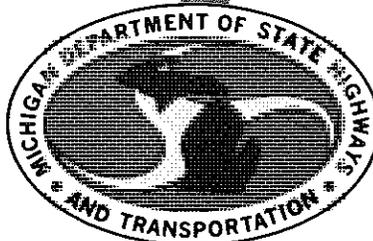


CONTINUOUSLY REINFORCED CONCRETE
PAVEMENT IN MICHIGAN
A HISTORICAL SUMMARY



**TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION**

**CONTINUOUSLY REINFORCED CONCRETE
PAVEMENT IN MICHIGAN
A HISTORICAL SUMMARY**

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**Research Laboratory Section
Testing and Research Division
Research Projects 61 F-64,
57 F-46, and 73 F-129
Research Report No. R-1120**

**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 1979**

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Introduction

From 1938 through March 1976, 12,789 equivalent two-lane miles of continuously reinforced concrete (CRC) pavement has been built or awarded in the United States. Thirty-four states have used this type of pavement. Among the states using CRC pavement, Michigan is ranked 13th in total lane miles constructed (1).

The first CRC pavement in Michigan was built in 1958 and through 1978 a total of 335 equivalent two-lane miles have been constructed or are under contract. Figure 1 shows the number of equivalent two-lane miles of pavement built or awarded each year since 1958.

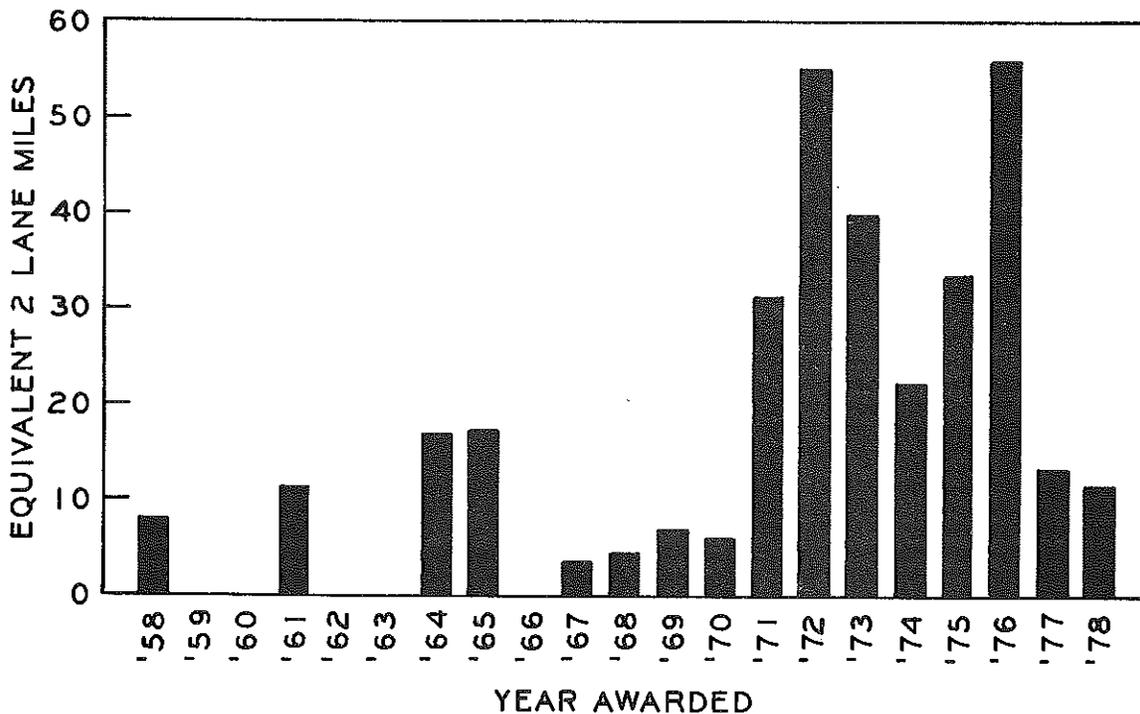


Figure 1. Equivalent two-lane miles of CRC pavement built or awarded since 1958.

The attractiveness of CRC pavement lies in its ability to function without relatively closely spaced transverse joints. However, the joints are eliminated at the expense of increasing the pavement reinforcement to about 3.5 times that used in a jointed pavement. As a result, the initial cost of CRC pavement has been higher than jointed pavement.

Although the performance of CRC pavements has, in general, been quite good, experience with these pavements indicates that corrosion is a serious problem, and a fatigue-induced type of failure develops in the concrete at the steel level on some projects constructed by the two-layer method. It is also apparent that loss of slab support causes punchout type failures in the slab. This report discusses the performance of CRC pavements to date based on field observations of construction and the numerous investigations concerning problems that have developed over the years. Design and construction changes made to improve performance as well as descriptions of the various types of failures experienced are included. Problems and procedures in maintaining CRC pavements are also discussed.

CRC Pavement Locations

The route, equivalent two-lane miles, construction years, and general location of the various segments of CRC pavement are as follows:

Route	Miles	Year	Location
I 96	8.0	1958	From M 66 to Portland Rd
I 96	11.1	1962	From Phillips Rd to Meridian Rd
I 75	37.2	1965 to 1969	From Pennsylvania Rd to Gratiot Ave
I 96	28.6	1969 to 1974	From I 75 to Southfield (M 39)
I 94	22.2	1972 to 1975	From Rawsonville Rd to Ozga Rd
I 696	4.1	1972	From John R Rd to Die St
I 196	46.2	1973 to 1974	From US 31 to 142nd St
I 275*	116.2	1973 to 1976	From I 75 to I 696 Interchange
US 31	53.2	1973	From Fruitvale Rd to Mason County Line**
I 696	2.5	1974	From Nieman Rd to I 94
US 31	6.2	1978	From State Line to US 12**

* Includes portion designated I 275/I 96.

** Construction not complete.

Design

Pavement Thickness - Both 8 and 9-in. pavement thicknesses have been used. Pavements in areas with less than 1,000 equivalent 18-kip axle loads per day is 8 in. thick and 9-in. pavements are in localities with more than 1,000 equivalent 18-kip axle loads per day.

Steel Percentage - Except for 0.60 percent steel used in the CRC pavement built in 1958 and 1962 the pavements contain 0.70 percent steel.

Reinforcement Type - Deformed bars, deformed wire, and smooth wire have been used as reinforcement. The smooth wire mesh was used on half of the pavement built in 1958 and 1962. Deformed wires were allowed from 1964 to 1976. The deformed bars (No. 5 or No. 6 bars) have been in use since 1958 and since 1976 are the only type of reinforcement permitted. Both of the two wire fabrics were installed in mat form which was also the case for deformed bars until about 1973. Since then, nearly all CRC pavements have been reinforced with loose bars.

Lap Length - Lap lengths utilized in 1958 were 12 in. for the smooth wire mesh mats and 13 in. for the bar mats. The laps were not staggered across the pavement.

On the 1962 CRC pavement a 14-in. lap length was used for both reinforcement types and a 2-ft stagger of the mats was used at each lap location. Since 1962 a lap length of 18 in. has been used with deformed wire mats and No. 5 deformed bars. For No. 6 bars the lap length has been 23 in. Mats have been staggered 4 ft at laps. The lap configuration used with loose bars has formed either a chevron or skewed pattern.

Construction Joints - At construction joints an additional bar of the same size as used in the pavement was placed adjacent to alternate longitudinal bars, except at construction joints in the 1958 and 1962 pavements where 18-in. long, 1-1/4-in. diameter steel bars were placed across the joint on 12-in. centers.

Transverse Steel - Transverse steel was required in all CRC pavements until 1973 when it was omitted from two and three-lane pavements. Tie bars or hook bolts through longitudinal joints are used in all CRC pavements.

End Treatment - A 10 expansion joint relief section was used at the ends on the 1958 pavement. Anchor lugs were used in 1962 through 1969. Since then, wide flange beam joints have been used except for lane pickups

and drops where anchor lugs were used. In mid-1978 the wide flange beam joints were discontinued and anchor lugs and a six expansion joint relief section were specified for clay and sand grades, respectively.

Concrete - The concrete for all CRC pavements was designed for a minimum 28-day compressive strength of 3,500 psi. All concrete was air entrained.

Curing - The concrete was cured by applying a membrane of white curing compound except in a few special cases curing blankets were placed after the curing compound had set.

Base - All CRC pavements constructed were placed on a base consisting of 10-in. granular material overlaid with 4 in. of dense graded aggregates, except the pavement built in 1958 was placed on a base of 9-in. granular material and 3 in. of aggregates. On depressed pavements the drainage is provided by use of 6-in. underdrains.

Construction

Concrete Mixing - The concrete used in building the 1958 CRC pavement was mixed in dual drum mixers traveling alongside the pavement edge. The remaining pavements have been constructed using central mixed concrete.

Forming - Until 1973 all CRC pavements were constructed using steel forms. Since then, with a few exceptions, the pavements have been slip-formed.

Steel Placement - Three methods have been used in placing the steel reinforcement. Up through about 1969 the steel was placed using the two-layer procedure, that is, the first layer of concrete was struck-off at the steel level and the steel mats placed on it. A second layer of concrete was then placed to complete the pouring operation. From 1969 through 1972 most of the steel was vibrated into the slab. The concrete would be placed full depth and the steel mats placed on top and depressed or vibrated into the concrete to the specified depth. Since 1972 the use of loose steel bars has been most common. The bars are laid out and tied ahead of the paver and are then passed through or over the concrete spreader. They are then spaced and depressed to proper depth just ahead of the slipform paver.

Concrete Consolidation - The concrete in nearly all CRC pavements placed prior to the beginning of the 1973 paving season was consolidated by use of pan type surface vibrators. In pavements where the steel was placed

on the surface and vibrated into place additional consolidation of the concrete would have occurred. Tube type internal vibrators have been used on all slipformed pavements since 1973. In addition, concrete placed at construction joints is required to be consolidated by use of hand-held vibrators.

CRC Pavement Failures

Since the initial use of CRC pavements in 1958, nine different types of failures have been experienced. Some of these occurred on the earlier pavements and have now been eliminated as a result of design and construction changes. The type of failures noted on Michigan's CRC pavements have also been noted on CRC pavements in other states (2, 3, 4).

Photographs and descriptions of each failure type along with a discussion of apparent failure causes and pavement location on which the failures occurred are as follows:

Lap Failure - Lap failures result in wide cracks forming in the pavement surface (Fig. 2), and in some cases, the slab breaks up over the lap area (Fig. 3). Investigations of this type of failure have shown that the cause of failure is a combination of insufficient lap lengths, failure of weld between crosswire and longitudinal wires, concrete bond failures to the steel, and horizontal separation of the concrete in the lap area.

Lap problems have been confined to the I 96 CRC pavements built in 1958 and 1962. Except for one lap failure that occurred after 18 years service in a section reinforced with deformed bar mats all failures have occurred in the sections reinforced with smooth welded wire mesh. By discontinuing the use of smooth welded wire mesh, increasing the lap length and staggering of the laps, lap failures have been eliminated from pavements constructed since 1962.

Construction Joint Failure - Construction joint failure is typified by the cracking and breaking up of the slab on one or both sides of the joint (Fig. 4). The failure normally occurs relatively soon after construction of the pavement. The cause of failure on the night side of the joint is attributed to depositing the soupy concrete mixture carried in front of the screed in the pavement adjacent to the joint. On the morning side, a poorly consolidated, lean concrete mixture and reinforcement lap problems have been diagnosed as the primary cause for pavement failure.

Construction joint problems were common in the earlier CRC pavements. The requirement that soupy concrete would not be allowed in the

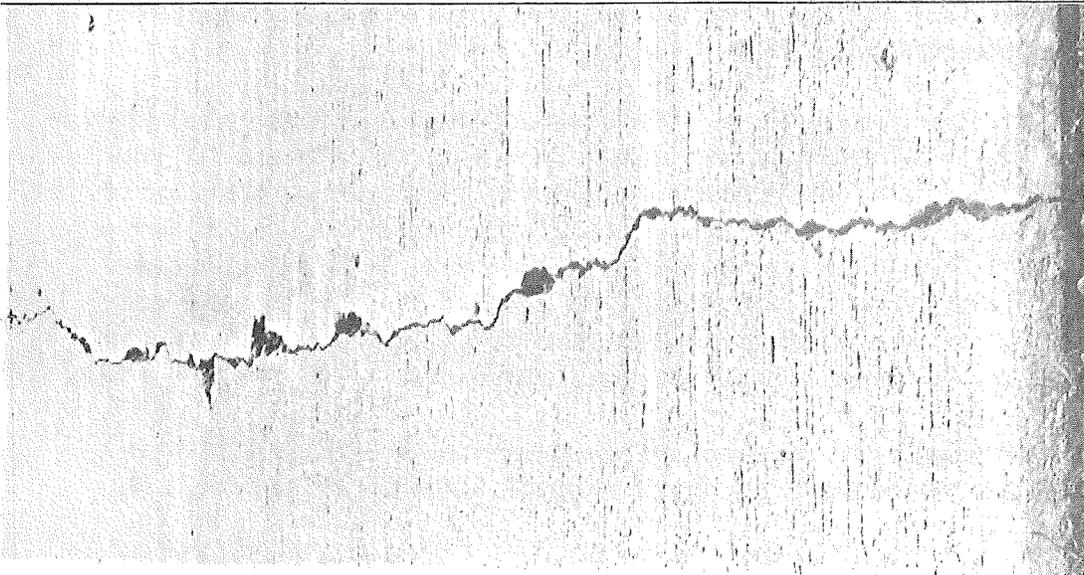


Figure 2. Lap failure in CRC pavement. Surface crack extends to steel level where a horizontal crack has formed and extends to the end of the lap where the bottom portion of the slab has cracked vertically.

