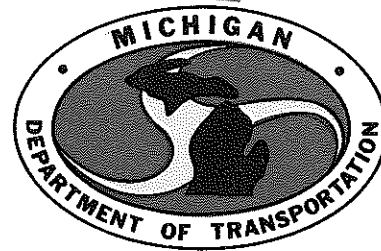


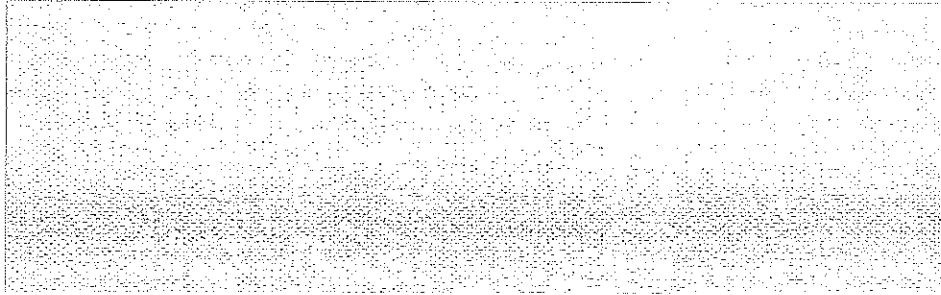
CONCRETE PAVEMENT PERFORMANCE  
PROBLEMS AND FOUNDATION  
INVESTIGATION OF I 75 FROM THE  
OHIO LINE NORTHERLY TO THE HURON RIVER



**TESTING AND RESEARCH DIVISION  
RESEARCH LABORATORY SECTION**



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TE278 .S55 1981 c. 2  
Concrete pavement  
performance problems and  
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I 75 from the Ohio line

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J. E. Simonsen  
E. C. Novak, Jr.

Research Laboratory Section  
Testing and Research Division  
Research Project 80 TI-669  
Research Report No. R-1171

Michigan Transportation Commission  
Hannes Meyers, Jr., Chairman; Carl V. Pellonpaa,  
Vice-Chairman; Weston E. Vivian, Rodger D. Young,  
Lawrence C. Patrick, Jr., William C. Marshall  
John P. Woodford, Director  
Lansing, June 1981

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I  
PAVEMENT PERFORMANCE

J. E. Simonsen

This is a follow-up to a June 23, 1980 memo to K. A. Allemeier from T. A. Coleman and J. E. Simonsen which reported on a cursory inspection of the subject pavement and recommended additional work in an attempt to find the causes of the performance problems (cracking and faulting of the slabs). These problems are confined to the outside (traffic) lane which was added four to seven years ago to the 25-year old two-lane pavement.

This portion of the report deals with the results of surveys conducted to establish the extent of the cracking and to determine if tying the new lane to an existing one and using concrete shoulders, have influenced the formation of the cracking. Also, concrete cores were taken to check the concrete strength and to examine the condition of the steel reinforcement.

Work to determine the performance of the pavement foundation was assigned to the Soils and Pavement Layer Analysis Group, and is included in the second portion of the report entitled, "Foundation Investigation."

The portion of I 75 involved is 24.9 miles long and consists of the following seven construction projects:

Code No.	Project No.	Length, miles	Construction Year
1	I 58151, 09678A	1.6	1976
2	I 58151, 08413A	1.6	1976
3	I 58151, 03867A	4.9	1975
4	I 58151, 03869A	5.2	1975
5	I 58151, 07685A	2.6	1976
6	I 58152, 00797A	3.0	1976
7	I 58152, 04472A	6.0	1973

Code numbers are assigned from south to north and Project No. 1 begins at the Ohio Line.

The original two-lane 10-in. reinforced concrete pavement was placed on a 16-in. sand subbase and the shoulders consisted of aggregate material with a bituminous surface. A review of the condition surveys indicate that

after five years' service the formation of transverse cracks had progressed at a normal pace, except for Project 4, where the number of transverse cracks was in the range for that of 15-year old pavements. Although this spread in performance is now less discernible the unusually large number of cracks that occurred on Project 4 so soon after construction indicates that future problems could develop. In this respect, our records of repairs with fast-set concrete show that more than four times as many sq yd/mile of freeway pavement have been replaced on Project 4 than on any of the other projects.

The added 10-in. reinforced lane was designed to be placed on a 12-in. sand subbase and a 4-in. aggregate base. The joints are spaced 99 ft apart to match the joints in the two inner lanes. The lane is tied to the old slab using hookbolt sleeves which were cast into the existing slab when it was built. The shoulders on Projects 1 through 6 consist of non-reinforced concrete 11 ft wide. The shoulder thickness tapers from 10 in. at the pavement edge to 6 in. in 3 ft, with the remaining 8 ft being of 6 in. uniform thickness. Four different shoulder construction methods were used:

1) Cast integrally with lane, no intermediate joints, longitudinal joint sawed at normal pavement-shoulder joint interface.

2) Cast integrally with lane, no intermediate joints, longitudinal pavement-shoulder joint sawed at the end point of the shoulder taper.

3) Cast integrally with lane, four intermediate joints per 99-ft slab, longitudinal pavement-shoulder joint sawed at normal location.

4) Cast separately from lane, four intermediate joints per 99-ft slab, bulkhead type longitudinal joint.

On Project 7 the shoulders are full-depth bituminous.

On September 11, seven concrete cores were removed from the pavement lane on the northbound roadway in the vicinity of Station 522 (Project 4) where county maintenance forces were repairing distressed areas. Three 6-in. cores were taken through cracks faulted from 1/4 to 3/8 in. for the purpose of examining the reinforcement and four 4-in. cores were taken for determining the concrete strength.

The compressive strength of each of the four cores was 5,600, 6,000, 6,700, and 5,800 psi, respectively. This is well above the 3,500 psi compressive strength for which the mix was designed and indicates that concrete strength was not a factor in the crack formation.

The reinforcement in the cores taken through the cracks had fractured. Although the steel wires showed some sign of corrosion it appears the failures are caused by bending stresses induced by traffic load. Examination of the cores shows that aggregate interlock to transfer the load across the cracks was lost, which had resulted in concrete bearing failures under the wires. Once the concrete fails in this manner, the wires are flexed every time a load passes over the crack, which induces bending stresses in the wires, ultimately causing them to fracture.

A pavement condition survey was conducted in the summer of 1980 on the new lane for Projects 1 through 6. Information for Project 7 was obtained from a condition survey made in 1978. The surveys recorded all transverse cracks and, for Projects 1 through 6, the estimated amount of faulting on cracks with this deficiency was also noted.

The distribution of the average number of transverse cracks per slab in the added lane for the seven projects is illustrated in Figure 1. The average varied from 3.3 to 8.3 cracks per slab on the northbound lane and from 3.7 to 7.3 on the southbound lane. The range in cracks per slab was 0 to 11 on the southbound lane and 0 to 13 on the northbound lane. In Research Report R-711, "Performance of Michigan's Postwar Concrete Pavements," it is reported that for 82 two-lane projects the average number of transverse cracks per slab varied from 0.03 to 5.17 after 10 years' service. Figure 1 shows that on five of the seven projects the average number of cracks per slab was more than the highest average (5.17) found on 10-year old projects, even though these projects are only about five years old. On the basis of this comparison, it is evident that more cracks than are normally expected have occurred in the added lane.

Generally, transverse cracks do not have any serious effect on pavement performance unless the steel fractures and faulting develops. Once the steel fails the cracks act as a joint and incompressible materials enter the cracks during contraction cycles which increase the compressive forces in the slab during expansion periods. Faulting can occur when aggregate interlock is lost at the cracks but this normally does not affect the performance before the pavements are 10 to 15 years old.

On the added lane of I 75, the number of cracks with visible faulting was recorded. A distribution of the average number of faulted cracks per slab is shown in Figure 2 for both roadways. The faulting problem, as can be seen, is most severe on the northbound roadway. Among the projects, No. 4 on the northbound roadway is the most severely faulted, with an average of slightly more than four faulted cracks per slab.

