largely surface infiltration through cracks, joints and edges of the pavement but may be from water bearing strata or high water tables.

A large number of heavily loaded trucks, in combination with a susceptible soil type and water, is the final and most important factor in pumping. The deflection of a slab end is dependent upon the total load passing over a joint or crack at any given time. Thus, the load which governs the amount of the deflection is the axle load regardless of the number and size of the tires carrying the load. The present war effort has resulted in an increase in the number of heavily loaded vehicles, which in turn has increased both the severity of the pumping action and the extent of its occurrence.

PREVENTIVE AND CORRECTIVE PROCEDURES

The problem of maintenance in relation to pumping occurs in connection with two conditions:

Case 1. Pavements on which pumping may occur, due to the character of the subgrade and expected heavy loads.

Case 2. Pavements on which pumping has progressed to an appreciable degree, due to the character of the subgrade and the presence of heavy loads.

PREVENTION

In case 1 an examination should be made of the subgrade on any section of pavement which is being subjected to a greater than normal number of heavy loads. Such an examination should be made by an experienced soils engineer and all the soils on the section should be classified by visual inspection or by the testing of such samples as may be deemed necessary. Since examination of many pavements has indicated that pumping does not occur on subgrades composed of pervious sandy soils or gravels, special maintenance in areas having such subgrades may be reduced to a minimum. On sections on which the subgrade and shoulder materials are composed of types of soils that are conducive to pumping, measures to prevent pumping, such as the sealing of joints, care of surface drainage, and attention to drainage problems, should be taken.

The complete waterproofing of joints and cracks by the use of bituminous fillers has never been accomplished. Therefore, constant attention to the maintenance of a seal will be necessary if pumping is to be prevented by this method.

The use of French or other type drains through the shoulders of the road and into the ditches has been reported as only partially successful in stopping active pumping The use of them cannot be recommended except as a temporary preventive measure.

It has been observed that new shoulders sloped to drain storm water to the pavement instead of to the ditches, high shoulders on existing pavements or shoulders which are badly rutted, contribute to pumping. Therefore, the maintenance of smooth shoulders sloping toward the ditch will help to eliminate pumping. (This seems obvious but throughout the United States high and rutted shoulders on pavements are often overlooked.) Filling ruts and holes along the edges of pavements with pervious materials and the use of unsurfaced widening strips composed of pervious materials may divert the flow of water toward the pavement. These conditions contribute to pumping and such measures should be avoided in maintenance operations. All ruts, holes and trenches for widening materials should be filled with impervious material and sloped toward the ditch.

CORRECTION

In Case 2 the most successful method of treatment consists in forcing a suitable mixture under the slab. Several commonly used mixtures for this purpose are described later in the report. This procedure forces water out from under the slab and fills the void between the subgrade and the slab. Settled slabs may also be brought back to proper elevation by continuing the operation.

The materials, equipment and procedures described hereafter are typical of those used by several States in the filling of void spaces under pavement slabs by means of the mud jack.

MATERIALS

Soils

The type of soil used in mud jack mixtures must necessarily vary with the location in which the work is being done. There are, however, some soil characteristics which will make mud jacking more efficient. A suitable soil must be of such character that it will slake readily in water to form a mixture of uniform consistency and shall be reasonably free of organic material. It must be reasonably free from glue-like colloids which do not soften readily when mixed with water and are productive of objectionable packing of the mixture in the mud jack. One State reports that soil having low shrinkage and a small quantity of colloidal clay gives the best combination. The clay adds to the workability of the mix and keeps the coarse particles in suspension.

Mixtures

The kinds of mixtures and the proportions of soil and admixtures used in various States differ. All States reporting indicate satisfactory results with the mixture they use. The mixtures and the methods of preparation used by several States are presented here for information: The term "slurry" used in the following descriptions and elsewhere in the report refers to the soil-water-admixture combination having the consistency required for use in the mud jack.

Illinois

The slurry shall consist of a mixture of cement, limestone dust, slow curing liquid asphalt, soil, and water. The percentages of these ingredients by volume, exclusive of water, are approximately as follows:

Material	*Percent*
Cement	11
Limestone dust	11
Slow curing liquid asphalt (SC-2 grade).	19
Soil	59
Total	100

or, in terms of parts by volume:

Cement	1 part
Limestone dust	1 part
Slow curing liquid asphalt (SC-2 grade).	1 75 parts
Soil.. .	5.5 parts

NOTE: Due to the fact that slow-curing (SC) liquid asphalts are now unobtainable, it is suggested by V L Glover, Engineer of Materials, Illinois State Highway Department, that the following grade of medium curing liquid asphalt be substituted.