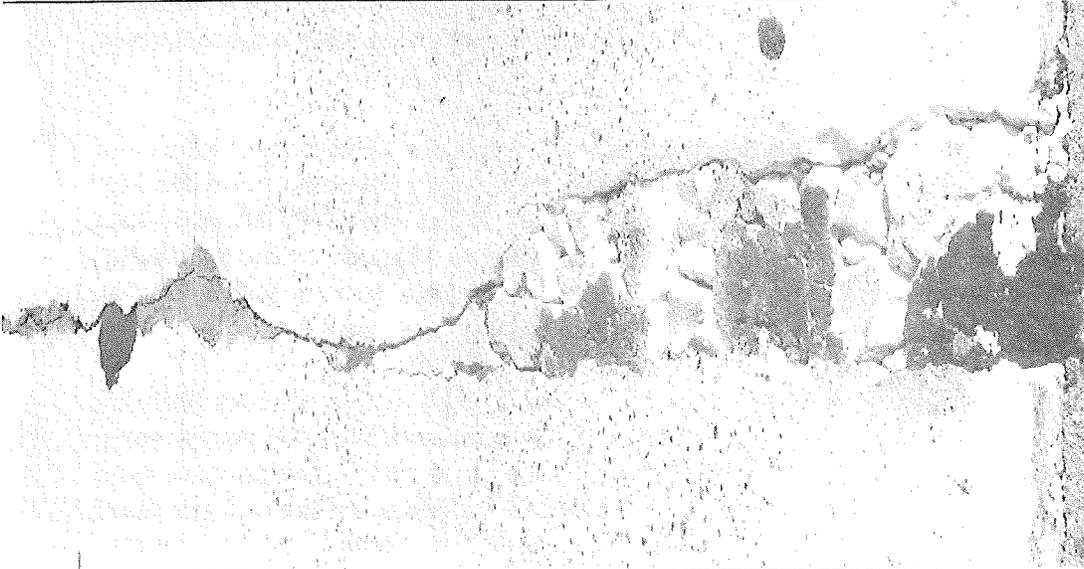


Figure 3. Spalling and breakup of the concrete develops at lap failures as a result of climatic changes and traffic loads.

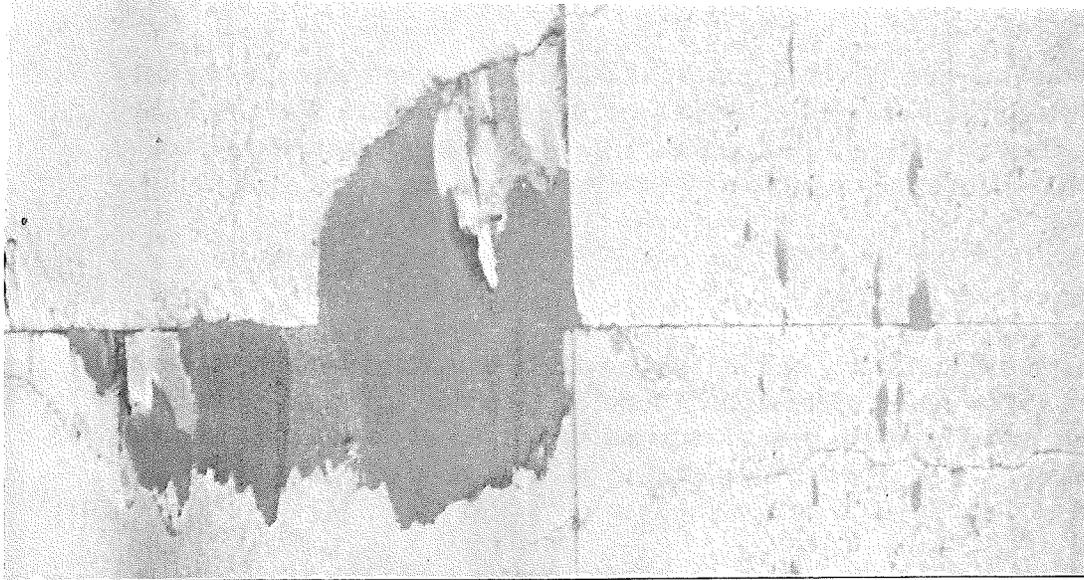


Figure 4. Typical construction joint failure in CRC pavement.

slab, the use of additional cement in the concrete mixture at the morning side, positive reinforcement support, and more careful attention to consolidation have basically eliminated construction joint problems since 1970.

Failures at Joints in Adjacent Pavements - A wide crack forms in the CRC pavement in line with a joint in the adjacent jointed pavement and in time the concrete breaks up between this crack and an adjacent one (Fig. 5). Apparently, the adjacent jointed pavement, which is tied into the CRC pavement, induces sufficient tension in the CRC when the jointed slabs change length, to allow the crack to open excessively at the edge. Eventually, the concrete separates at the steel level, and then fractures under load.

This type of failure has occurred only on the I 96 pavement built in 1962 and then only at two ramps that were tied to the CRC pavement. A change in design that allows the adjacent jointed concrete to change length without inducing excessive tension in the CRC pavement accounts for the absence of this type of failure on other CRC pavement projects.

High Steel Failure - The shallow layer of concrete above the steel fractures and is kicked out by traffic, thus, exposing the reinforcement (Fig. 6). High steel failures occur after several years service and are normally preceded by rust stain on the pavement surface. This indicates that corrosion of the steel induces fracture planes and once separation of the concrete occurs, traffic breaks the concrete into pieces.

To date, high steel failures have not occurred in pavements constructed with loose steel bars nor have they appeared on pavements built between 1970 and 1973 when the steel was vibrated into position. The total number of this type of failure is relatively small and nearly all of them are on a 3 mile section of I 75 built in 1966.

Concrete Breakup Failure - The concrete above the reinforcement is fractured (Fig. 7). Concrete breakup normally occurs in areas of narrow crack spacing and on pavements with more than five years of service. It is generally found in the wheel track areas of the most heavily traveled lanes. All failures of this type are in CRC pavements constructed by the two-layer method and nearly all of them are on depressed sections where drainage is provided by use of edge drains.

The concrete breakup failures appear to be caused by a combination of several factors: poor consolidation of the two concrete layers used in two-course construction, corrosion of the reinforcement, a base condition that allows excessive slab deflection, short crack spacings, and traffic loads.

Figure 5. Failure in CRC pavement adjacent to conventionally jointed pavement.

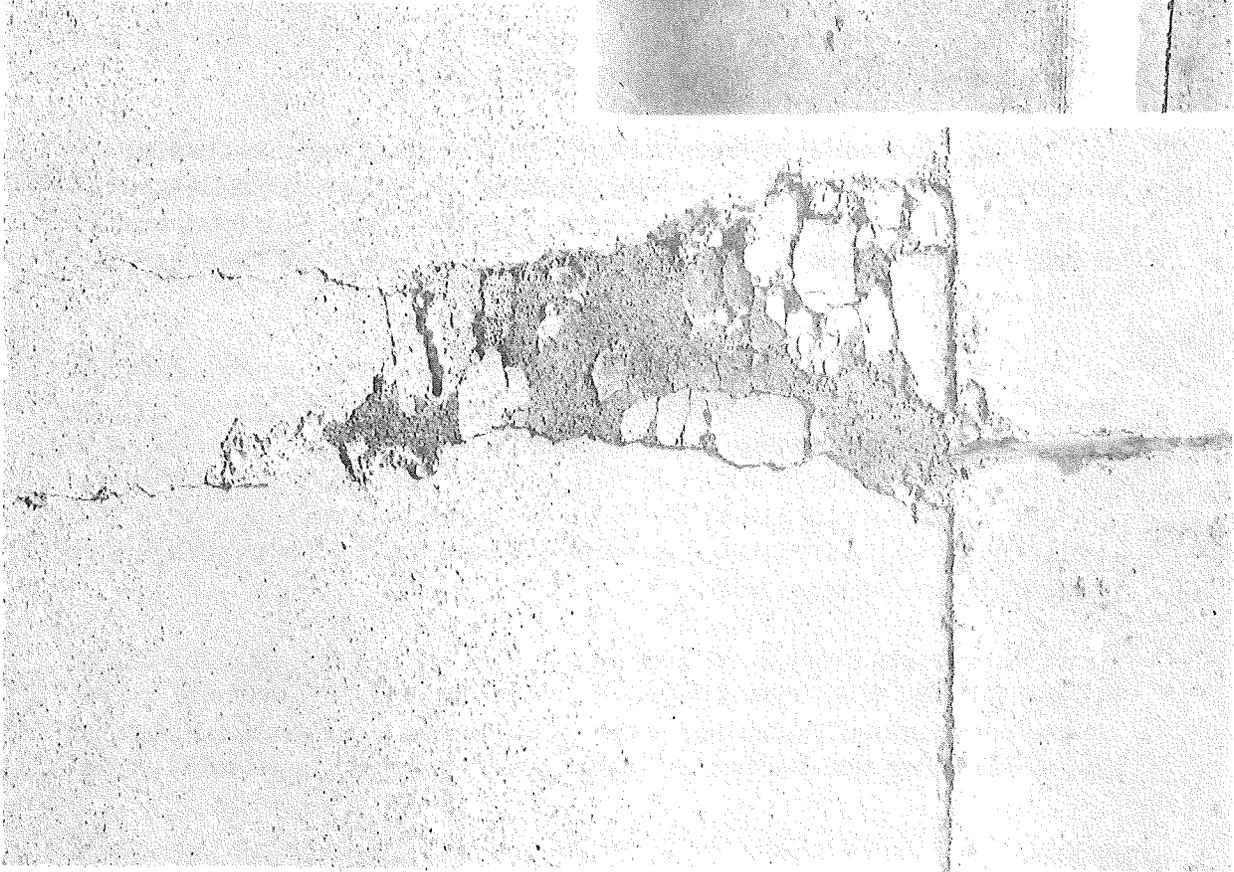


Figure 6. CRC pavement failures caused by the reinforcement placed too high during construction.



Figure 7. Typical concrete breakup failures in CRC pavement.





Figure 8. Horizontal crack at steel level in lane under repair.

Figure 9. Horizontal crack at steel level in lane adjacent to the one being repaired. Note excellent condition of pavement surface.

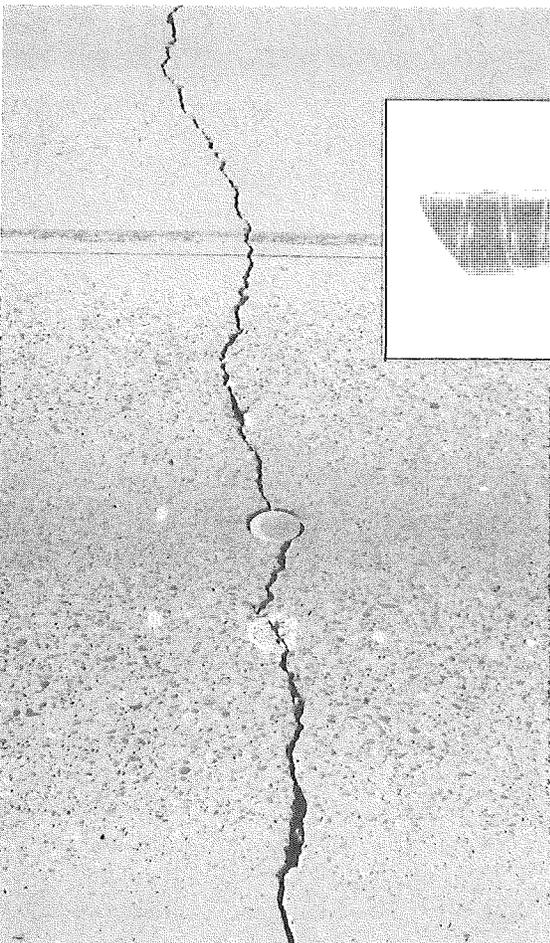
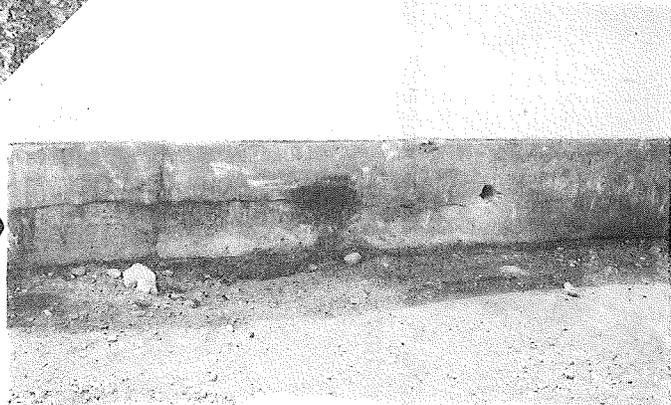


Figure 10. Steel failure in CRC pavement 16 years old. Reinforcement is No. 5 deformed bars, 8-in. pavement with 0.60 percent steel.

During repairs of failures it was found that the concrete had separated at the steel level. Figure 8 shows the separation crack at the end of a repair and Figure 9 shows separation of the concrete in an adjacent lane.

The apparent failure process is summarized as follows: a horizontal plane of weakness is created at the steel level, in some areas, due to inadequate concrete consolidation. The stress relieving cracks form at close spacings at some locations and at larger ones at other locations. Moisture, and especially saltwater, begins entering the cracks causing rusting of the steel resulting in corrosion product pressures tending to split the concrete apart at the steel level. At the same time, changes begin to take place in the base because the drains begin to clog and differences in base support result. Now, as loads are applied, the pavement deflects more in areas of short crack spacings than in areas with long ones and eventually the concrete fatigues or breaks apart at the steel level with the first failures occurring at the weakest points in the pavement. Once the slab has separated, load applications break the top layer into pieces.

The depressed sections where these failures have occurred are located on I 75. It is anticipated that the use of loose bars being depressed into the concrete and use of internal vibrators will eliminate concrete breakup failures or prolong the time until they appear.

Steel Corrosion Failure - The steel has corroded and fractured and a wide crack is noticeable in the pavement surface (Fig. 10). Corrosion failures are caused by rusting of the steel at cracks where high tensile stresses occur. These failures have been observed where the crack spacing is unusually large which, of course, means the cracks are wider than normal, and thus, saltwater from ice and snow removal can easily penetrate to the steel level.

Steel corrosion failures have occurred only on the 16 year old pavement between Phillips and Meridian Rd on I 96. Examination and tension tests conducted on steel samples removed from repair areas on 14 and 19 year old pavements indicate corrosion of the steel will eventually cause continuity failures of the pavement. A detailed description and discussion of the study concerning steel corrosion of the removed samples are given in the Appendix.

Punchout Failures - The settlement and breakup of a small section of pavement bounded by intersecting cracks (Fig. 11). On the basis of examination of the failures and cores taken through punchout areas, it appears that the cause of failure is related to localized loss of base support, heavy truck traffic, poor consolidation of the concrete around the steel bars, narrow transverse crack spacing, or Y-type cracks intersected by longitudinal cracks.