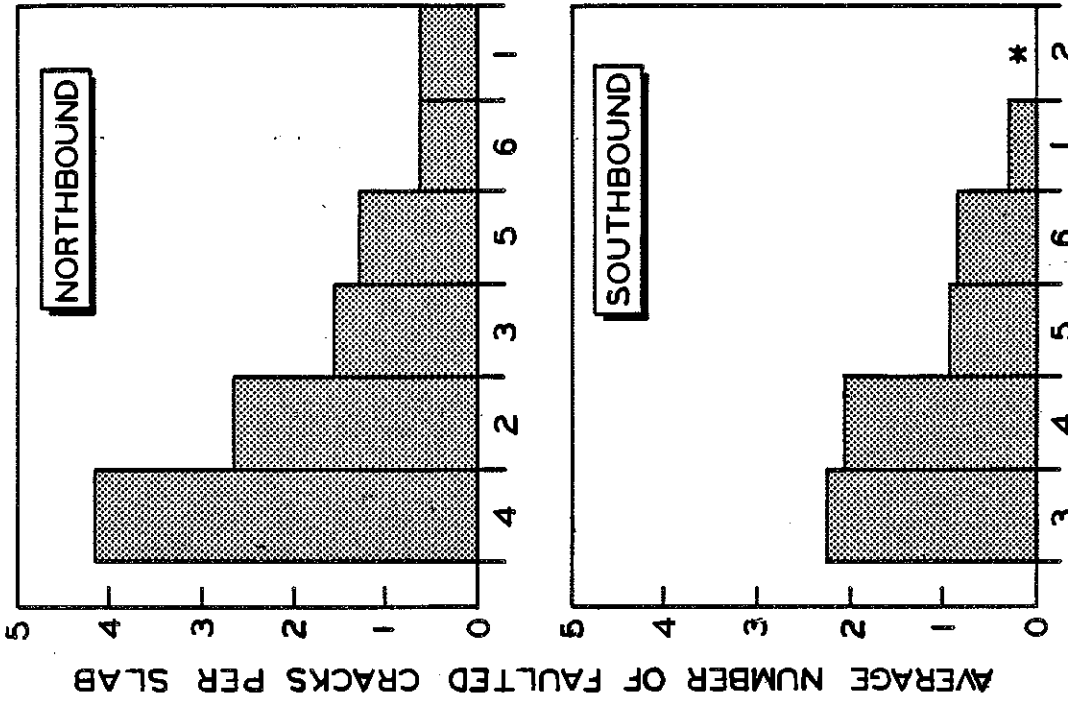


Figure 1. Average number of transverse cracks per 99-ft slab in four to seven year old traffic lane for seven projects.

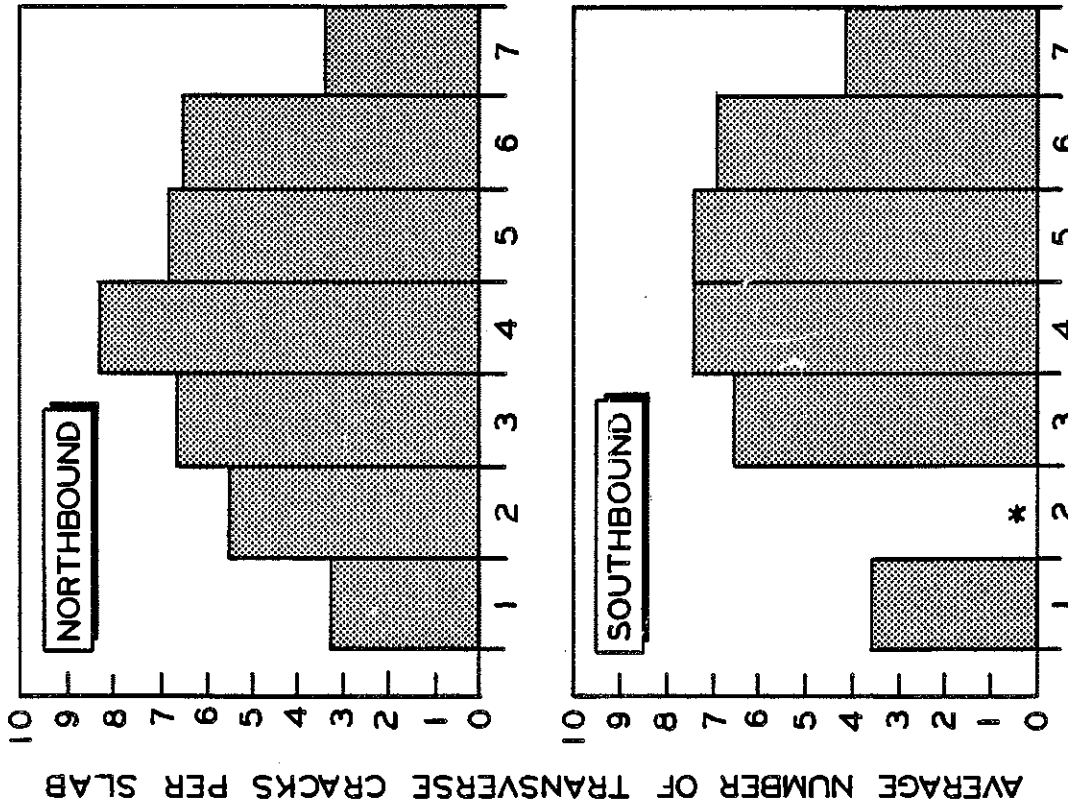


Figure 2. Average number of faulted cracks per 99-ft slab in four to seven year old traffic lane on six projects. (Projects arranged in order of most cracks per slab from left to right.)

NOTE: \* DATA NOT INCLUDED BECAUSE MOST OF THE SOUTHBOUND ROADWAY WAS REPLACED WITH 71 FT. SLAB PAVEMENT IN 1976.



As previously discussed, it was found that the steel had fractured in cores taken through cracks faulted 1/4 to 3/8 in. From the survey data, the number of cracks with an estimated 1/4 to 1/2 in. of faulting has been tabulated for Projects 1 through 6 as follows:

Project No.	Faulted Cracks, 1/4 to 1/2 in. *
1	1
2	19
3	196
4	384
5	63
6	27

\* A total of 33 cracks were estimated to have faulted 1/2 to 1 in. and are included in the totals.

A special survey was conducted to study the relationship of cracking in the old and new lane and in the concrete shoulder. An equal number of 99-ft slabs (16) in each section with a different shoulder type were inspected. A systematic sampling plan with a random starting point was used. The cracks in the old and new lane and in the shoulder were drawn on a plan view of each selected location.

The average number of transverse cracks per 99-ft slab in the old and added lane and in the shoulder was tabulated and are shown in Table 1. As can be seen in all but one case, the average number of cracks in the old lane slabs is less than in the added lane and in the shoulder slabs. It should be noted that the inspected slabs for which the above condition is reversed are located on Project 1. This project also exhibits the least amount of faulting among the projects under study (Fig. 2). This should indicate that a better foundation support condition exists on Project 1, which may be the reason for less cracking in the added lane and shoulder. The section with the most cracking in the added lane is the one having bituminous shoulders. This may be because the lane was added two to three years earlier than on the other sections, but it indicates that the concrete shoulder on the subject portion of I 75 is not the major factor in the formation of cracks in the added lane.

A tabulation of the cracks in the old lane in line with cracks in the added lane revealed that about 60 percent of the cracks have a corresponding crack in the added lane. For the purpose of this tabulation, a crack was counted

TABLE 1  
SUMMARY OF TRANSVERSE CRACK SURVEY

Shoulder Type	Average Number of Transverse Cracks Per 99-ft Slab		
	Old Lane	Added Lane	Shoulder
Cast integrally with lane (no intermediate joints) <sup>1</sup>	6.1	6.4	6.3
Cast integrally with lane (no intermediate joints) <sup>2</sup>	6.3	7.4	7.4
Cast integrally with lane (four intermediate joints) <sup>3</sup>	5.5	3.8	4.6
Cast separately from lane (four intermediate joints) <sup>3</sup>	5.0	6.9	7.3
Bituminous shoulder (full depth)	4.8	8.5	---

<sup>1</sup> Longitudinal shoulder-pavement joint sawed at normal location.

<sup>2</sup> Longitudinal shoulder-pavement joint sawed at beginning point of shoulder-thickness taper.

<sup>3</sup> Sawed intermediate joints are included in the total number of cracks in shoulder.

as being in line with another one if they were less than one hookbolt spacing (40 in.) apart at the longitudinal joint. With respect to the formation of the remaining cracks in the added lane it is theorized that hookbolt restraint and traffic loading were the causes.

Past studies of transverse slab cracks have shown that more cracks occur in the traffic lane than in the passing lane. This is attributed to the fact that the traffic lane carries most of the heavy traffic. Since the new lane was added on the outside edge of the existing two-lane roadway, it became the traffic lane and carries some of the largest truck volumes in the state.

Observation of a repair on the added lane revealed that the cracks in the new concrete had formed between the hookbolts. Since these bolts restrain independent movement of the added lane it is possible that shrinkage

cracks could have formed between bolt locations during the early stage of concrete curing. In time, such cracks could have been extended across the slab by subsequent traffic loading.

On the basis of the results of the pavement condition surveys and testing of concrete cores removed from a distressed area on the added lane, it is concluded:

1) The concrete compressive strength is well above the design strength; thus, it does not appear to be a factor in the cause of the transverse cracking problem.