"*Asphalt MC-1* This material shall be a medium-curing cut-back asphalt consisting of a petroleum residuum fluxed with a suitable distillate. It shall be free from water, show no separation on standing and shall conform to the following requirements

(a) Flash point (Tag open cup), not less than	100°F
(b) Viscosity, Saybolt Furol, at 122°F, seconds	75 to 150*
(c) Distillation test	
Distillate, percent by volume of total distillate to 680°F.	
Distillate to 437°F., not more than	20
Distillate to 500°F	25 to 65
Distillate to 600°F	70 to 90
Residue from distillation to 680°F percent volume by difference, not less than	60
(d) Characteristics of residue from distillation test.	
(1) Penetration at 77°F, 100 g, 5 sec	150 to 300
(2) Ductility at 77°F (when the penetration at 77°F is between 150 and 200), not less than	100 cm.
(3) Ductility in centimeters, at 39 2°F, rate ½ cm per minute, not less than 1/5 of the penetration at 77°F	
(4) Bitumen soluble in carbon disulphide, not less than	99 5%
(5) Spot test	Negative

The amount of water used shall be varied in accordance with the moisture content of the soil and the consistency desired. Generally, 3.25 parts of water will be sufficient.

Silty clay or clay loam top soils are the most suitable. The clay adds workability; the sand reduces the shrinkage and increases the stability of the mixture; the oil is the waterproofing agent; the cement increases stability, helps to absorb the water, reduces the shrinkage, and accelerates set; the limestone dust accomplishes the same purposes as the cement although to a lesser degree, reduces the amount of cement required, and aids in holding the oil in the mixture.

A small trial batch should be made to determine the amount of water required to produce the proper consistency, and also to determine the amount of cement and limestone dust needed to obtain the necessary initial set and stability. The amounts of these three ingredients will vary with the type of soil used and the purpose for which the mixture is to be used; that is, whether it is to be used for lifting the slab back to grade or filling the voids under the slab caused by the pumping action.

In preparing the mixture, part of the water and the soil should be mixed first, then the liquid asphalt added, and finally the cement, limestone dust and such additional water as may be needed for proper consistency and to accomplish the purpose for which the mixture is to be used.

*The viscosity used should be from 100 to 150

Kansas

The mixture used is made by combining 8 parts by volume of cement, 5 to 8 parts of MC-2 or MC-3 cutback asphalt and 84 to 87 parts of soil with sufficient water to produce a consistency about like very thick buttermilk. The mixing is done in a small concrete mixer.

Missouri

When mud jack work for the prevention of the pumping action was started in 1937, 1½ sacks of cement per cubic yard of earth were used. For recent work more satisfactory results have been obtained with a mixture containing 4 sacks of cement per cubic yard of soil.

Ohio

On one project Ohio reports the following:

During November and December of last year the French drains were supplemented by pavement jacking at all joints where pumping was observed. Various mix proportions were used and all of the mixes substantially reduced the amount of pumping.

Mix 1 consisted of 50 percent SC-2 and 50 per cent powdered asphalt by weight. For the weather conditions at the time this work was done a temperature of about 250 degrees F. was necessary.

Mix 2 consisted of soil, powdered asphalt and SC-3. This material has extruded from the joints and tracked the pavement badly.

Mix 3 consisted of 67 per cent soil, 25 per cent SC-3, 4.5 per cent cement and 3.5 per cent powdered asphalt by weight with sufficient water to make the mixture workable in the mud jack. This was the first mix to be tried and it was gradually changed by lowering the percentage of soil, cement and water until mix 1 was attained. Mix 2 was essentially a stage in this transition.

Study of the performance of the various treatments used indicates that the mixture of SC-2 and powdered asphalt alone, has given the most satisfactory results. However, all of the mixes tried have resulted in considerable improvement of the pavement when compared to its condition in the fall of 1941.

NOTE: In view of the fact that slow curing (SC) liquid asphalts are now unobtainable Mr. Charles W Allen, Acting Chief Engineer, Bureau of Tests, Ohio State Department of Highways, reports that Ohio intends to experiment with the lighter consistency medium curing (MC) liquid asphalts and road tars.

Admixtures

The reports received indicate that the choice of an admixture for use with soil is a matter to be left to the judgment of the engineer. The quantity of cement used varies from 0 to 5 sacks per cubic yard. The bituminous material, where used, may be slow curing asphaltic oil or medium curing cutback asphalt and the quantity varies from 8 to 25 per cent by volume.

The final mixture of soil and cement or soil-bituminous material and cement must have low shrinkage when dried and must flow at a relatively heavy consistency.

Testing Soil and Soil-mixtures

Laboratory tests should be utilized to measure the shrinkage and hardening characteristics of the soil and soil-mixtures. In an emergency and as a routine field test, the shrinkage may be judged by mixing a properly proportioned batch of the materials to the consistency desired by the mud-jack operator and molding parts of the mixture in the top of ice cream cartons or other flat containers and drying in the sun or any other available heat source. The entire absence of shrinkage or the occurrence of a very small smount of shrinkage will indicate a satisfactory mixture. Appreciable shrinkage will indicate the necessity for a change in soil or the use of an additional quantity of admixture.

Hardening may be judged by means of similar samples buried at the edge of the slab for 24 hours.