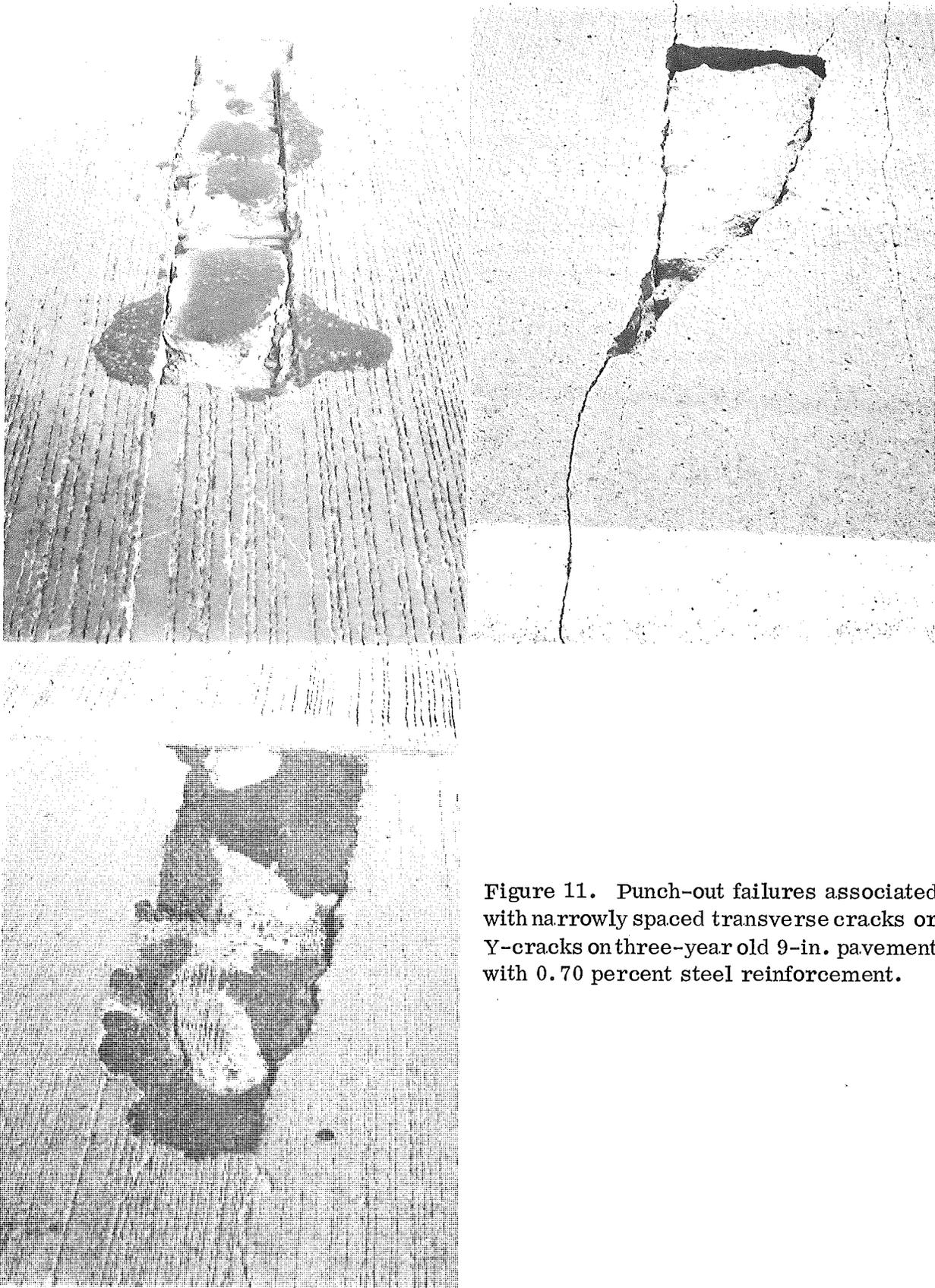


Figure 11. Punch-out failures associated with narrowly spaced transverse cracks or Y-cracks on three-year old 9-in. pavement with 0.70 percent steel reinforcement.

The failure process apparently starts by a longitudinal crack or cracks forming between two narrowly spaced transverse cracks located over a weak spot in the base. Once the concrete piece is surrounded by cracks the wheel loads destroy the aggregate interlock and then the concrete is supported only by the rebars going through it. The continued application of wheel loads fails the concrete bond to the steel and eventually the concrete wears away around the bars. The concrete now settles into the base equal to the thickness of concrete worn away over the rebars. In some cases, the concrete piece breaks into smaller ones and some spalls off at the steel level.

Punchout failures have occurred only on I 94 which was constructed in 1975.

Longitudinal Crack Failure - There are two types of longitudinal cracks: one type is where the crack parallels the centerline joint (Fig. 12) and the other one is where the crack is located somewhere in the center 8 ft of a 12 ft lane (Fig. 13). It is generally agreed that the crack along the sawed centerline joint is caused by late sawing or loading the slab before sawing. Crack formation in the interior portion of a lane is normally associated with base problems. It should be noted that longitudinal cracks of both types occur in conventional reinforced concrete pavements as well as in CRC pavement.

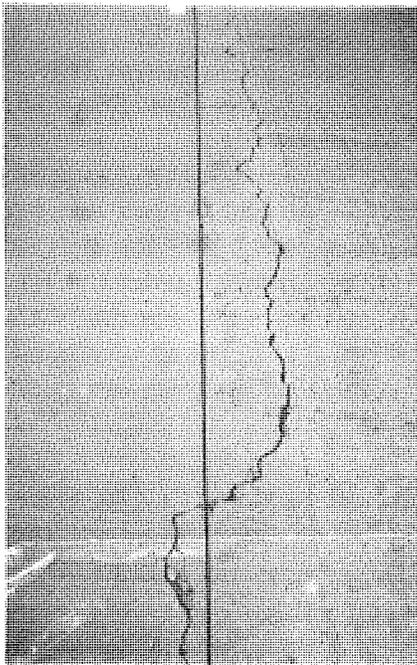
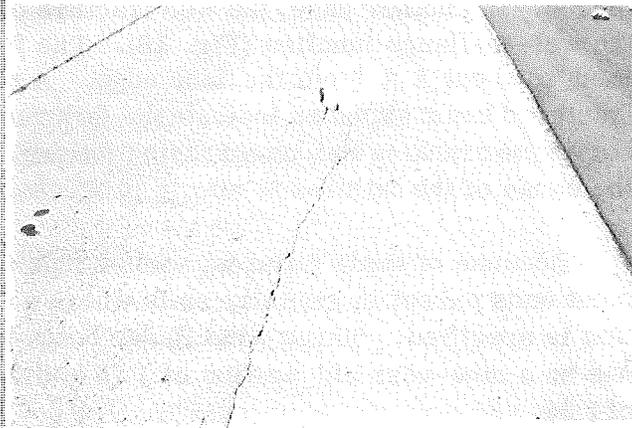


Figure 12. Longitudinal crack paralleling centerline sawcut.

Figure 13. Longitudinal crack in the interior portion of the lane.



Other than minor spalling along the crack that follows the centerline joint, no problems have been experienced to date. The tie bars through the joint prevent the crack from opening excessively. At the interior cracks in CRC pavement, punchout failures have occurred (Fig. 14). They occur where a second longitudinal crack forms close to the main crack and then develops in the manner described earlier for this type of failure at transverse cracks.

Longitudinal cracks have been noted on I 196, I 275, and US 31. These pavements were constructed during the past five years.

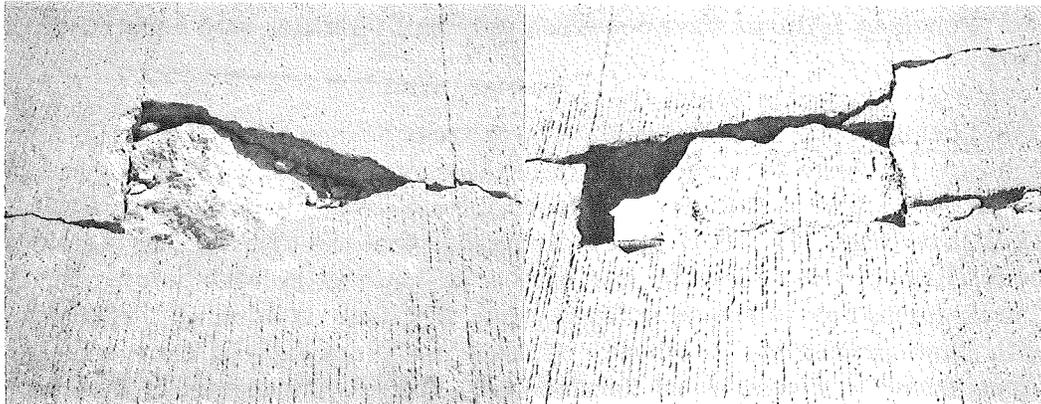


Figure 14. Punch-out failures associated with longitudinal cracks.

Wide Flange Beam Joint Failure - This type of joint failure results from the web of the wide flange beam fracturing. In one case (Fig. 15), the fracture occurred in the web just above the sleeper slab. The web had fractured for its entire length and the beam would bounce when traffic passed over it. Before the failure was discovered the beam had broken into two pieces. In another case, the web fracture occurred just below the fillet at the web-to-flange junction (Fig. 16). The fracture extended from the lane joint to about 1 ft from the lane edge. A preliminary examination of the fractured beam surfaces indicate the failures could be the result of stress-corrosion cracking and/or corrosion fatigue. A detailed study to determine the cause of the failures is now in progress.

Because of these failures, anchor lugs are now required at CRC pavement ends except in granular soils where a six-expansion joint relief section is specified. Three wide flange beam failures have occurred to date: one on a nine-year old section of I 75 and two on a three-year old portion of I 94.

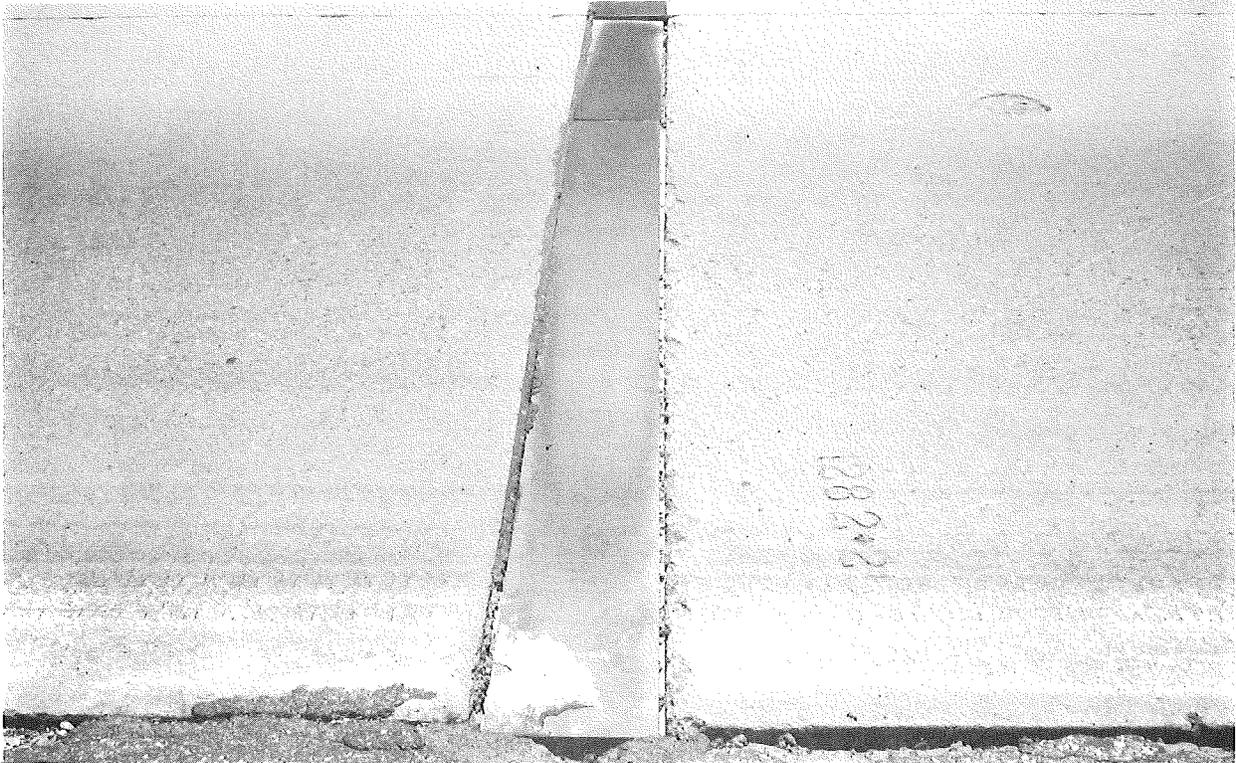


Figure 15. Wide flange beam failure. Web is broken at the sleeper slab level and the flange is broken 3 ft from centerline joint.



Figure 16. Wide flange beam failure. Web failed just below the fillet at the web to flange junction.

Maintenance of CRC Pavement

Maintenance of this type of pavement is similar to that of conventional pavement. The failures are patched with bituminous material until more permanent concrete repairs can be made. The concrete repairs are of two types: one requires the steel continuity to be maintained, and the other utilizes expansion relief joints with or without dowels.

A repair where the steel continuity is to be maintained is made by first making a shallow sawcut at the end repair limits and a full-depth cut 2 ft 6 in. inside the end limits. The concrete and steel are then removed in the center portion of the repair; whereas, in the end portions only the concrete is removed. New steel bars of the same size as the existing ones are placed and tied to the old bars using a lap length of 30 in. Concrete is poured over and through the rebars, vibrated, hand finished, and cured. A 9-sack concrete mix is used to allow opening to traffic after 24 hour cure. The repairs of this type are done one lane at a time. If the steel continuity has failed in both lanes, the new steel is welded to the existing bars in the lane repaired first. The welding is postponed until late afternoon when expansion of the slab has ceased.

The distressed pavement at repairs using dowelled joints is removed in the same manner as for repairs where the steel continuity is maintained, except the existing steel extends only 14 in. into the repair area. An expansion joint assembly with a 2-in. wide filler is placed in the middle of the repair, if repairs are spaced less than 400 ft apart. Two expansion joints, providing a 4 in. filler width, are used when repairs are spaced over 400 ft apart. Conventional reinforcement is used in all repairs. If lane closures are allowed overnight a 9-sack concrete mix requiring 24 hr cure is used, otherwise calcium chloride is added to reduce the closing time to daylight hours.

Undowelled repairs are made by making full-depth saw cuts at the repair end limits. The concrete is removed and a 1-in. expansion filler placed against the vertical edge of the existing slab at each end of the repair. The concrete, a 9-sack mix with or without calcium chloride depending on closure limitations, is placed and finished in the normal manner.

All concrete repairs are 6 ft minimum in length and 12-ft wide. Partial-depth repairs (minimum size 3 ft by 3 ft by 3 in. deep) were tried two years ago for high steel failure areas. However, the cost of a minimum sized partial-depth repair was about the same as for a minimum sized full-depth repair. Also, in some cases, it was found that it was necessary to remove the concrete full depth, which drastically increased the cost of

partial-depth repairs, since they were paid for on a cubic foot basis. Therefore, all CRC pavement repairs are now full depth.