2) About 60 percent of the cracks in the added lane were continuations of existing cracks in the old lane. The remaining cracks are thought to be caused by hookbolt restraint and heavy load application.

3) Core and faulting data indicate that currently there are about 700 cracks on the subject pavement with fractured or yielded steel reinforcement.

4) Based on the average number of transverse cracks per slab and number of faulted cracks per slab, the northbound roadway exhibits more distress than the southbound roadway. The poorest performing section is Project 4, northbound roadway, located from milepost 8 to 13.

The amount of faulting experienced and fracturing of the steel indicate base support problems, which are addressed in Part II of this report.

## II FOUNDATION INVESTIGATION E. C. Novak, Jr.

The following are the results of a foundation study conducted in Monroe County on the northbound lanes of I 75 between exits 2 and 15. The purpose of this study was to determine if the foundation has detrimental characteristics that contribute to, or are responsible for, premature pavement failure and if so, what measures may be necessary to attenuate further deterioration.

### Pavement Description

The section of I 75 studied has the typical cross-section shown in Figure 3. The subgrade and subbase materials are the same under all three

lanes. The only differences between lanes is that the outside or add-on lane has an approximately 3-in. thick crushed limestone aggregate base and a subbase which is thinner by 3 in. The original lanes are constructed directly on the subbase. Therefore, the total thickness of original subbase and new subbase plus base is the same.

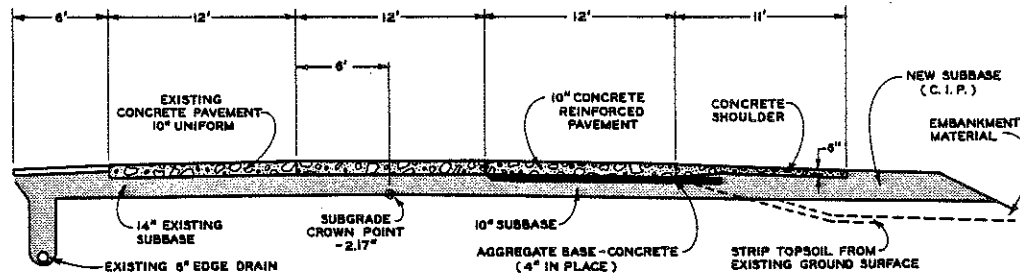


Figure 3. I 75 pavement cross-section, as designed.

The distress occurring in the add-on lane is illustrated in Figures 4 through 11 for the section under investigation (Project 4). Transverse cracking varied from a maximum of about 13 cracks per slab to a minimum of around 5. At some locations, broken slabs are being punched into the foundation; whereas, in other cases the broken slabs, although faulted, have the same general surface elevation as the original pavement.

### Investigation

During repair of two distressed areas, base and subbase samples were obtained for laboratory analysis. When the damaged concrete slabs were removed, it could be seen that pumping of fines from the base material was causing the pavement slabs to fault. When the pavement is distressed sufficiently to warrant slab replacement, typically only 15 to 20 ft of pavement are replaced. In sawing the distressed pavement for slab removal, the cut slabs move more freely causing truck traffic to bounce and impact-load the pavement in the direction of traffic. This accelerates the deterioration of adjacent down-traffic pavement. It was observed that the long time period between identifying distressed areas, sawing, and replacement caused the distress to extend beyond its original limits. A more timely identification replacement procedure should help to prevent enlargement of distressed areas.

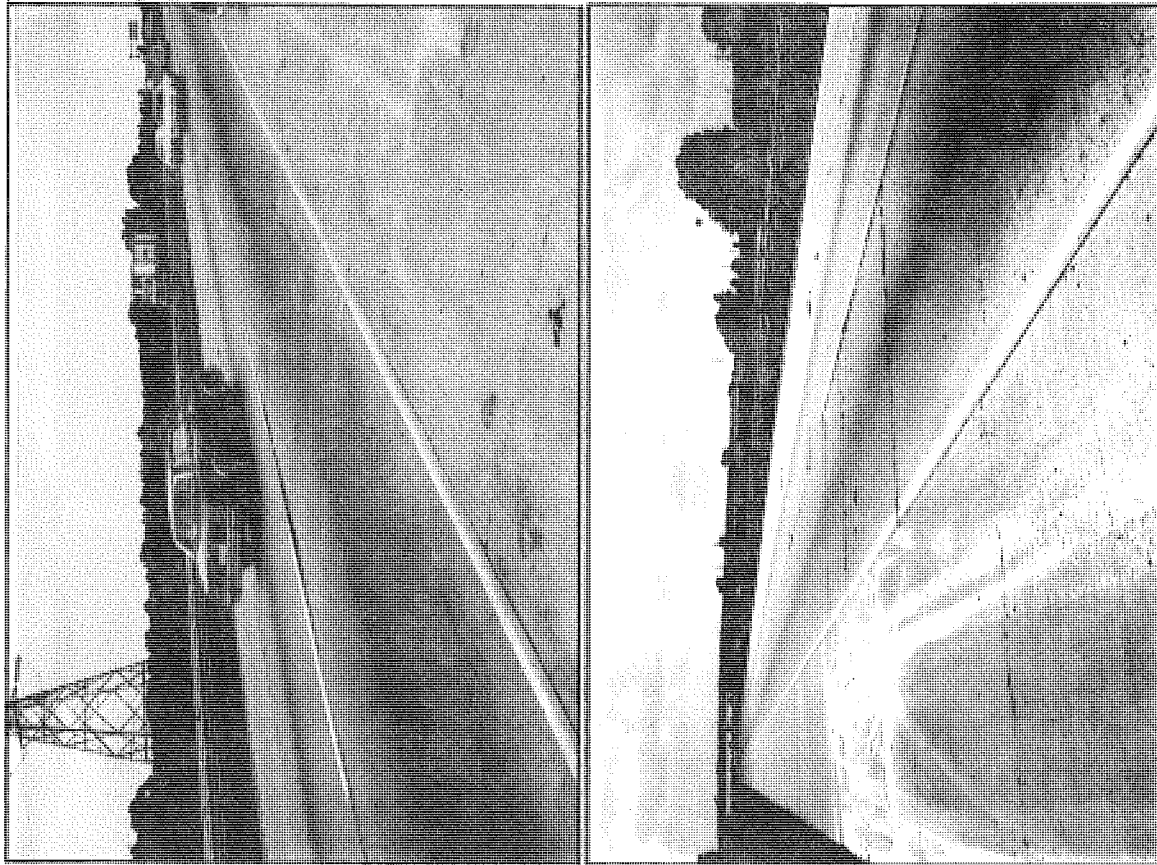


Figure 5. Typical distressed areas illustrating the maximum and minimum range of differential movement of the surface of the add-on lane compared to the adjacent pavement.



Figure 4. Typical distressed areas illustrating the maximum and minimum range of differential movement of the surface of the add-on lane compared to the adjacent shoulder.

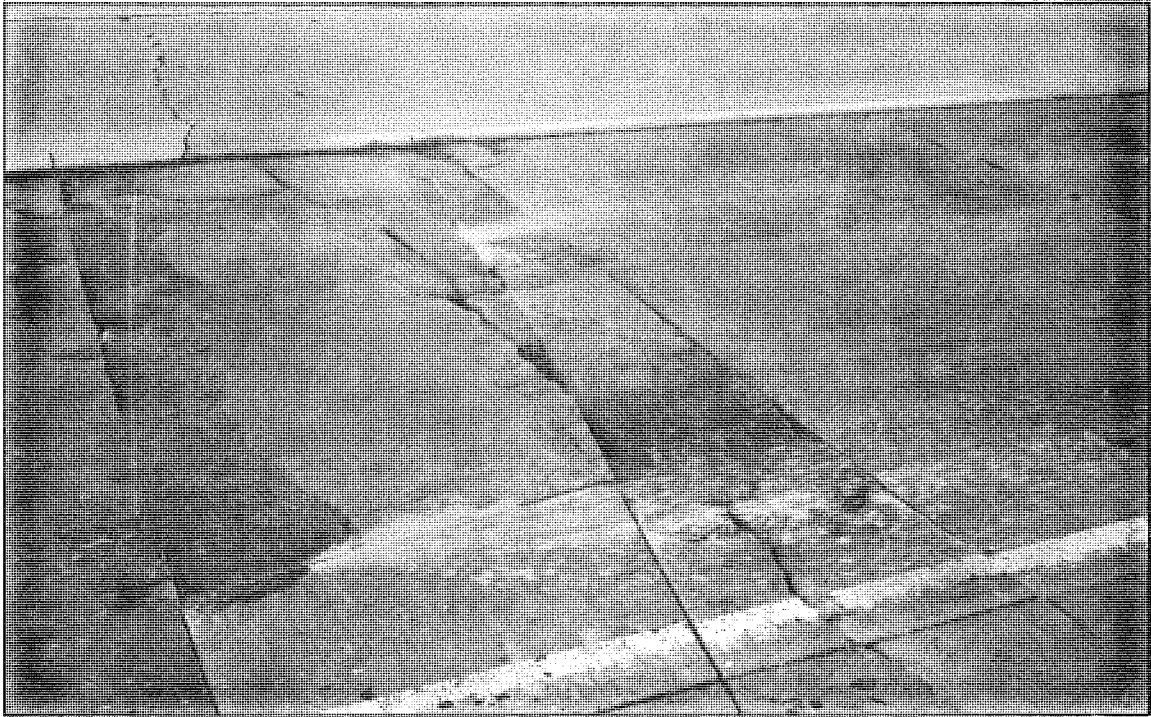


Figure 6. Severely distressed section of add-on pavement sawed and ready for repairs. Note pumping of base fines and that the surface of the add-on lane is depressed next to the original pavement but is approximately flush with the shoulder.

