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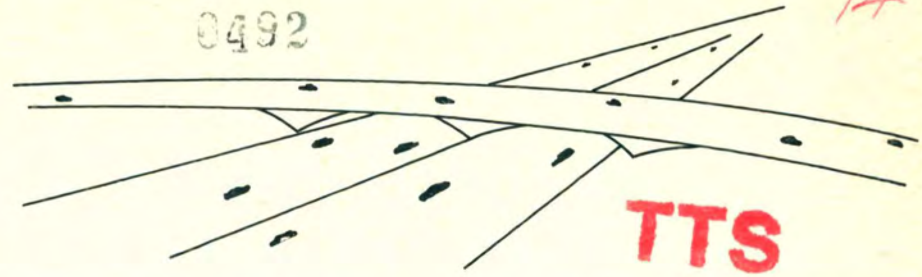

DEPARTMENTAL RESEARCH

FOURTH YEAR PROGRESS REPORT ON EXPERIMENTAL C R C P IN WALKER COUNTY

by

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and

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TEXAS HIGHWAY DEPARTMENT

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I. INTRODUCTION

During 1960, an experimental continuously reinforced concrete pavement eight inches thick was constructed in Walker County on IH 45 Project No. I-45-2(3) 102. The experimental nature of the project was to evaluate the relative performance of two different percentages of longitudinal steel, i.e. 0.5 percent (No. 5 @ 7½ in. c-c) and 0.6 percent (No. 5 @ 6½ in. c-c).[1] Each directional roadway was divided equally between the two steel percentages. In addition to the stress study, another experimental factor was the use of a minimum cement factor of four sacks per cubic yard in lieu of the normal minimum of five sacks per cubic yard.

In an attempt to evaluate the effect of several parameters on average crack spacing, sections were selected over the project that encompassed the parameters under study, in addition to the 0.5 percent and 0.6 percent parameters.[2] Two test sites were also selected for making longitudinal steel stress studies for each percentage.[2] The steel stress study was discontinued in 1961, but a summary of the findings is included in this report. Several interim reports have presented the influence of the selected parameters on average crack spacing, and a follow-up on this work will be presented herein.[3,4,5]

Several deflection investigations have been conducted on this project to evaluate irregularities and determine the effect of percent longitudinal steel on deflection. A limited amount of this data is included. The general performance of the pavement structure will be covered with an explanation of the irregularities found on this project.

II. STRESS

A detailed analysis of the steel stress study and its conclusions may be found in Departmental Research Report 62-1. It was found, as would be expected, that the steel stress and the concrete movement is greater at the crack than in the area between cracks. This study indicates that the stress in the longitudinal steel is a function of the cement type, the slab temperature, the average crack spacing, and the percentage of longitudinal steel.

The cracking in concrete with a Type III Cement was found to be of an explosive nature relative to that in concrete with a Type I Cement. During the early curing period, the pavement with the Type III Cement had three to four times as much longitudinal steel stress as pavement with Type I Cement. The stresses in the two pavements tend to approach one another after the concrete curing period, but the early life stress differential is of such magnitude that the use of Type III Cement should be banned on continuously reinforced concrete pavement.

The stresses in the steel were of a cyclic nature and for any given 24-hour period, the steel stress could be correlated in a linear fashion with a temperature change. The tensile stresses of the steel were found to increase as the pavement slab temperature decreased. Although there was an excellent correlation between slab temperature and steel stress, the correlation varied as the pavement aged. This variation in the stress-temperature correlation was found to be a function of the change in the average crack spacing. It was found that with all other factors constant, the steel stress decreased as the average crack spacing decreased. In relation to the steel percentage, it was found that the steel stress decreased as the percentage of steel was increased. At no time during the measurement period of observation did the steel stresses exceed the yield strength of the steel (50,000 psi). It was found that the factors previously discussed have more effect on steel stress than the percent of longitudinal steel.