The type of repair to be used depends on the condition of the pavement to be repaired. The repair calling for maintaining the steel continuity is normally used until steel fractures or lap failures (in smooth mesh reinforced sections only) occur. When steel fractures are noticed and the integrity of the slab itself is still good, repairs with dowelled joints are used. Undowelled repairs, to date, have only been used on smooth welded wire mesh reinforced sections where the failures are relatively closely spaced and continue to occur in the remaining longer slab sections.

Current Condition of CRC Pavements

I 96 (From M 66 to Portland Rd) - This 8 mile section of a two-lane pavement is now 20 years old. The half of the pavement reinforced with smooth welded wire mesh began failing in the reinforcement laps shortly after it was built. As soon as it was determined that lap problems were going to continue to occur, repairs with undowelled slabs were specified. In 1977, 58 two-lane repairs were made and the pavement is now essentially a jointed pavement.

The section reinforced with deformed bar mats has given excellent service. Two construction joint problems were corrected in 1962 and no further maintenance was done until last year when one construction joint and a lap failure near a relief section were repaired. Tests of steel samples removed from the 1977 repair areas indicate that corrosion of the reinforcement is occurring (see Appendix).

I 96 (From Phillips Rd to Meridian Rd) - One roadway of this 5.5-mile section of freeway is reinforced with smooth welded wire mesh and the other with deformed bar mats. Lap failures began to occur shortly after construction of the mesh reinforced section and periodic maintenance with undowelled repairs has been necessary. Presently a contract is being prepared for 180 new repairs.

On the bar mat reinforced section, failures caused by corrosion of the reinforcement were discovered this past winter. An inspection of the pavement revealed the following: fourteen corrosion failures, seven imminent corrosion failures, six failures at ramp joints, and five areas with surface spalling. These 32 locations are included in the repair contract under preparation. The repairs are designed to be tied into the existing steel reinforcement and dowelled joints are to be used. To provide for pressure relief the joints will be of the expansion type.

I 75 (From Pennsylvania Rd to Gratiot Ave) - This pavement ranges in age from 9 to 14 years. Except for failures at construction joints and where paving equipment problems were experienced during construction, the pavement has performed reasonably well with only temporary bituminous patching being needed until two years ago. In 1976 the inspection of the pavement indicated that concrete repairs were needed and contract repairs were made during the summers of 1977 and 1978.

Most of the failures are caused by breakup of the top layer of concrete (see section on CRC pavement failures). The conditions for this type of failure to occur are present or developing in the slab and numerous new failures have occurred. Currently, these new failures are planned for contract repairs in 1979. One wide flange beam joint has failed and a temporary repair has been done.

I 96 (From I 75 to Southfield (M 39)) - Construction of this pavement section began in 1969 and was completed in 1974. To date no failures of any type have been found.

I 94 (Rawsonville Rd to Ozga Rd) - This pavement ranges in age from three to six years and consists of 22 miles of equivalent two-lane pavement. Six punchout failures and two wide flange beam joint failures have occurred to date. The punchout failures are being maintained with bituminous patching and the wide flange beams have been removed and temporarily repaired with bituminous material.

I 696 (From John R Rd to Die St and from Nieman Rd to I 94) - The pavement between John R Rd and Die St is six years old and from Nieman Rd to I 94 it is four years of age. No failures have occurred on either pavement to date.

I 196 (From US 31 to 142nd St) - On this 23 mile section of freeway, four to five years old, the only problem noted to date is minor longitudinal cracking along the centerline sawcut on two projects. The cracks are still fairly tight and no maintenance has been necessary.

I 275 (From I 75 to I 696 Interchange) - This section of pavement contains 116 two-lane miles and it was constructed from 1973 through 1976. Mid-lane longitudinal cracking has occurred on some portions of the pavement, and punchout failures have developed in two areas. They are being maintained with bituminous material at the present time, but no maintenance work has yet been done to the cracks themselves.

US 31 (From Fruitvale Rd to Mason County Line and from Indiana State Line to US 12) - Both of these freeway sections are still under construction. On the portion between the State Line and US 12 the southern half was constructed in the fall of 1978. Construction of the pavement between Fruitvale Rd and the Mason County Line began in 1973, and by the end of the 1978 construction season, a total of 44 miles of equivalent two-lane pavement had been placed. Mid-lane longitudinal cracks in two pours placed in 1978 have been found, but no punchout failures have developed as yet and no maintenance work has been done.

Conclusions

Based on 20 years of experience with CRC pavement, it is now evident that along with the advantages of this type of pavement certain disadvantages must be dealt with after a number of service years. In general, CRC pavements, except for construction and design related failures, have performed excellently for the first eight years; in fact, one pavement section has given 19 years of maintenance-free service; whereas, other pavement sections—especially those constructed by the two-layer method—are deteriorating at a rapid rate after 10 years of service. Fortunately, design and construction changes have been made during the period CRC pavement has been in use, which should eliminate problems experienced on the earlier pavements. Although pavements placed since about 1970 are anticipated to perform longer and better than those built during the sixties the following conclusions pertain to all CRC pavements.

1) CRC pavement is sensitive to construction related problems. For example, the reinforcement must be placed within narrow tolerances to prevent fracture planing from high steel, and continuity loss with low steel. The concrete must be properly consolidated around the reinforcement to ensure good bond and minimize separation of the concrete at the steel level. Evidence indicates that the concrete breakup failures on I 75 are, in part, caused by poor consolidation between the top and bottom concrete layer. Since concrete placed after about 1970 has been internally vibrated, placed in one layer, and the reinforcement vibrated into position failures caused by high steel and poor consolidation should not develop on these pavements to the degree they have on pavements placed by the two-layer method.

2) CRC pavement requires a firm, uniform base support to prevent premature failures. As mentioned previously, the I 75 concrete breakup failures are for the most part located in depressed areas and it is suspected that the underdrains are not functioning as efficiently as when new, resulting in reduced base support for the slab. Consequently, the slab deflections increase which contribute to the formation of the horizontal separation of

the concrete at the steel level. Likewise, the punchout failures experienced on I 94 and the longitudinal cracking on I 275 that occurred within a year after construction appear to be related to base problems.

3) CRC pavement reinforcement corrosion and fracture across the entire lane or roadway will eventually occur. Fracture of the steel reinforcement in CRC pavement, due to corrosion, was discovered on the I 96 pavement between Phillips and Meridian Rd after 15 years of service. In Minnesota and Iowa steel fractures were found in pavements four or five years old (4). The fractures on I 96 occurred in No. 5 deformed bar mats whereas, those in Minnesota and Iowa were in deformed wire mats. As yet, there is not enough information on the corrosion rate of the reinforcement to forecast the time a certain section of CRC pavement will experience steel failures. Long crack spacings are apparently conducive to steel failures, since the failures on the I 96 pavement are located in areas with crack spacings in the range of 6 to 12 ft. Test and examination of steel samples from I 75 also indicate corrosion of some bars at a pavement cross-section, in normal crack spacing areas, is quite advanced (see Appendix).

4) Maintenance of CRC pavements is more difficult than that of conventionally reinforced jointed pavements. One of the reasons for this is that it contains about 3.5 times as much steel that must be cut when full-depth repairs are made. Also, at least during the early life of a CRC pavement, the steel continuity must be maintained at repairs by letting the existing steel extend intact into the repair area. This requires a good deal of hard and time-consuming handwork. If the entire roadway has failed the resteel must be welded and concrete pouring scheduled at favorable temperature conditions. Once the pavement reaches the point when its continuity can no longer be maintained (because of corrosion failures) the maintenance procedures revert to those used on jointed pavement. Although the final determination of a rehabilitation procedure for CRC pavement has not yet been decided upon, there appear to be two choices: one would be to cut the pavement into short sections and then place an overlay on it, and the other would be to recycle or remove and replace the entire slab. In some cases, such as on I 75 where the top concrete layer is breaking up there may be only one solution and that is removal and replacement of the entire pavement.

Recommendations

From our experience with CRC pavement it is evident that it is not a problem-free pavement. The most disturbing fact is that the reinforcement will rust through. Once this occurs it appears necessary to convert to a

jointed pavement or saw it into short slabs and then place a bituminous overlay on it. Considering this, it would appear that a better solution would be to construct a conventionally reinforced pavement to begin with. It is, therefore, recommended that the use of CRC pavement be discontinued or at least suspended until economical corrosion protection of the reinforcement is developed.

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2. McCullough, F. B., "Report on Performance of CRCP in Indiana," Austin Research Engineers, November 1972.
3. Teng, Paul T. C., and Coley, J. O., "Continuously Reinforced Concrete Pavements in Mississippi," Final Report, April 1973.
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APPENDIX

CORROSION OF REINFORCEMENT IN CRC PAVEMENT

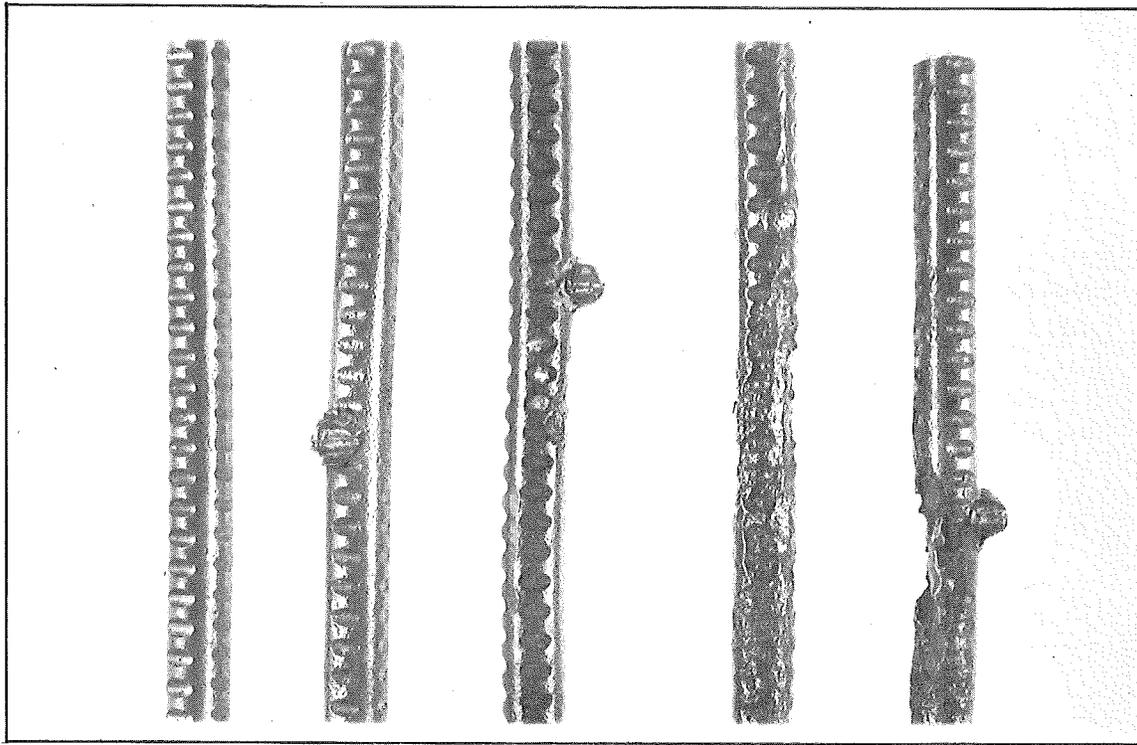
Investigation of early failures on the CRC pavement sections on I 96, built in 1958, and reinforced with smooth welded wire mesh revealed considerable corrosion of the steel wires at crack locations. Although the corrosion had occurred at wide cracks within the failed areas, it generated concern as to the condition of the steel at cracks of normal widths. As a result, cores containing steel bars were taken at various crack locations and samples of reinforcement were removed from repair areas at various times, for the purpose of checking the condition of the steel reinforcement.

Unfortunately, it is difficult to develop a meaningful procedure for determining the degree and effect of corrosion that has occurred on reinforcing steel. In an attempt to establish consistency for rating and comparing the corrosion attack the corrosion scale given below was devised.

CORROSION SCALE

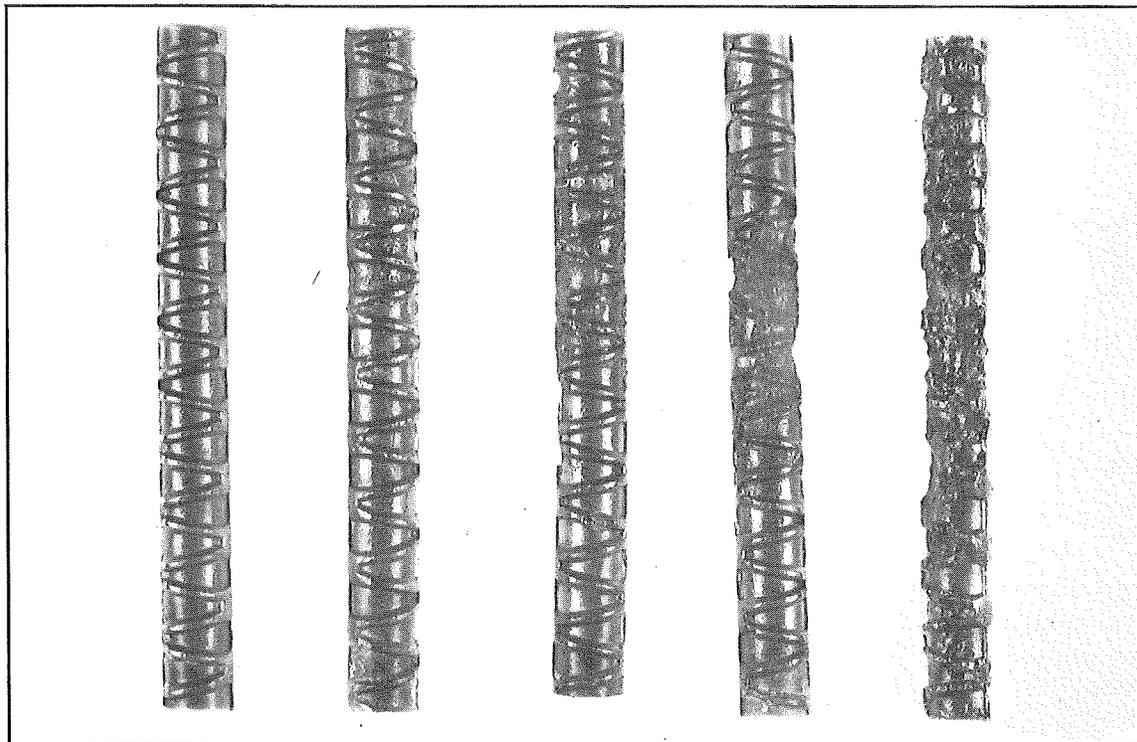
Index	Definition
1	No corrosion. Discoloration of the surface of the bar may have occurred.
2	Mild corrosion. Scattered minute pitting of the bar surface.
3	Moderate corrosion. Concentrated pitting of the bar surface with pit depths up to approximately 1/32 in., and/or the beginning of slight uniform reduction of the bar diameter.
4	Severe corrosion. Concentrated pitting of the bar surface with pit depths greater than approximately 1/32 in., and/or uniform reduction of the bar diameter up to approximately 25 percent.
5	Critical corrosion. Deep concentrated pitting of the bar surface, and/or uniform reduction of the bar diameter from over approximately 25 percent to failure of the bar.