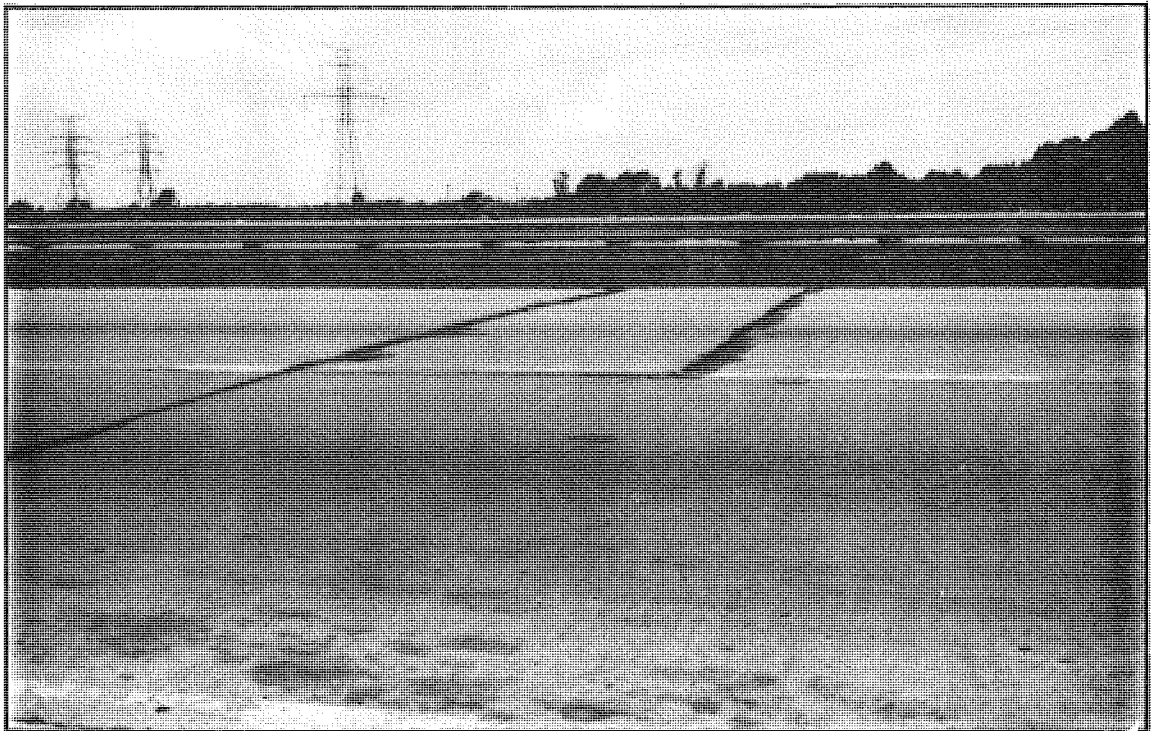


Figure 7. Faulting of joint repair slab in the original pavement while adjacent add-on lane is free of distress. Indicates a tendency for the sand subbase to cause faulting.



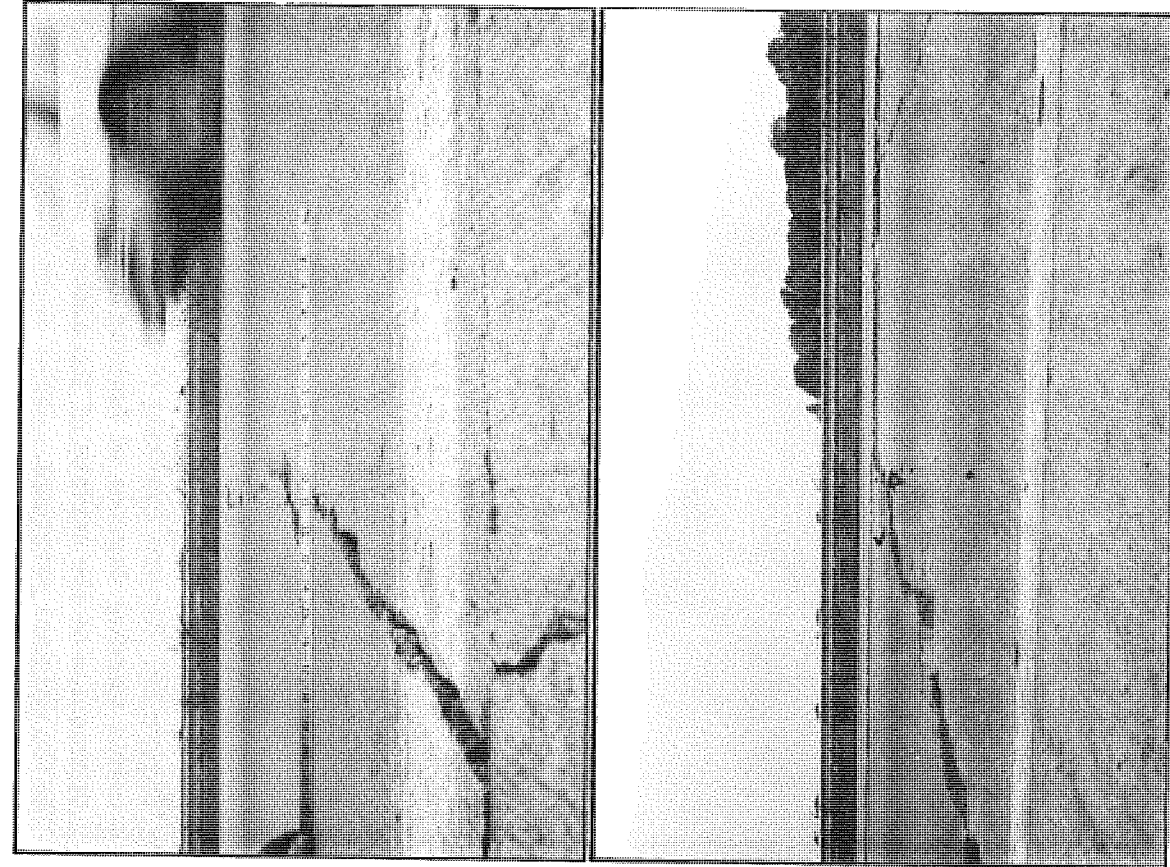


Figure 9. Faulting, even when severe, may or may not be associated with a depressing of the surface of the add-on lane relative to that of the original pavement.