RECOMMENDED FIELD PROCEDURES

The following procedure is typical of that being followed by most States and is recommended by the Committee. Such variations as local conditions may require are desirable and permissible.

Drilling

For the expulsion of water and for filling the voids under the slab a hole is drilled near the intersection of the transverse joint or crack with the center joint. The location of each hole should be approximately 10 in. from the center joint and 10 in. from the transverse joint or crack (Fig. 1). The hole should be located on the far side of the transverse joint or crack in the direction of traffic.

When the pumping action has caused settlement of the slab and it is desired to lift it back to grade, holes should be drilled 30 in. each way from the intersection of the transverse crack or joint with the outside edge of the slab (Fig. 1).

The location of both the void filling holes and the lifting holes shown in Figure 1 are for average conditions. Other positions of the holes may be desirable under some field conditions. In the case of 4-lane pavements, settlement almost always occurs in the outside lane which carries the bulk of the heavy traffic. The inside or passing lane seldom settles.

Filling Void Spaces

The first step in the void filling operation is to force all trapped water from under the slab by means of compressed air. This is necessary in order to avoid dilution of the soil mixture with water and to insure the filling of the void space. To allow the escape of water, each transverse joint should be vented at the outside edge of the pavement by digging away the shoulder material. Where there is evidence of considerable water under the slab, it is sometimes necessary to strip short sections of the bituminous filler from the joints to allow free escape of the water.

After the water has been forced out the slurry is pumped into the voids until it appears in the cracks and in the vents at the edge of the pavement. In some cases, it is advisable to admit shots of air between shots of slurry in order to force the mix the maximum possible distance.

When the slab is not to be lifted, pumping is stopped when the slurry appears at cracks, joints or the edges of the pavement or when the slab starts to lift. In lifting slabs, the vents at the edges of the pavement are closed by filling with dirt, and, if necessary, wooden plugs are driven into adjacent holes and clay is tamped into leaky joints. Slurry is then pumped in the lifting hole until the slab is raised to the desired elevation.

After all the void filling and lifting operations have been completed, the holes are sealed with clay. Later the holes should be cleaned out and sealed with bituminous filler.

The successful filling of void spaces under concrete pavements or the lifting of depressed slab ends to their original elevation by means of the mud-jack requires careful engineering supervision and skillful operation of equipment. It is the opinion of the Committee that the entire attention of one experienced engineer will be required during the operation of each mud-jack unit and that operators of equipment should be carefully trained before being assigned to work on heavily traveled roads. The use of the mud-jack without adequate supervision or with inexperienced operators is likely to result in failure to stop pumping at slab ends and may result in damage to the pavement.

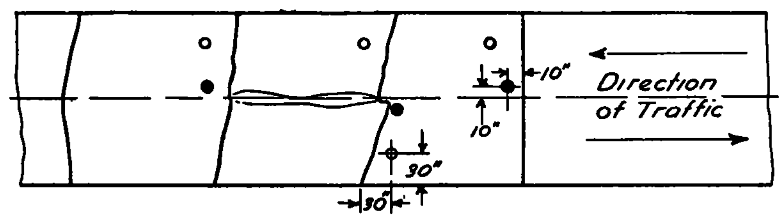

FIG. 1—Typical Locations of Void Filling and Lifting Holes at Transverse Joints and Cracks ●, Void filling hole; ○, lifting hole.

Equipment

For ordinary operations, the standard mudjacking equipment, which most of the highway departments already have may be used for this work. This consists of:

- No. 50 mudjack
- 2½-in. mud-jack bit
- Air compressor
- 28- to 35-lb. jack hammer

It is possible to obtain a reducing

nozzle with a 1¼ in. opening, which will allow smaller holes to be used.

A smaller machine, which is especially adapted to this work is also available. The equipment required is:

- No. 10 mud-jack
- Smaller mixer
- Air compressor
- 1¼-in mud-jack bit
- 28- to 35-lb. jack hammer

The mixer may be a concrete mixer or a bituminous mixer, preferably the latter. Either type may vary from 5 to 10 cu. ft. in capacity

For more extensive operations, equipment and methods similar to those described in the November 1940 issue of Roads and Streets under the title "Subgrade Treatment by Mud-jacking and Filling" may be used.

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WARTIME ROAD PROBLEMS

No 1. Curing Concrete Pavements Under Wartime Restrictions on Critical Materials

No 2 Design of Highway Guards

No 3 Design of Concrete Pavements Requiring a Minimum of Steel.

No 4 Maintenance Methods for Preventing and Correcting the Pumping Action of Concrete Pavement Slabs.

No 5. Granular Stabilized Roads

No 6 Patching Concrete Pavements with Concrete.

No 7. Use of Soil-Cement Mixtures for Base Courses

No. 8 Thickness of Flexible Pavements for Highway Loads.

No 9. Treatment of Icy Pavements

No. 10 Salvaging Old High Type Flexible Pavements.

No. 11. Compaction of Subgrades and Embankments.

IN PREPARATION

Soil-Bituminous Roads

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