The five stages of corrosion are illustrated in Figure 1A, for both deformed wires and deformed bars. To determine the corrosion index for the steel samples each bar was visually examined independently by three different persons. Since the rating scale is subject to interpretation, the three ratings did not agree in all cases. Where two of the three observers were in agreement the index number, which was assigned by the two observers, was used. In cases where all three observers assigned a different index to the same sample the middle index number was used to indicate the corrosion.



1 2 3 4 5

RATING INDICES
No. 29 Deformed Wires



1 2 3 4 5

RATING INDICES
No. 5 Deformed Bars

Figure 1A. Steel samples showing the five stages of corrosion.

During repairs on I 75 and I 96 in 1977, samples of reinforcement across the entire lane were removed. These samples were examined for corrosion and later tested to determine their tensile strength. The results of the corrosion examinations and tensile tests are discussed below.

I 96 (From M 66 to Portland Rd) - This CRC pavement section was built in 1958. Steel samples from cores of both smooth wires and deformed bars were removed in 1963, 1966, and 1968. In addition, samples of deformed bars were removed by coring in 1973 and in 1977 samples of the reinforcement across the traffic lane at two repairs were taken. All steel samples were cleaned by wire brushing and rated with respect to the extent of corrosion. After rating, the samples removed from the repair areas were tested in tension to determine their ultimate tensile strength.

The corrosion ratings of the three sets of samples from the smooth welded wire mesh section are shown on the top portion of Figure 2A. As can be noted, none of the samples in the three age brackets were rated higher than 3 or 'moderately corroded.' Since the use of this type of reinforcement was discontinued in the early sixties no further study of its corrosion was made.

The corrosion of the deformed bar samples from cores is shown on the middle portion of Figure 2A. Both the five and eight year old samples were rated in the 1 and 2 categories, whereas, in the 10 year old samples one sample was given a rating of 3. Of the 14 year old samples 22, 34, and 44 percent were rated 2, 3, and 4, respectively. Thus, the steel samples removed from cores indicate that corrosion increases with time as would be expected.

In order to determine the corrosion at a crack across a lane, steel samples were removed from two repair areas in 1977. The percentage of bars with a given rating number is shown on the bottom portion of Figure 2A. As can be seen, after 19 years of service a large percentage of bars show corrosion ratings of 4 and 5. At the construction joint, 96 percent were rated in the 4 and 5 categories. The condition of the pavement area from which the steel was removed is shown in Figures 3A, 4A, and 5A for the lap failure, crack near construction joint, and for the construction joint, respectively.

Results of tensile tests are shown in Figures 3A, 4A, and 5A along with a photograph of the pavement area from which the samples were taken. The bars are numbered from the shoulder toward the pavement centerline. As can be seen, the strengths of the bars vary across the pavement section, reflecting, in part, the degree of corrosion on each bar. The average tested yield and tensile strengths of the specially designed hardgrade steel bars are shown on the graphs.

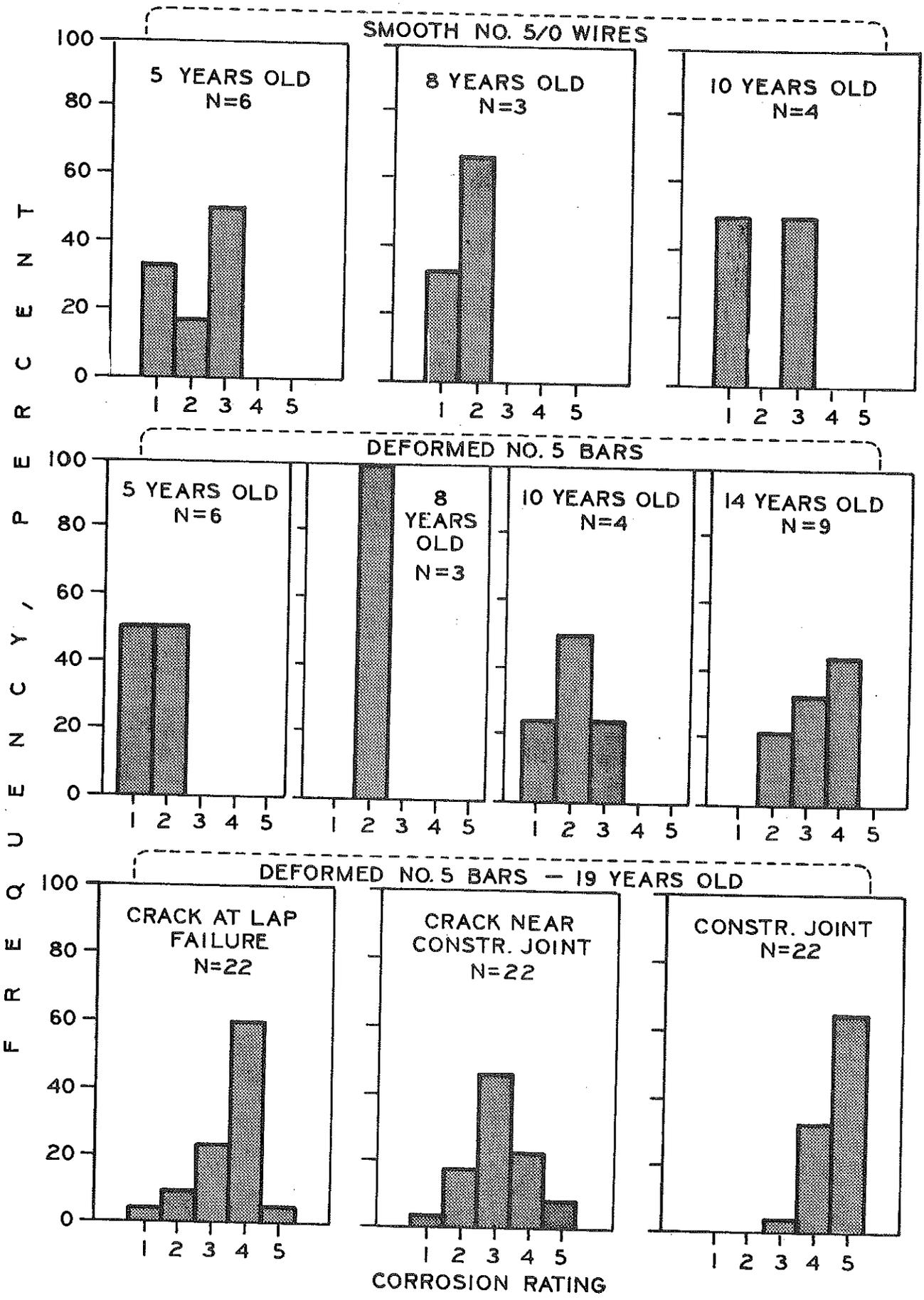


Figure 2A. Corrosion ratings of 5 to 10-year old smooth No. 5/0 wires and of 5 to 19-year old deformed No. 5 bars.

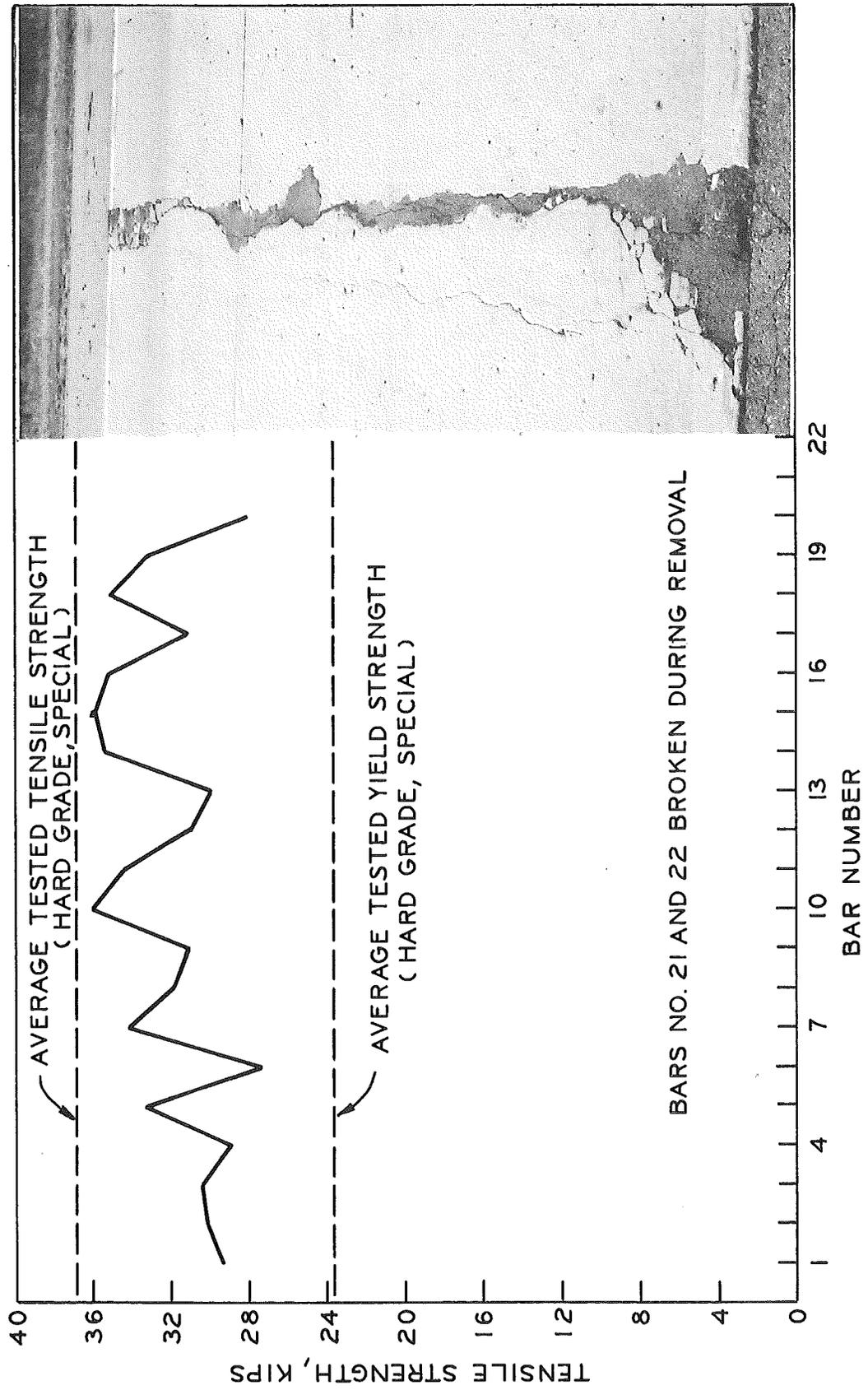


Figure 3A. Tensile strength of deformed No. 5 bars after 19 years service (Sta. 999+50) through lap failure shown at right.

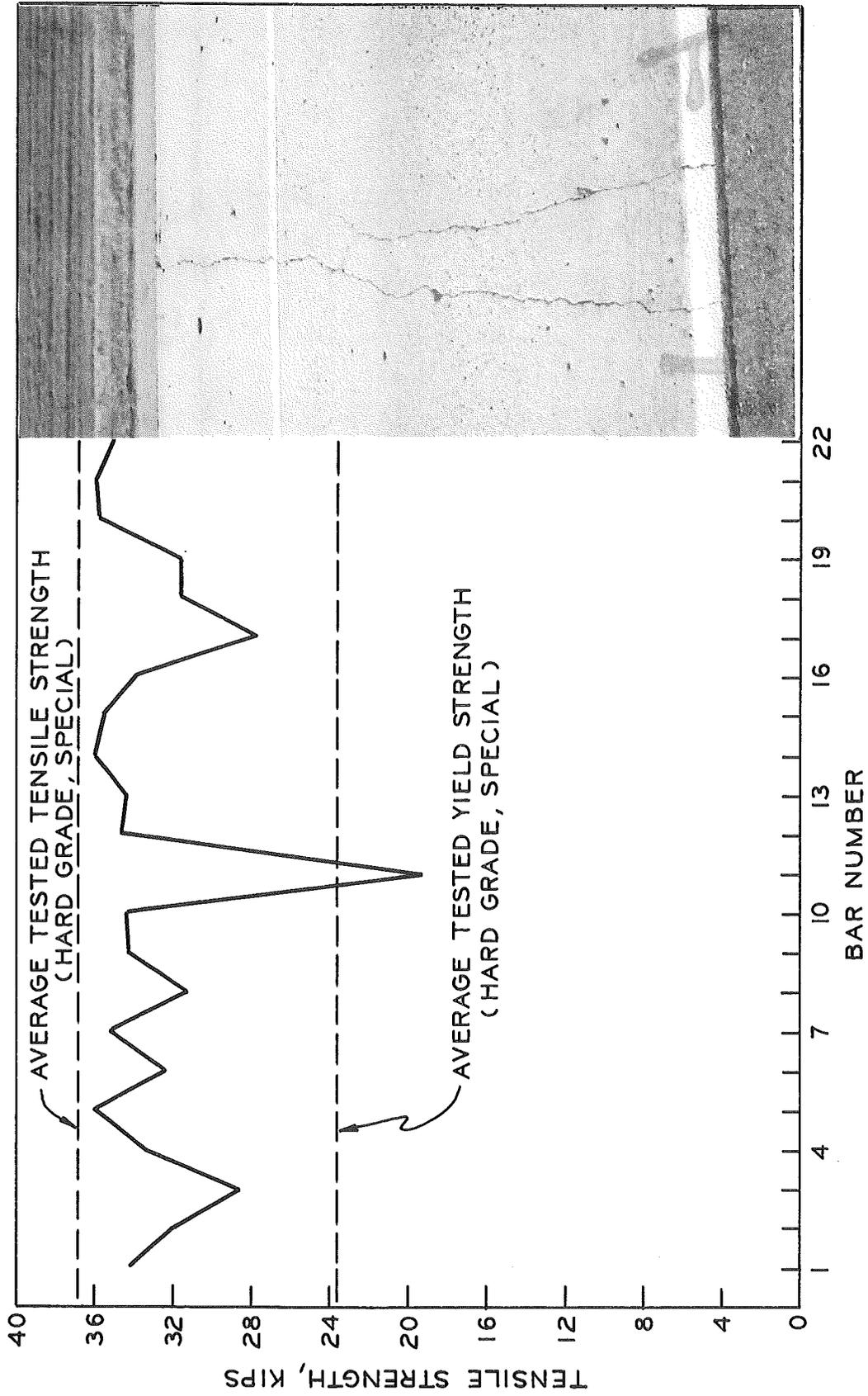


Figure 4A. Tensile strength of deformed No. 5 bars after 19 years service (Sta. 929+00) through crack shown at right.