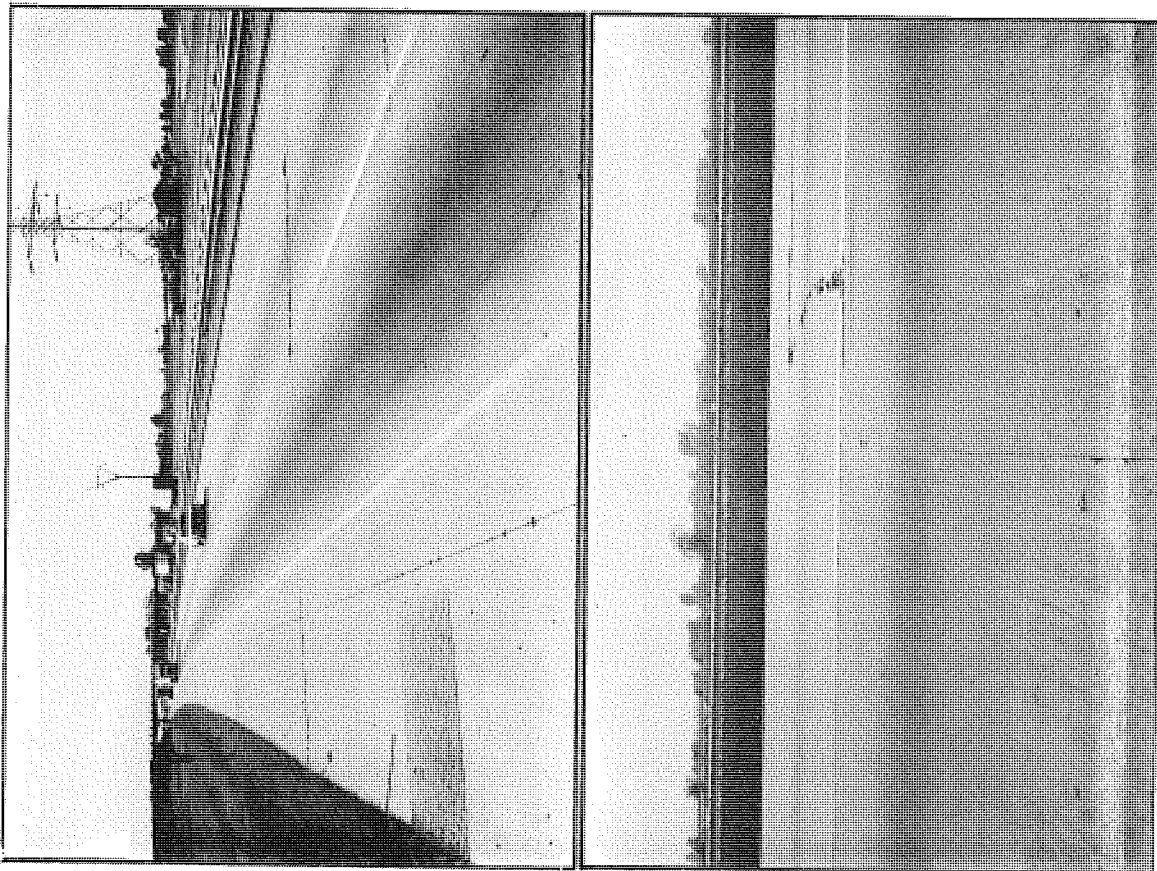


Figure 8. Typical transverse cracking in add-on lane where little or no faulting is occurring.

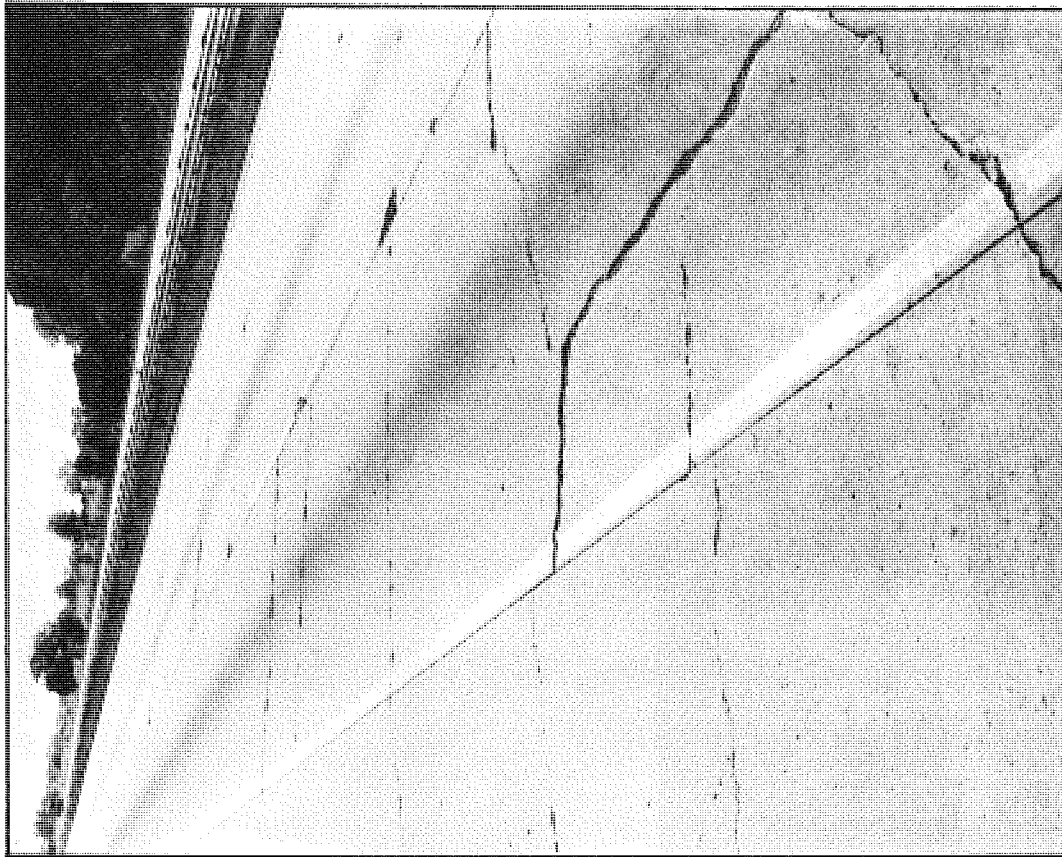


Figure 10. Severe cracking of the add-on lane indicating varying degrees of faulting and no associated depression of the surface of the add-on lane relative to the original pavement.

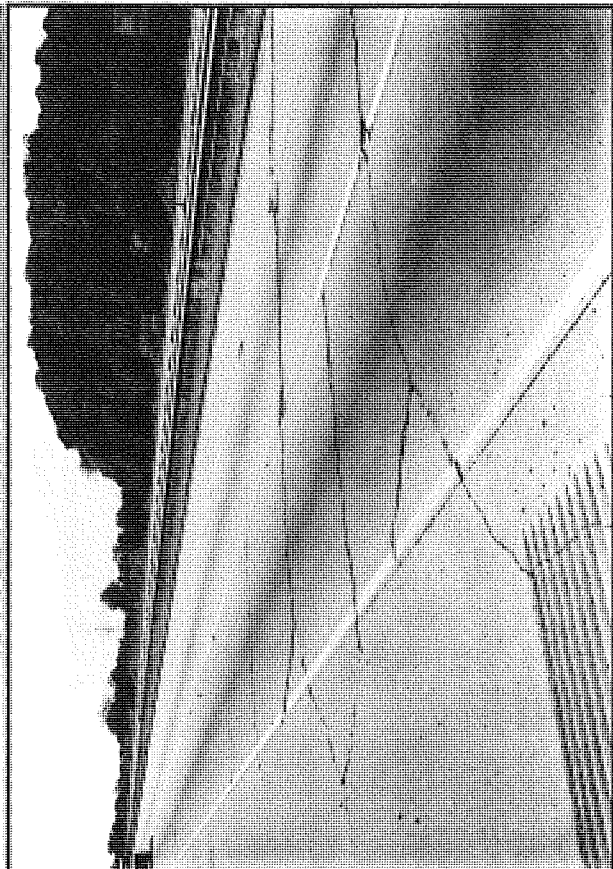
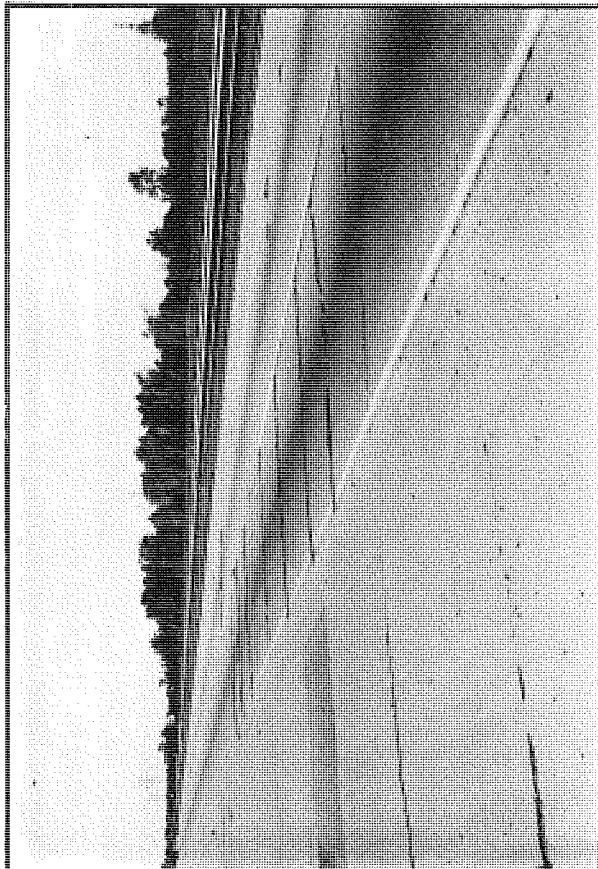






Figure 11. Moderate transverse cracking and faulting. Add-on lane is depressed compared to original pavement indicating there is no bond between them.