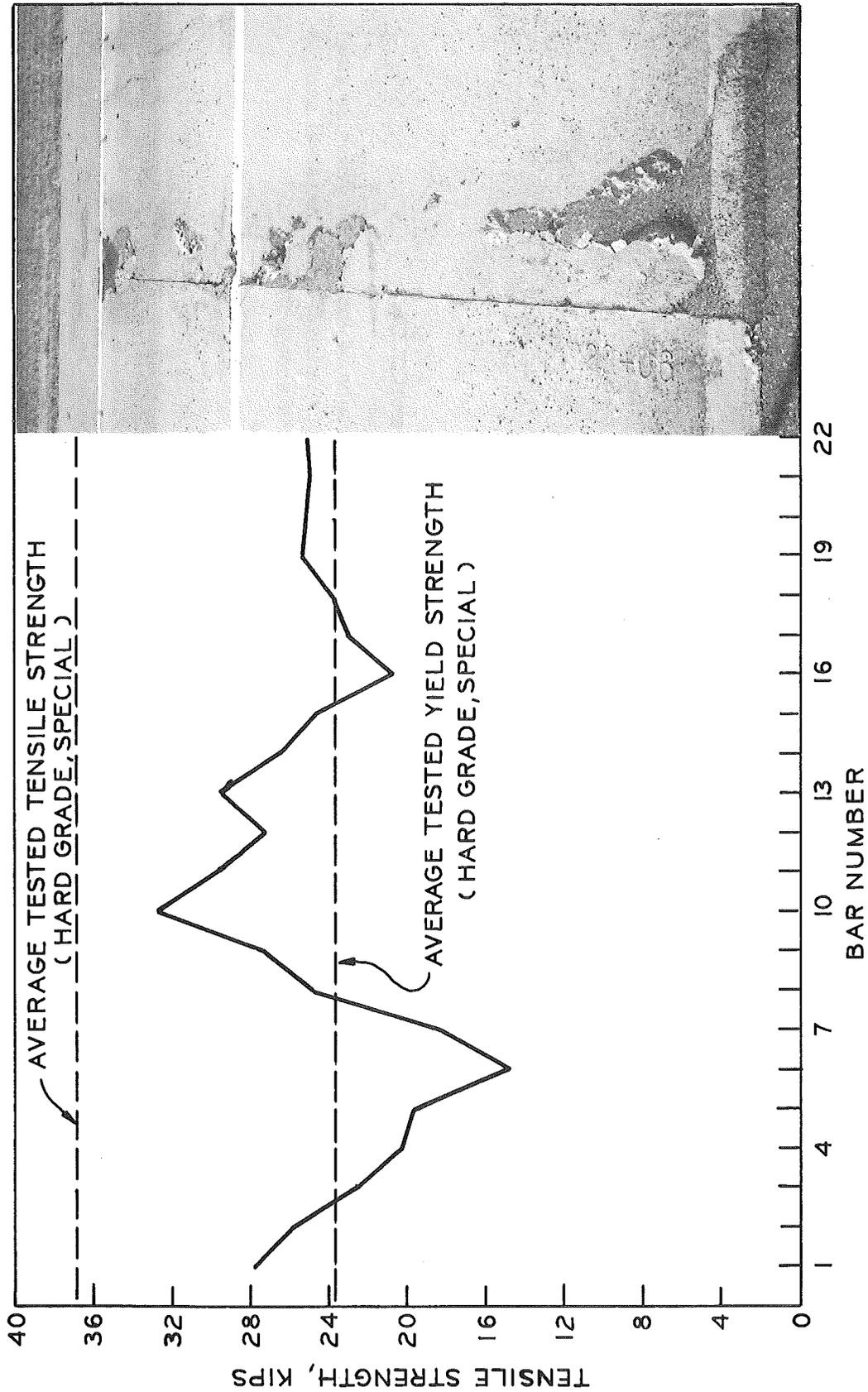


Figure 5A. Tensile strength of deformed No. 5 bars after 19 years of service (Sta. 929+06) through construction joint shown at right.

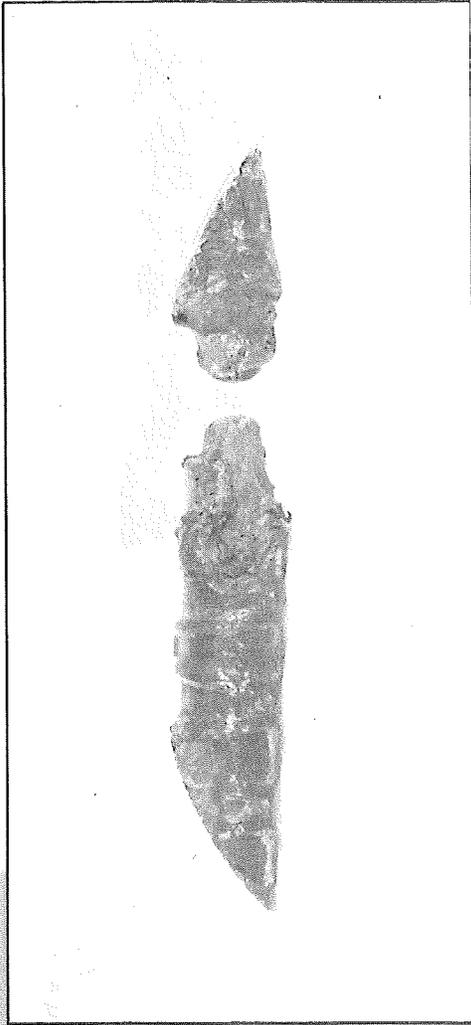
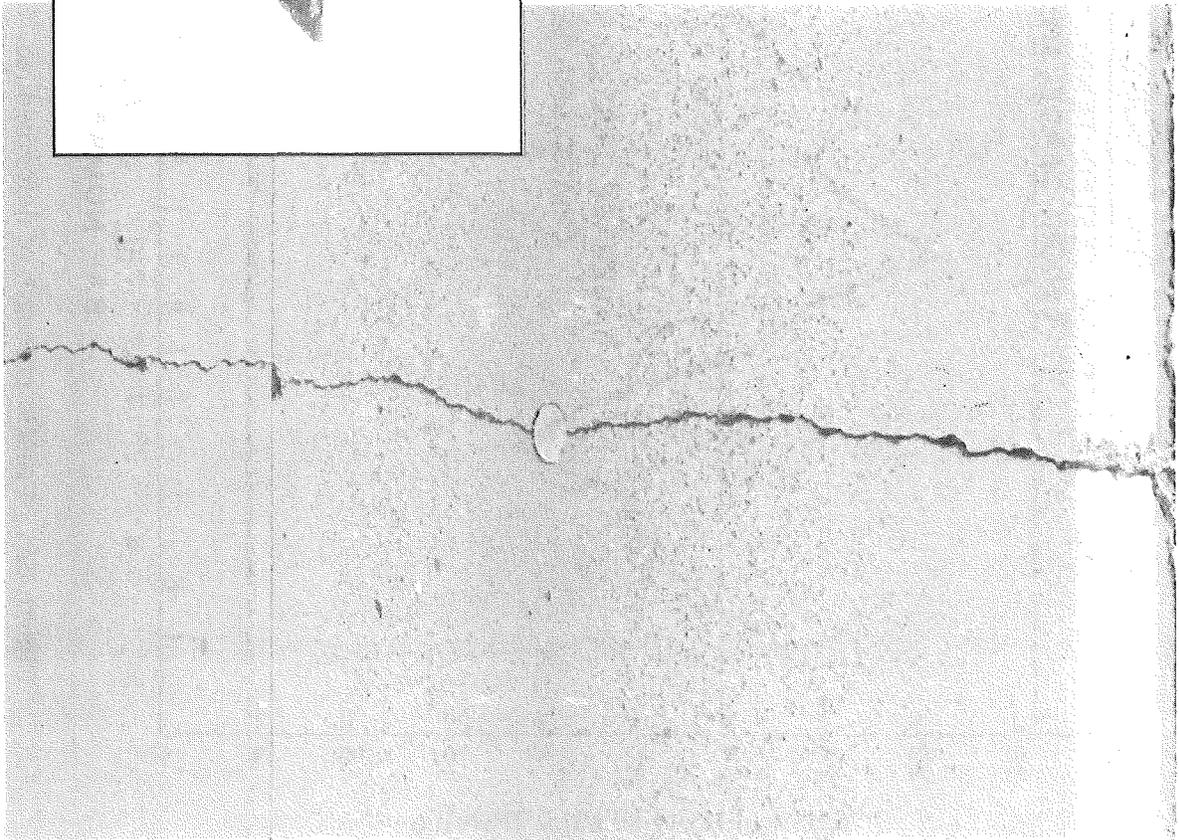


Figure 6A. Appearance of crack with fracture steel in 15 year old CRC pavement. Fractured steel sample removed from core is shown above.

At the three locations all bars broke at strengths below the 36,900 lb value obtained during testing in 1958. The average tensile strengths were 32,000, 32,900, and 24,500 lb at Stations 999+50, 929+00, and 929+06, respectively. This corresponds to a 13, 11, and 34 percent reduction in average tensile strength. At the construction joint, 32 percent of the bars broke at strengths below the average yield strength of the bars before installation.

I 96 (From Phillips Rd to Meridian Rd) - On this section of CRC pavement, constructed in 1962, the one roadway was reinforced with smooth welded wire mesh, which began to fail in the laps shortly after construction and since the use of this type of reinforcement was discontinued, no effort was made to check the corrosion of the steel on this portion of pavement. On the other roadway periodic inspections of its performance were made, and on a survey in late 1977 several wide cracks were discovered. Subsequently, cores were taken through four of these wide cracks and it was found that complete steel fracture had occurred. Figure 6A shows a typical crack with fractured steel and the fractured steel sample removed from the core.

A survey conducted during cold weather in the winter of 1978 revealed 14 cracks with fractured steel and seven where failure appeared imminent on the 6 mile section of bar reinforced pavement. One factor contributing to the failures is the large crack spacings found next to the failure cracks. The crack spacings adjacent to the four cracks where cores were taken are:

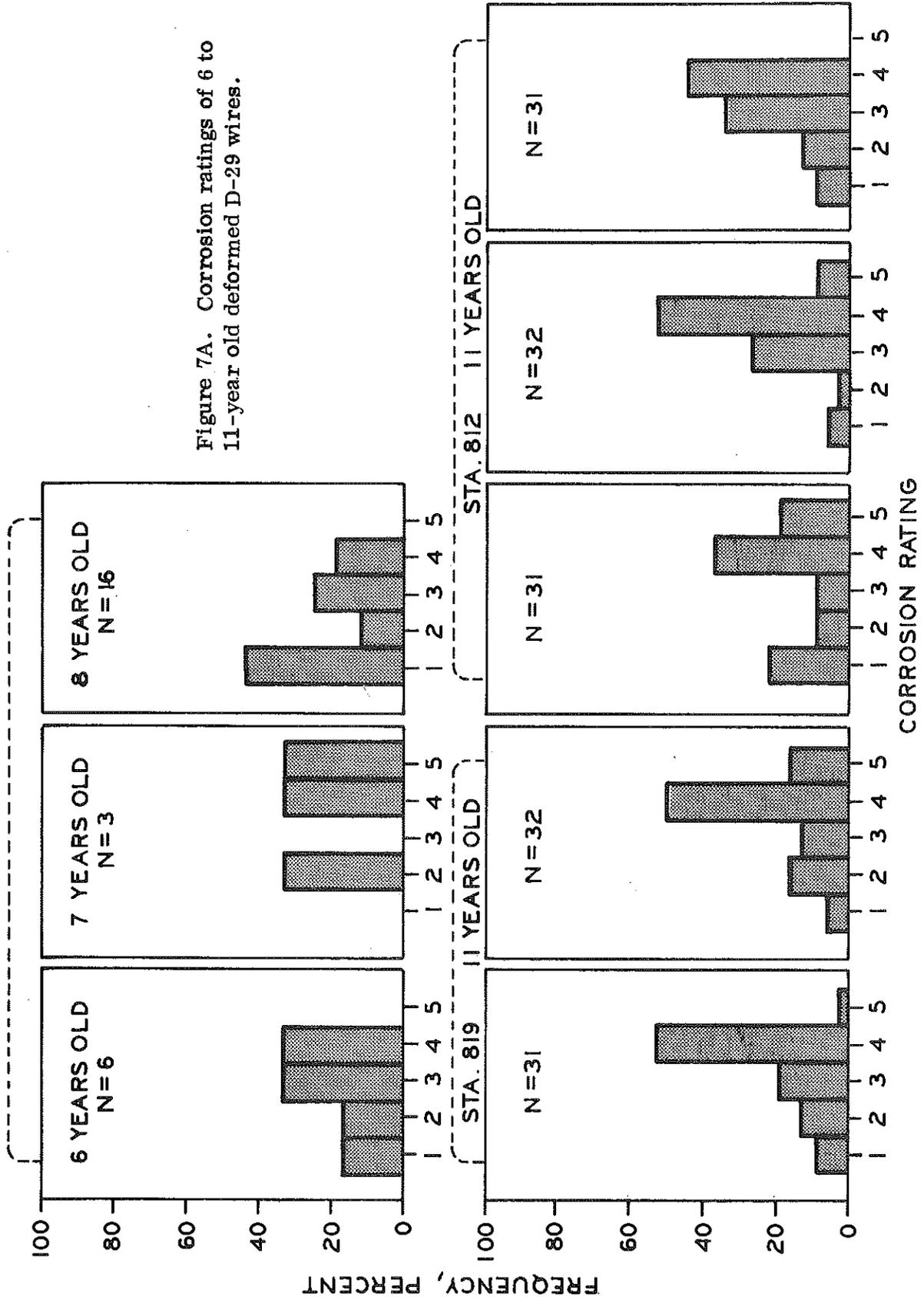
Location 1.	7 ft-4 in.,	8 ft-1 in.,	fracture,	6 ft-0 in.,	9 ft-4 in.
Location 2.	4 ft-6 in.,	7 ft-4 in.,	fracture,	10 ft-3 in.,	12 ft-6 in.
Location 3.	5 ft-4 in.,	4 ft-9 in.,	fracture,	10 ft-8 in.,	10 ft-3 in.
Location 4.	2 ft-10 in.,	11 ft-0 in.,	fracture,	7 ft-11 in.,	5 ft-6 in.

The rather large crack spacings result in wider crack openings which allow saltwater easier access to the steel. Thus, the rate of corrosion is increased which resulted in fractures earlier than expected based on the 20 year performance of the CRC pavement on I 96 near Portland.

I 75 (Pennsylvania Rd to Gratiot Ave) - On I 75 steel samples were removed by coring in 1973 on pavement sections six, seven, and eight years old. Both deformed wires and deformed bars were sampled. In 1977 the reinforcement at two lane repairs in the deformed wire reinforced section was removed for testing and the bar reinforcement at one repair location was also saved for testing. The deformed wire reinforcement had been in service 11 years when removed and the bars had been in use for 12 years.

The steel samples were rated with respect to the degree of corrosion that had occurred. The results of the rating are shown on the top portion

Figure 7A. Corrosion ratings of 6 to 11-year old deformed D-29 wires.



of Figure 7A for steel samples of deformed wire six, seven, and eight years old. On the lower left portion of Figure 7A is the results of wire samples removed from two consecutive cracks at the repair at Station 819 and the results from three consecutive cracks at Station 812 are shown on the bottom portion of Figure 7A. The condition of the pavement, from which the samples were removed, is shown on Figures 8A and 9A before repairs were made.

As shown on the graphs for six, seven, and eight year old samples, 28 percent of the samples were rated 4 and 5 (severe and critical corrosion). For the samples removed from consecutive cracks at Station 819 and 812 58 percent of the samples were rated 4 or 5. Thus, the degree of corrosion has apparently increased during the four year period since the first samples were taken.

The results of tensile tests on wire samples and the condition of the pavement from which the steel was removed are shown in Figures 8A and 9A for Stations 819 and 812, respectively. The wires are numbered from the shoulder edge toward the pavement centerline joint. On the graphs, the guaranteed minimum yield and tensile strength of the wires are shown. As can be seen at both locations the tensile strength of some wires is below the minimum and a few wires tested out below the yield strength. The percentage of wires below the minimum tensile strength at Station 819 is 25 at crack 1 and 39 at crack 2. Two wires at crack 1 and one at crack 2 had strengths below the minimum yield strength. At Station 812, 26, 31, and 19 percent of the wires at cracks 1, 2, and 3, respectively, were below the minimum tensile strength. Three of the wires at crack 1 and three at crack 2 had strengths below the yield strength.

The corrosion ratings of deformed bar samples are shown on Figure 10A. For the seven and eight year old samples about 40 percent were rated 4 and 5. The 30 bars removed from a repair at Station 562 were all rated 4 and 5. The tensile test results are shown on Figure 11A. The bars are randomly numbered and two bars were broken during removal. The samples are from the middle lane of a three-lane pavement and the traffic lane had been repaired two years earlier. As shown on the graph, 85 percent of the bars had tensile strengths below the guaranteed minimum, and the tensile strength of four bars was below the minimum yield strength.

The relationship between average tensile strengths and corrosion ratings is shown in Figure 12A. On the basis of the graphs, it can be seen that corrosion ratings of 1 and 2 have little influence on the bar strengths. A rating of 3 still does not affect the strength significantly. However, ratings of 4 and 5 show much greater reductions in strength. A deformed wire

Figure 8A. Tensile strength of deformed D-29 wires after 11 years of service (Sta. 819). Condition of pavement from which the samples were removed is shown at right.

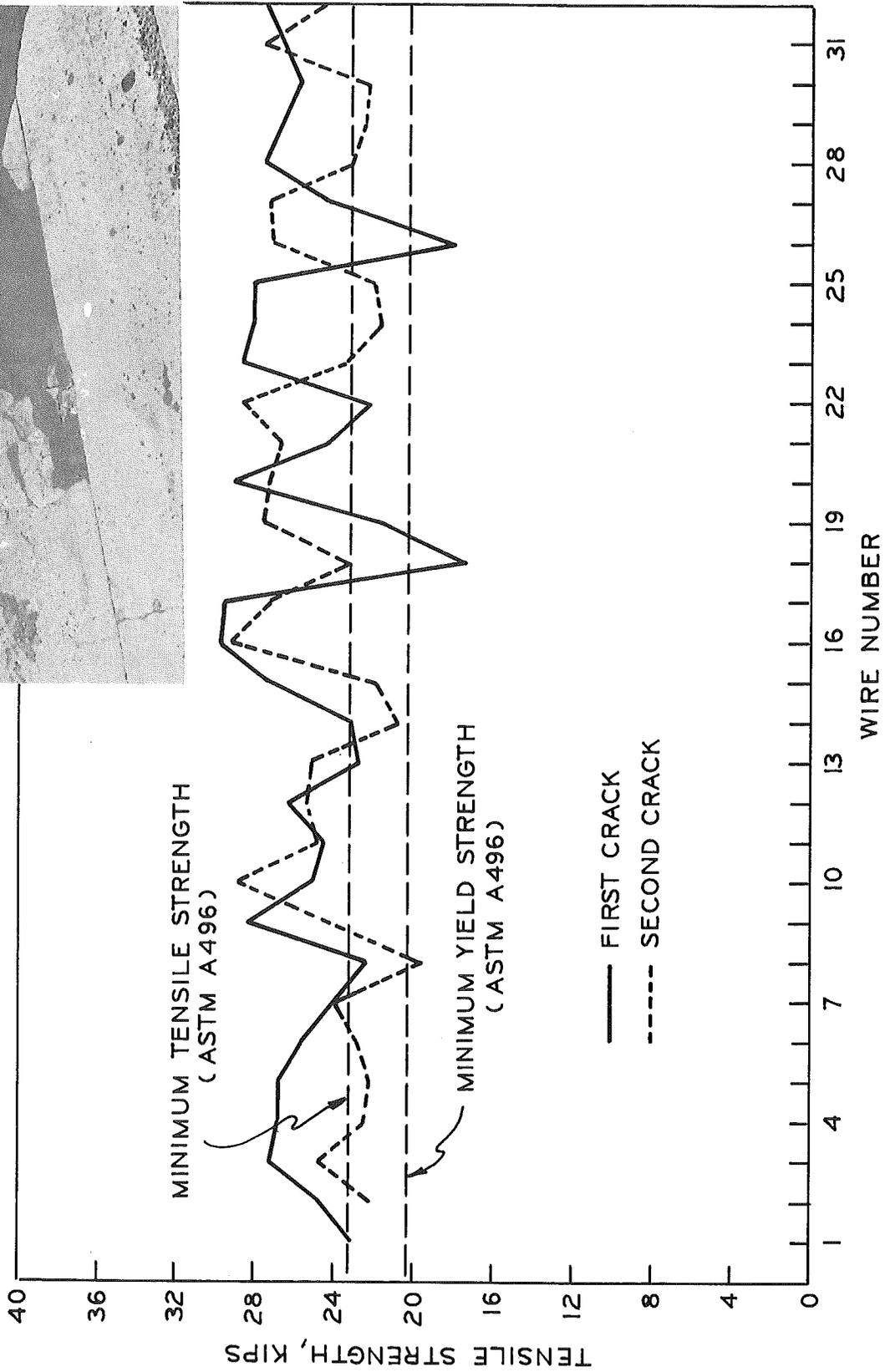
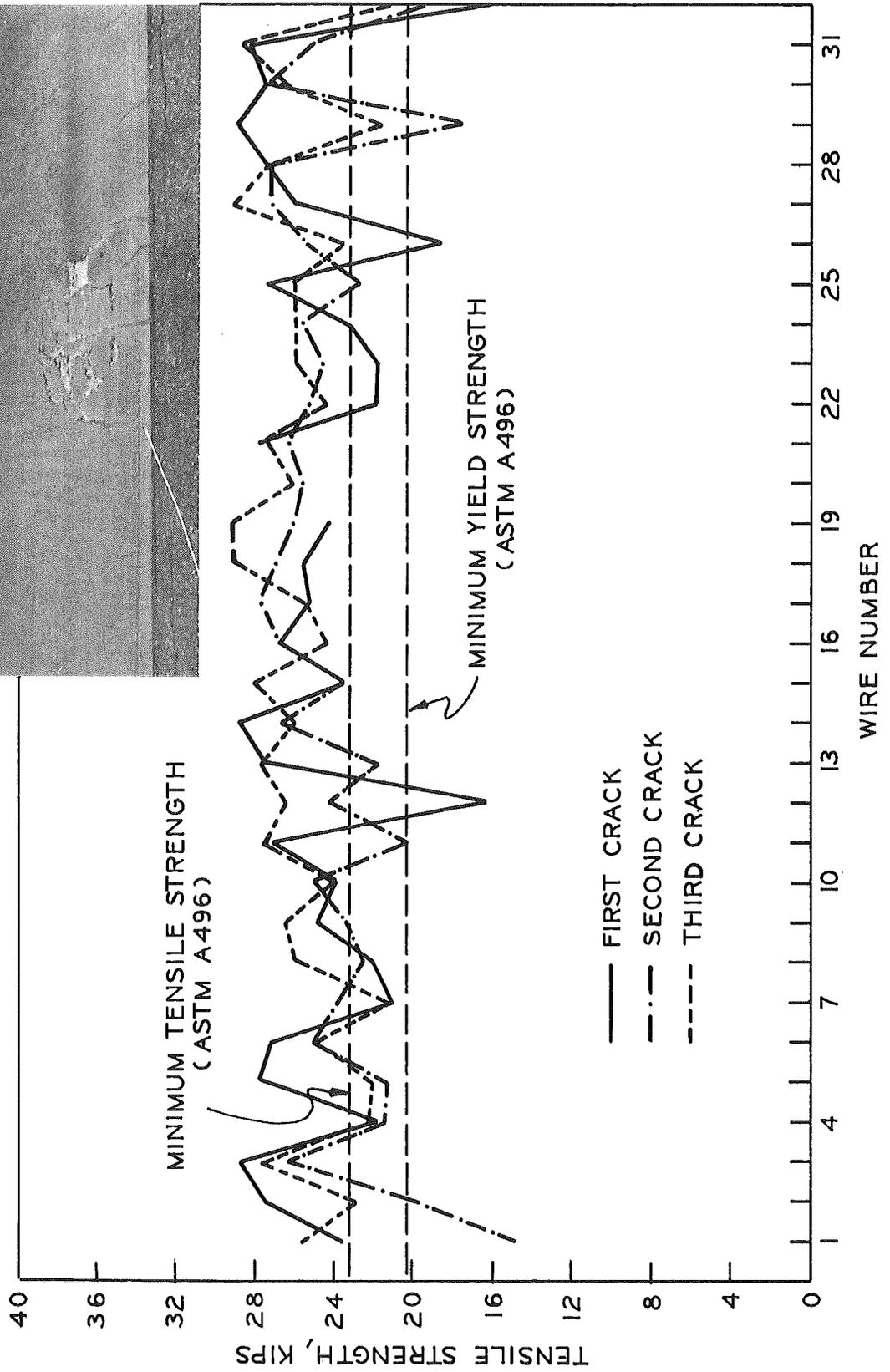


Figure 9A. Tensile strength of deformed D-29 wires after 11 years of service (Sta. 812). Condition of pavement from which the samples were removed is shown at right.



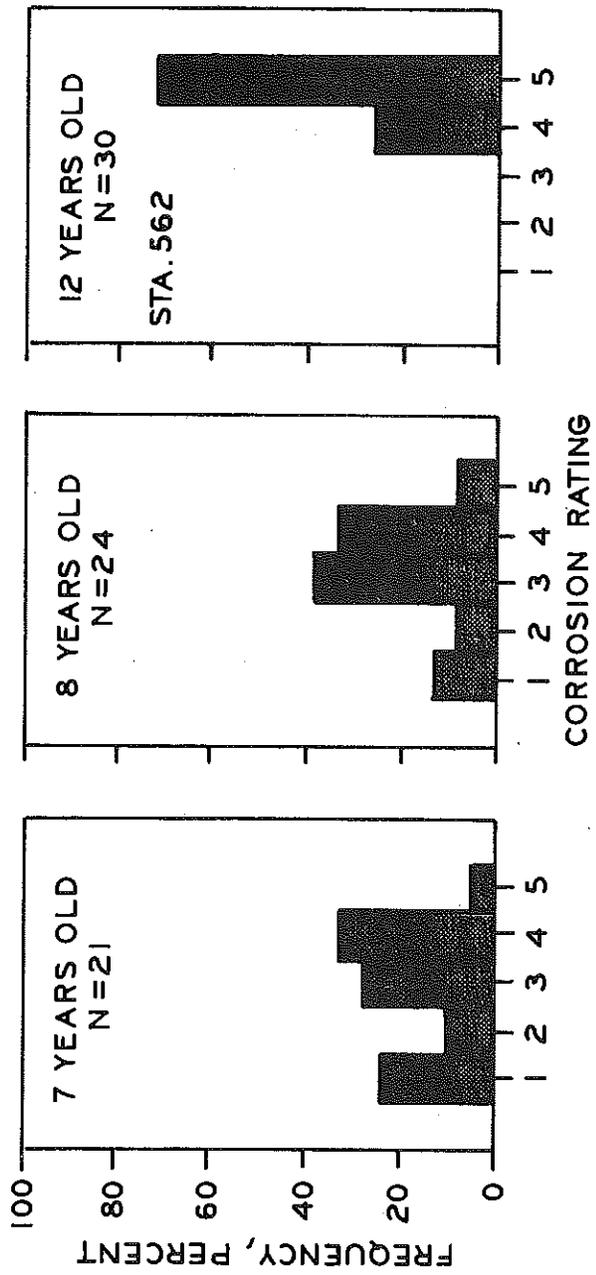


Figure 10A. Corrosion rating of 7 to 12-year old deformed No. 5 bars.