The as-built cross-sections from the outside lane to the ditch line of two sites selected for more detailed study are shown in Figures 12 and 13. Generally, the side slope area is covered with a thick layer of relatively impervious clayey soil. Thus, gravity water drainage is restricted at the side slope. It was beyond the scope of this study to classify foundation drainage conditions over all of the 13 miles of pavement investigated; however, differences do appear to exist in various sections of the project.

Base and subbase samples were tested for gradation, drainability and frost susceptibility, the results of which are summarized in Table 2 and Figure 14. Because the clayey subgrade soil was relatively dry and stiff, therefore in good condition, no samples were retained for laboratory analysis.

### Investigation Results

Base - The aggregate bases at the sites tested do not meet gradation specifications, having a loss-by-washing content of roughly 20 percent which is considerably higher than the 4 to 6 percent at placement indicated by the construction records. On the basis of these data, it is concluded that the

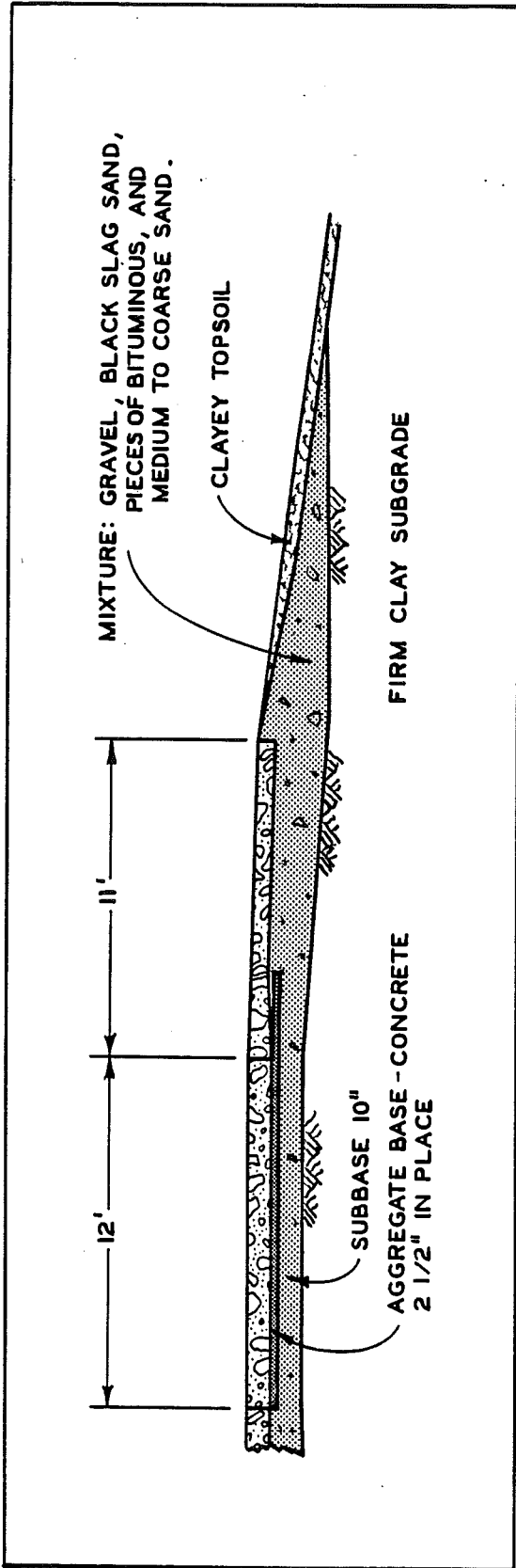


Figure 12. I 75 pavement cross-section at Station 320+25.

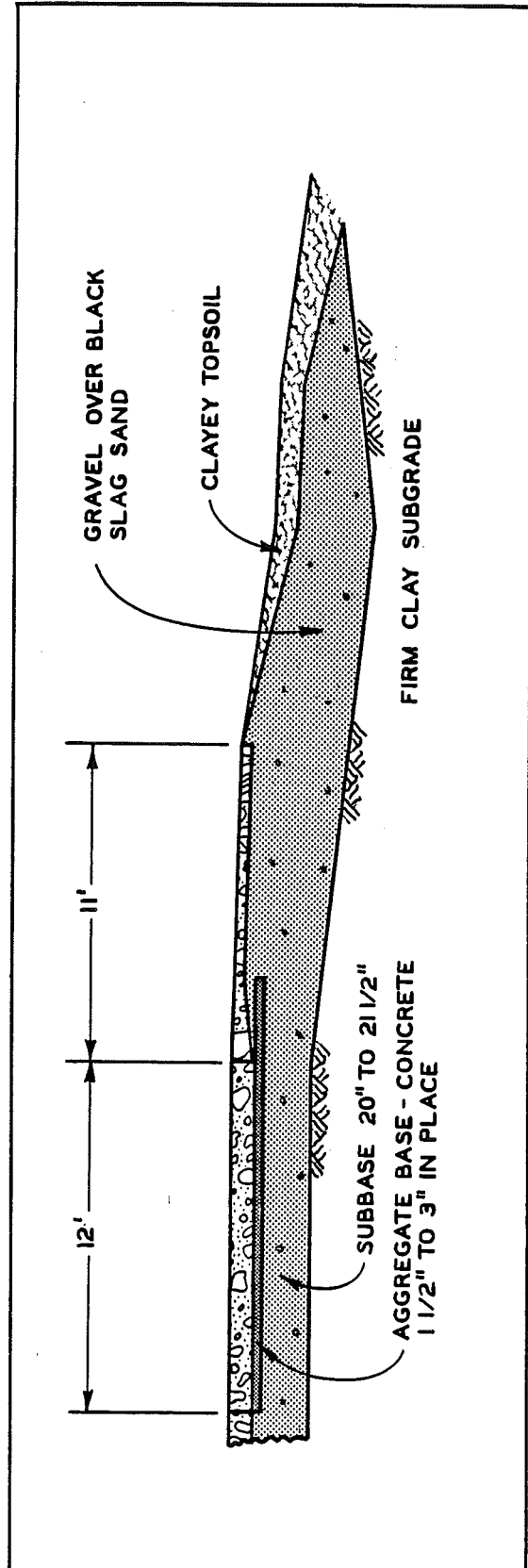


Figure 13. I 75 pavement cross-section at Station 512+15.

TABLE 2  
SUMMARY OF TEST RESULTS

Station	Test Type	Material	Dry Density, lb/cu ft	Moisture Content, percent	Permeability k, ft/day	Moisture Content Drained, percent	Drained Saturation, percent	Effective Porosity, Ne	k/Ne	Drain Time, days		Frost Susceptibility
										Original	Present	
320+25	Remolded	Base	140.6	6.9	0.50	7.4	100.0	--	--	--	--	High
320+25	Remolded <sup>1</sup>	Subbase	109.2	12.7	0.08	13.8	100.0	--	--	--	--	Low
320+25	Remolded <sup>2</sup>	Subbase	107.9	9.3	0.07	13.9	100.0	--	--	--	--	Low
320+25	In Situ	Subbase	111.3	13.6	0.07	16.7	87.4	0.04	18.0	7	23	---
320+25	In Situ	Subbase	108.3	13.3	0.08	17.9	88.5	0.04	20.0	6	20	---
513+15	Remolded	Base	136.1	5.8	0.80	7.7	89.8	0.02	36.0	--	--	Medium
513+15	Remolded <sup>1</sup>	Subbase	110.5	12.1	0.03	12.8	100.0	--	--	--	--	Low
513+15	Remolded <sup>2</sup>	Subbase	107.4	12.1	0.05	12.3	100.0	--	--	--	--	Low
513+15	In Situ	Subbase	116.8	11.5	0.10	11.5	71.2	0.09	1.1	109	360	---

<sup>1</sup> Initial test results.

<sup>2</sup> Rerun of initial test.