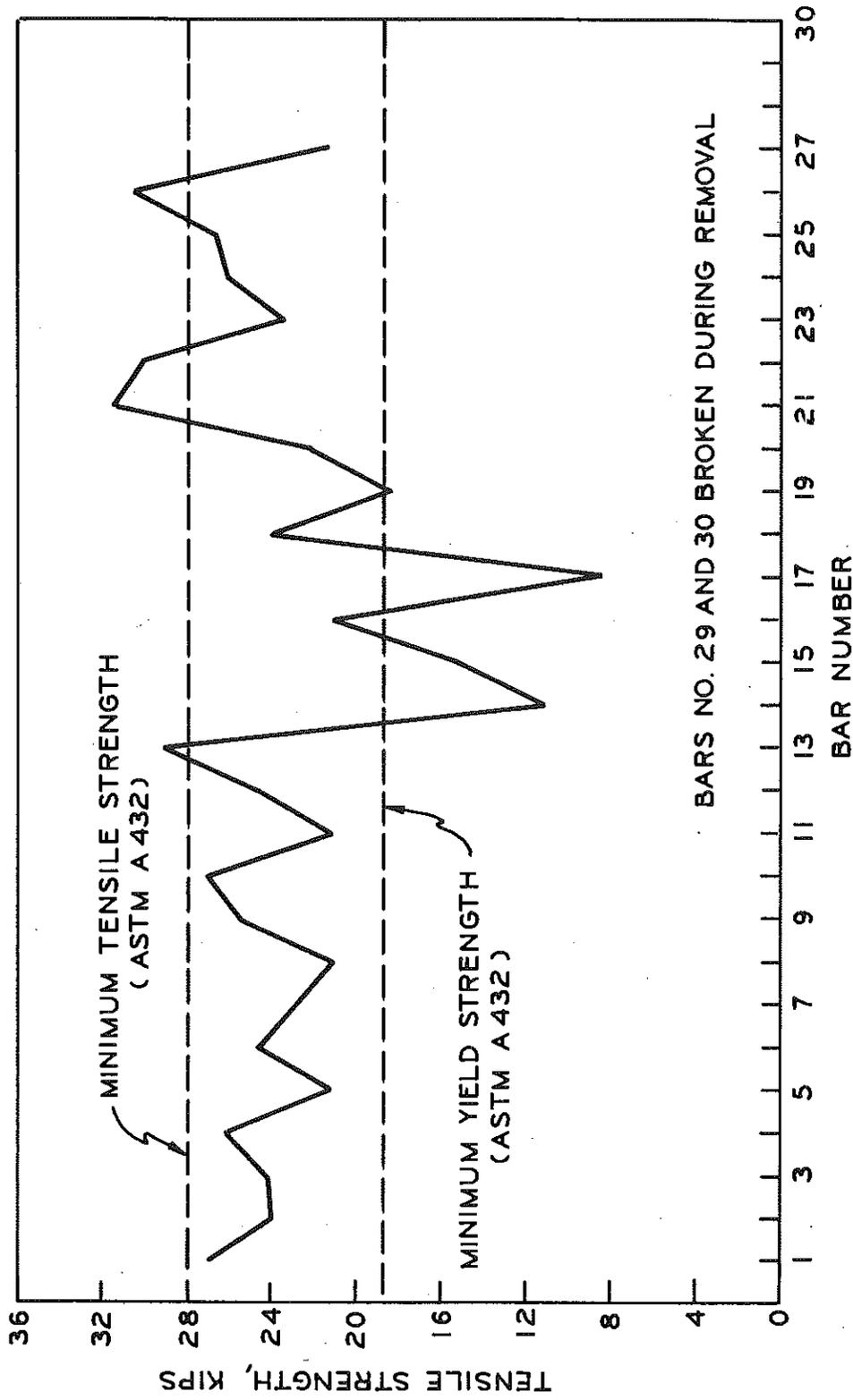


Figure 11A. Tensile strength of deformed No. 5 bars after 12 years service (Sta. 562).

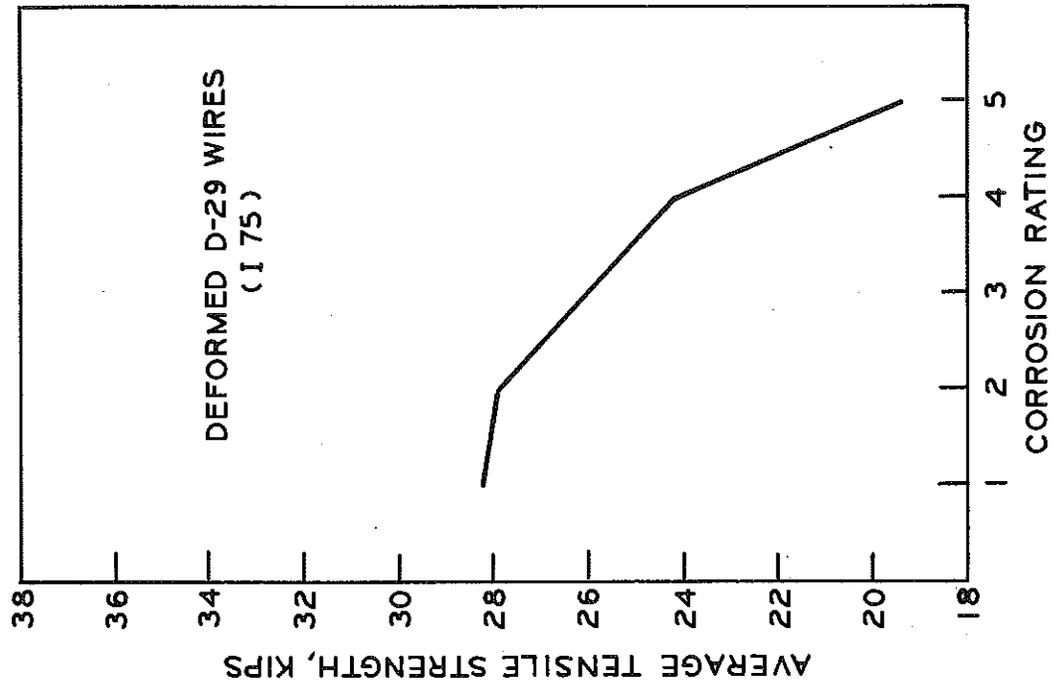
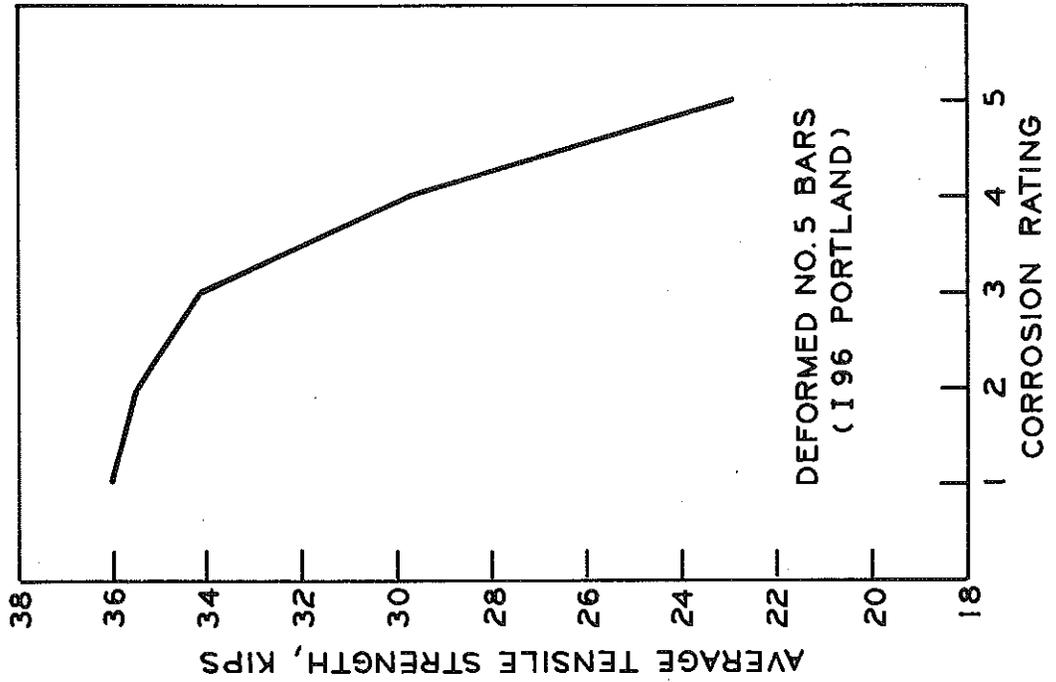


Figure 12A. Relationship of tensile strength and corrosion ratings of deformed No. 5 bars from I 75 and I 96.

with a 5 corrosion rating has, on the average, lost 32 percent of its ultimate tensile strength compared to one with a rating of 1 (no corrosion). For the bars from I 96 near Portland an average loss of 36 percent in tensile strength had occurred for bars with a 5 rating compared to bars with a 1 rating.

The average length of corrosion on each side of a crack was found to be about 3 in. The range in length of the deformed wire samples was from 2.6 to 12.5 in. and for the deformed bars the range was from 1.2 to 12.0 in. Since rusting increases the bar or wire volume the corrosion of the steel creates stresses in the concrete tending to split the slab apart at the steel level.

On the basis of the information presented on corrosion of the steel in CRC pavements on I 96 and I 75, it is evident that corrosion is a serious problem. Although no steel fractures have occurred on the 20 year old pavement near Portland, 14 failures were noted on the I 96 pavement near Okemos when it was 15 years old. On I 75, 48 percent of the deformed wires from the 11 year old pavement was given a corrosion rating of 4, and 9 percent a rating of 5. At the 12 year old sample location of No. 5 deformed bars 27 percent were rated 4 and 73 percent rated 5. It should be noted that once corrosion of the reinforcement occurs, the reduction in cross-sectional area results in increased tensile stress in the steel. The increase in tensile stress will increase the rate of corrosion. Thus, the corrosion of the reinforcement may proceed at a fairly slow rate during the first few years, but then increase at a much faster rate resulting in early unexpected failures.

Another factor that is very significant in the corrosion of the reinforcement is the use of salt for removing snow and ice from the pavement. The amount of salt applied per equivalent two-lane mile of pavement in the areas from which steel samples have been examined is shown in Figure 13A for the winters 1963 through 1979. The average tonnage applied per year per equivalent two-lane mile in Wayne County and in Districts 5 and 8, is 47, 19, and 22, respectively.

The corrosion of steel reinforcement in concrete is an electrochemical process and depends on a number of variables. In a CRC pavement the more than 1,200 transverse cracks per mile, provide direct access for salt laden moisture and oxygen to reach the steel. The salt provides the source of chloride ions which release ferrous ions at the anodes, increases the conductivity of the electrolyte enhancing the corrosion current, and affects the passivity normally afforded the reinforcement by the high alkalinity of the concrete environment. It is of interest to note that even in the examination of all reinforcement bars through the same pavement lane crack, the degree of corrosion is highly variable.

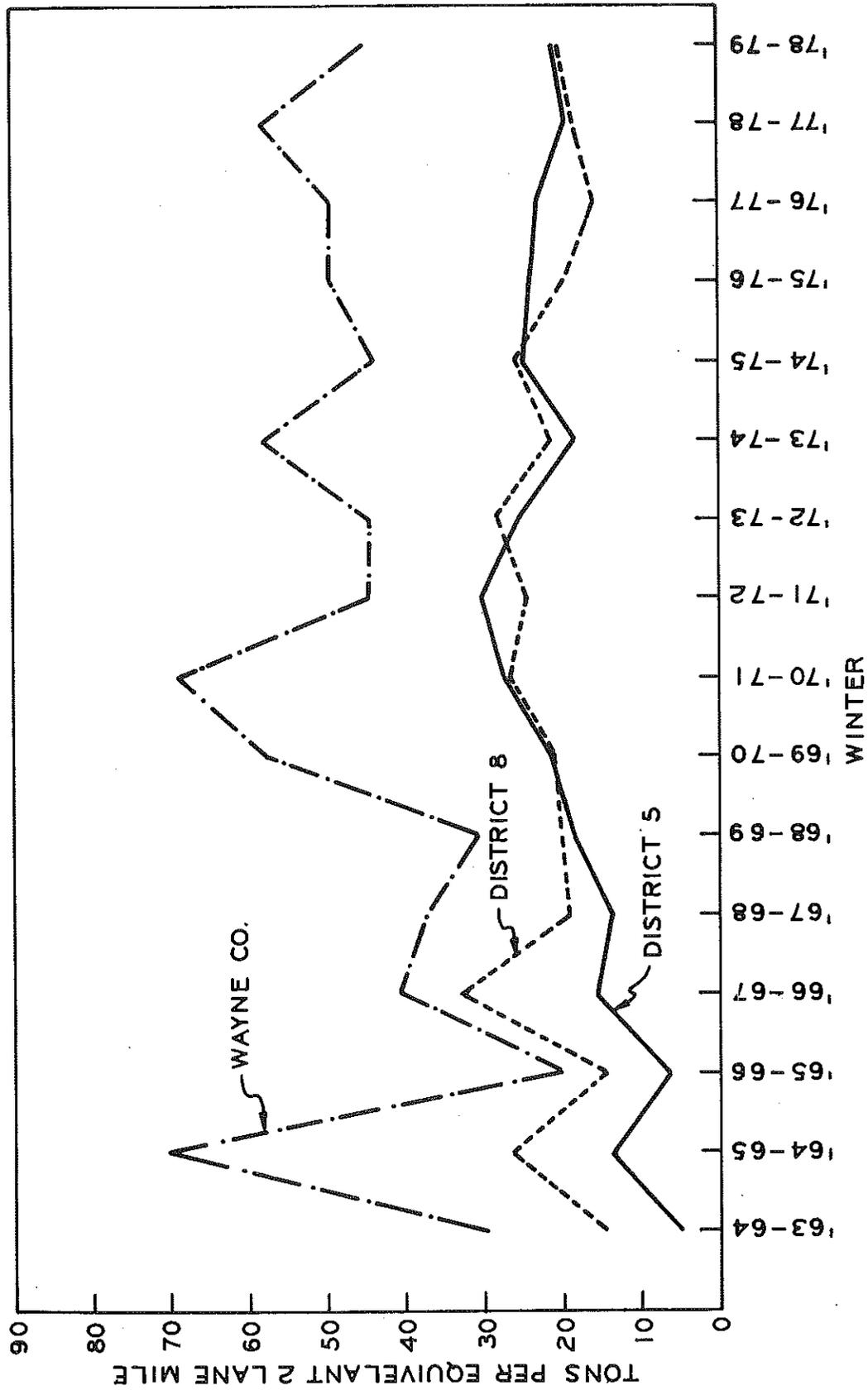


Figure 13A. Tons of salt applied per equivalent two-lane mile in Wayne County and in Districts 5 and 8 (from Maintenance Division records).