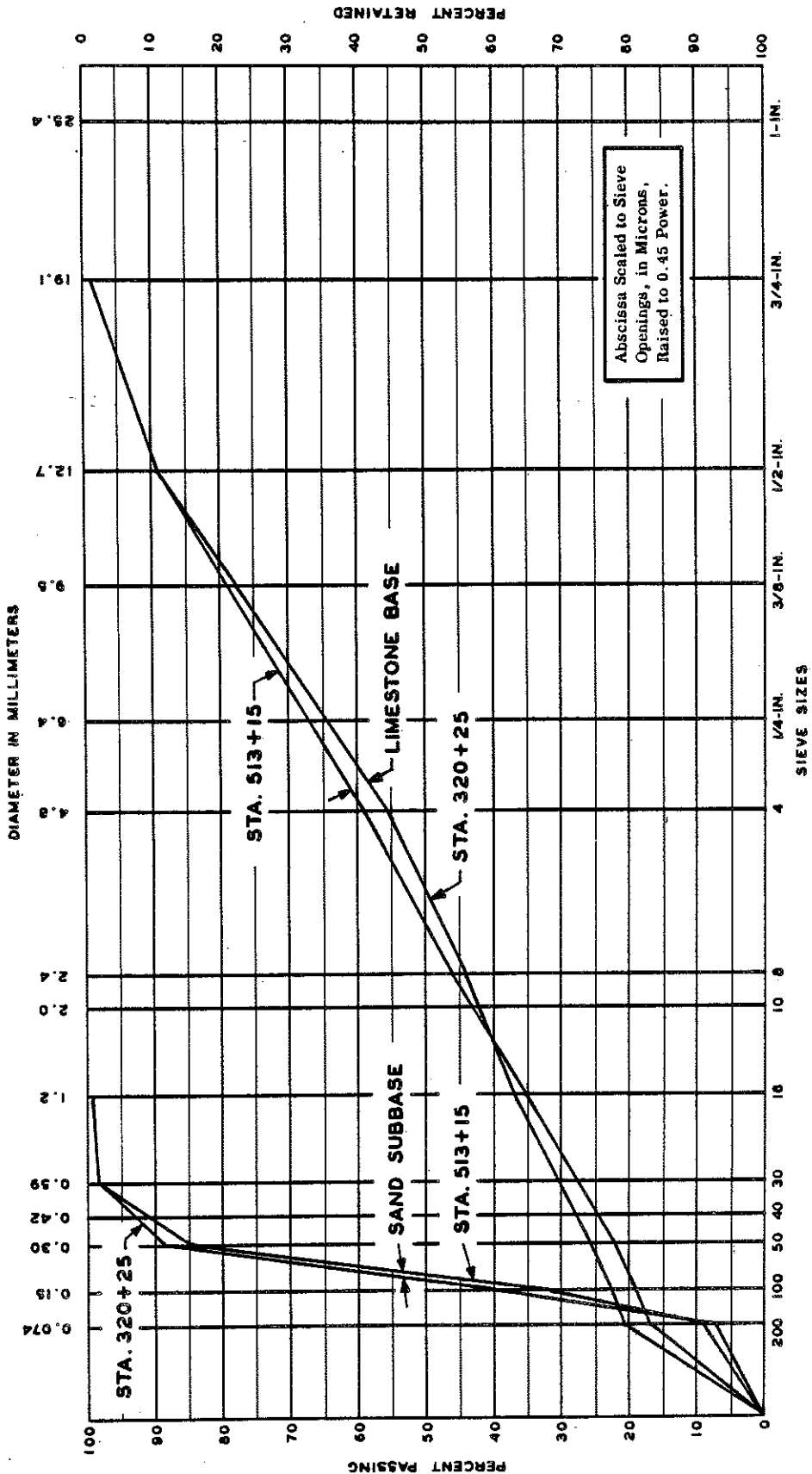


Figure 14. Gradation of base and subbase samples from I 75.

limestone base is, at the sites tested, subject to a high rate of degradation and should be highly susceptible to pumping because of its high fines content. Drainability analysis of the base indicates that its permeability may be optimal for pumping and that its high water-holding capacity might further enhance pumping characteristics. In addition, frost susceptibility tests indicate the base material to be moderately to highly frost susceptible. The frost susceptibility of the base, in the areas tested, is roughly equivalent to that of silt soils.

As stated in the pavement performance phase of this report, the general absence of faulting in the northern and southern ends of the pavement investigated, indicates that degradation of the base in these areas is not as rapid as it is in the central section. However, there appears to be no definite relationship between degradation and source of base material since all base material came from either France Stone (Pit 58-1) or France Stone (Monroe) (Fig. 15). While more data would be necessary in order to more accurately predict future pavement performance characteristics, indications are that all of the add-on lane between northbound exits 2 and 15 will be subject to premature failure of the type shown in Figures 10 and 11.

Subbase - The subbase gradation, at the sites tested, does not meet the porous Granular Material Class II specification requirement for percent passing the No. 100 sieve being composed principally of fine and very fine sands. Frost susceptibility of the subbase is low, that is, the subbase should frost heave to a limited extent in addition to the volume change that takes place when the subbase freezes at over 90 percent saturation. Therefore, the combination of these two forms of volume change on freezing could cause the pavement surface to heave as much as 1 in. or more at Station 320+25 where the subbase is 10 in. thick, to as much or more than 2 in. at Station 512+15 where the subbase is over 20 in. thick.

The drainability data consist of in situ or field test results and remolded (laboratory) test results. In general, these data indicate that the subbase, in the areas tested, has a very high water-holding capacity and an extremely low permeability. In combination, these properties will increase the time required for consolidation of the subbase layer on thawing. This means the pavement should experience a prolonged spring period of poor foundation support. It should be noted that the subbase at Station 320+25 was, on the basis of in situ tests, adequately drainable before the third lane was added but now drains too slowly to be acceptable because, due to the added width, drainage time now exceeds the suggested 10 day maximum allowable. Since drainage is restricted at the side slope, the actual drainage time should be even longer than that listed in Table 2.

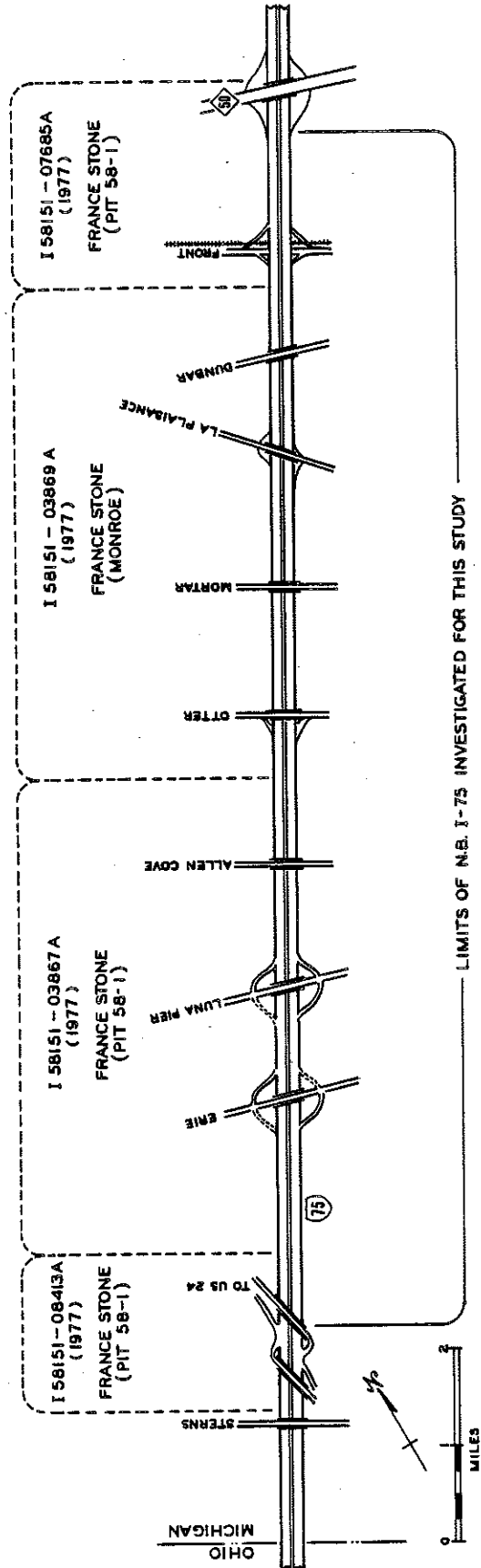


Figure 15. Aggregate base material sources for the control sections shown.

These data indicate that the installation of subbase underdrains may be beneficial in removing gravity drainable water from subbase and side slope areas in those cases where the subbase materials contain gravity drainable water. Such drains could also serve to reduce the time required for subbase consolidation. However, these benefits may not significantly improve the pavement's performance for two reasons: 1) the add-on lane is already heavily transverse cracked; 2) faulting, which for the most part leads to the need for repairs, is caused by pumping of the base which should be largely unaffected by the presence of a subbase underdrain.

#### Cause of Premature Failure

Transverse cracking of the add-on lane is believed to result from a combination of poor drainability and frost heaving properties of the base and subbase layers. When these layers thaw, the pavement provides a fluid support rather than an elastic support. When the support passes from an elastic to a fluid condition, it has been computed that the bending moment sustained by the pavement as a result of an applied axle load can increase by 30 times or more. That is, the stress sustained by the pavement during thawing can be 30 times or more greater than that which occurs later in the spring when the foundation has been reconsolidated by the traffic load and, where applicable, by gravity drainage.

Traffic load induces faulting, the magnitude of which is dependent upon base properties. Faulting has, in some areas, reached a magnitude sufficient for heavy trucks to create an impact load condition on the slab in the direction of traffic flow. This impact loading significantly magnifies the distress condition by increasing the number of transverse cracks and accelerating the rate of faulting and vertical displacement of the pavement surface. Where the subbase is subject to volume change, or its drainability does not permit rapid consolidation, the free slabs are vertically displaced or punched down below the surface elevation of the original pavement. Where the subbase is not subject to volume change or its drainability permits rapid consolidation, little differential surface movement appears to occur; however, additional investigation is needed to confirm this.

The original two lanes contain a large number of transverse cracks that are the result of poor support during springtime conditions that occur when the thawed subbase is reconsolidating. However, the subbase is apparently not as susceptible to pumping as is the limestone base under the add-on lane; therefore, the steel reinforcing mesh has been able to overcome any tendency for faulting.

In general, the add-on lane is more heavily transverse cracked than are the original lanes because not only did the addition of the third lane reduce subbase drainability for all lanes but it was constructed with an aggregate base that has degraded to the point of being highly susceptible to detrimental frost action and pumping. Faulting of transverse cracks in the add-on lane is the result of pumping properties of the limestone base which exceed the ability of the concrete reinforcement to prevent faulting. In this respect, the better performance of the northern and southern ends of the pavement studied is attributed to base material that is less susceptible to degradation and attendant detrimental performance characteristics. Downward displacement of broken pavement slabs appears to occur wherever faulting is severe enough to induce high-impact loads and subbase support is periodically reduced because of poor drainability.

### Conclusions

Rigid pavements have traditionally been designed with the assumption that the base and subbase layers are free draining. The results of this investigation adds to growing evidence that rigid pavement foundations are not free draining. The foundation of I 75 was found to be deficient in two critical respects: the base permeability and frost susceptibility is similar to that of silts; and, the subbase has a high water-holding capacity. In the case of rigid pavements, such foundation deficiencies can be minimized by using a greater thickness of concrete than would be used for a non-deficient foundation and by using reinforcing steel and load transfer devices. The two original lanes of pavements, included in this study, have no base layer and the subbase is subject to volume change on freezing; hence, on thawing is in a very unstable, fluid condition until reconsolidation is completed. Although this subbase deficiency has caused a high degree of transverse cracking in the concrete surface, reinforcement steel has held the pavement together in such a way that it has given good serviceability for the past 25 years or so.

The add-on lane includes a base that rests on the same subbase material as does the original two lanes. Addition of the third lane required extending the subbase which reduced subbase drainability and increased the detrimental performance characteristics of the subbase. In addition, the limestone aggregate base material is, in selected areas, subject to excessive degradation so that the base is now subject to a high degree of frost and pumping actions. The combined detrimental frost effect on both base and subbase has increased the intensity of transverse cracking in the add-on lane compared to the original two lanes, and the pumping action of the base has decreased pavement support to a point beyond the ability of the reinforcement steel to resist faulting of the cracked slabs. Faulting in the



add-on lane causes heavy vehicles to apply impact loads to the pavement surface, the intensity of which is a function of the degree of faulting. In those cases where the subbase is seasonally weakened for prolonged periods of time by frost action and where faulting develops severe impact loading, the cracked slabs are punched down, necessitating repairs and the intensity of transverse cracks increases. That is, once punching of slabs occurs and repairs are not quickly made, the severely distressed area rapidly increases in size.

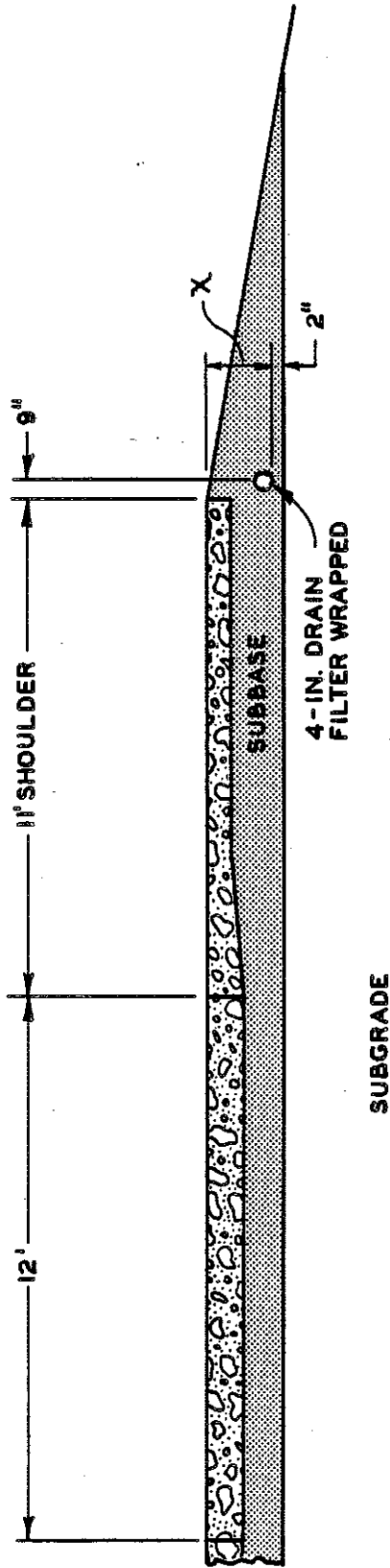
Two primary pavement deficiencies are causing premature pavement failures: 1) the subbase is subject to volume change on freezing thereby losing support capacity when thawing; 2) the limestone aggregate base, used for the add-on lane only is, in localized areas, subject to rapid degradation causing the base to be subject to highly detrimental frost and pumping action.

The opportunities for corrective action are very limited because no means exist for correcting foundation deficiencies. However, the addition of a subbase underdrain at the edge of the outside shoulder should reduce the time required for the subbase to reconsolidate after thawing. This may help reduce the frequency with which slab punching failures occur but should not entirely prevent the occurrence of such failures. In addition, subbase underdrains should have no influence on base degradation and its concomitant detrimental actions.

### Recommendations

It is recommended that a 'plowed in' retrofit subbase underdrain be installed to improve foundation drainage conditions. A recommended retrofit underdrain design and design notes are shown in Figure 16. Such a drain should allow for positive drainage of gravity drainable subbase water, enable consolidation of the base and subbase to take place more quickly, and have the lowest possible unit cost-estimated at around \$2.00 per foot plus the cost of delineator replacement. A more traditional retrofit underdrain could be designed and constructed but would cost significantly more without improving effectiveness.

If retrofit underdrains are to be installed, it is recommended they be added to Control Sections I 58151-03867A and I 58151-03869A only. If as a result, punching failures are attenuated by retrofit underdrains and if other sections of pavement begin to significantly fault, additional retrofit underdrains could then be installed where needed. Prior to design of retrofit underdrains, it is recommended that a survey be conducted to establish the depth to subgrade. By placing the bottom of the drains at or slightly above



**NOTES:**

1. Begin drain at high point outlet at lowest point using the elevation of the outside edge of the shoulder as the point of reference.
2. Longitudinal runs should not exceed 500 ft.
3. Drain pipe should be 4 in. diameter plastic pipe filter wrapped.
4. Drain should be plowed into place. If trench method is used, maximum trench width is 12 in. and the top 6 in. of material must be removed and windrowed on the side slope. The excavated subbase must be windrowed on the shoulder. The drain pipe must be placed directly on the bottom of the trench, back-filled with the subbase material, compacted with one pass of a vibratory roller and the top 6 in. of material replaced.
5. Drain must be placed at a constant depth and in the subbase layer so that drainage can take place whenever subbase thaws.

**X:** To be established on the basis of a soil survey. Dimension may change depending on depth to subgrade and the longitudinal extent of significant depth variations.

Figure 16. Recommended retrofit drain for I 75.

the subgrade, the cost of installation could be reduced, since the underdrain may be 'plowed in,' and no granular materials will have to be purchased for drain backfill. The suggested depth of placement should enhance performance since drains so placed work as soon as the subbase thaws, which is when drainage is most needed.

The repair of distressed areas should be made after placement of retrofit underdrains and within two weeks after the pavement has been sawed into removable slabs. This should prevent heavy truck traffic from increasing the size of the distressed area before repairs are made.

With respect to repair of the concrete slab, it is recommended that our standard procedures be followed except all slab replacements should be reinforced. Moreover, where several cracks in a row within a slab have faulted, consideration should be given to replacing the entire affected area. It should be emphasized, however, the performance of undowelled repair slab replacements under the extremely high truck traffic volume and prevailing foundation characteristics will undoubtedly be less than desired.