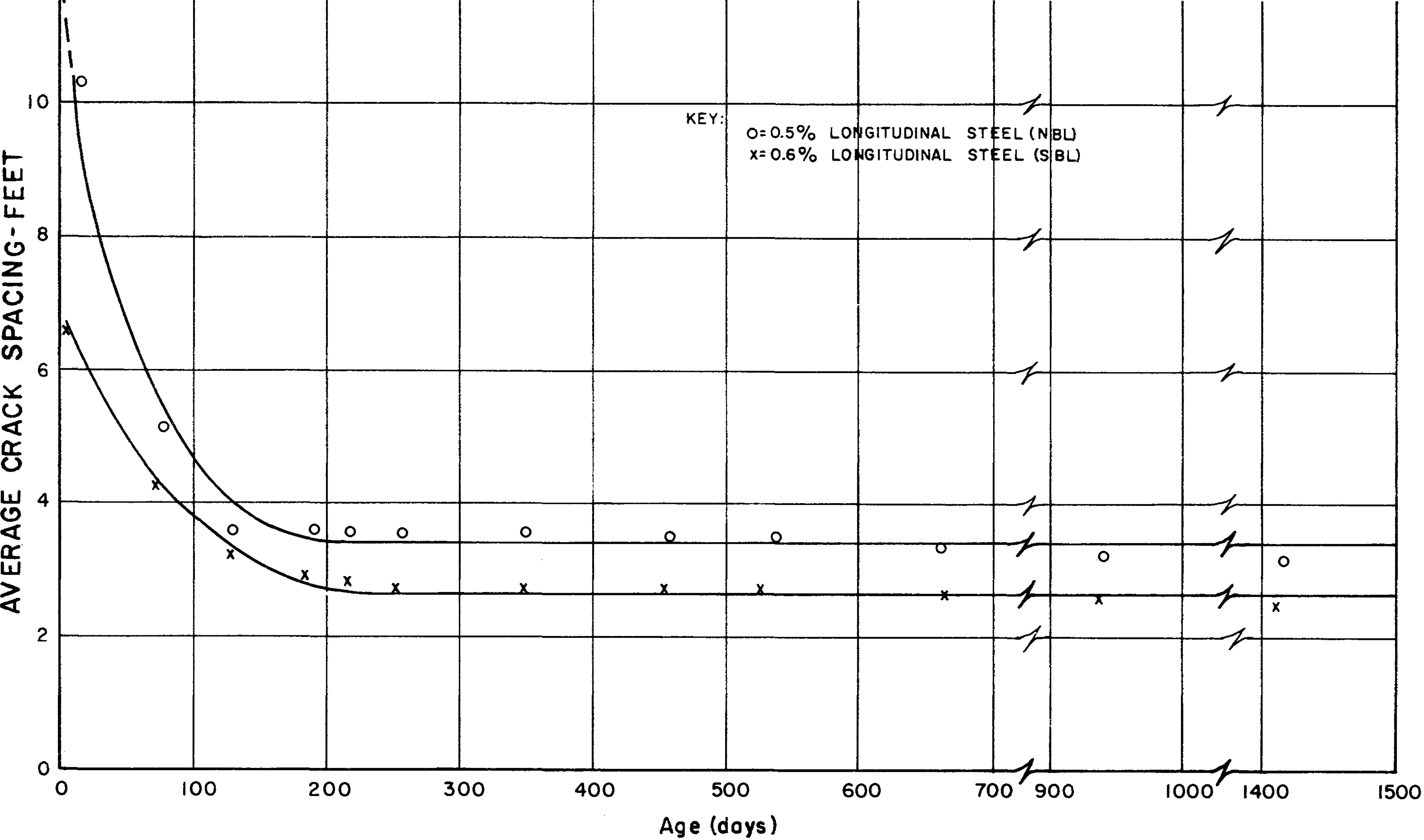
III. CRACK PATTERN OBSERVATIONS

To evaluate the effect of percent longitudinal steel on average crack spacing, two test sections of approximately

2,000 feet each were selected and periodic crack surveys have been conducted since the construction of the project. Figure 1 shows the age-crack spacing relationship for these two test sections. The data shows that there was a rapid initial decrease in the average crack spacing, but no noticeable change has occurred for either percent steel since an age of 200 days. The present age is a little more than four years. The average crack spacing for the 0.5 percent steel is 3.3 feet and 2.6 feet for the 0.6 percent steel. Therefore, from the standpoint of average crack spacing, both percentages of steel are performing equally well. In the original design concept for CRCP, it was felt that the average crack spacing should not exceed eight feet.[6]

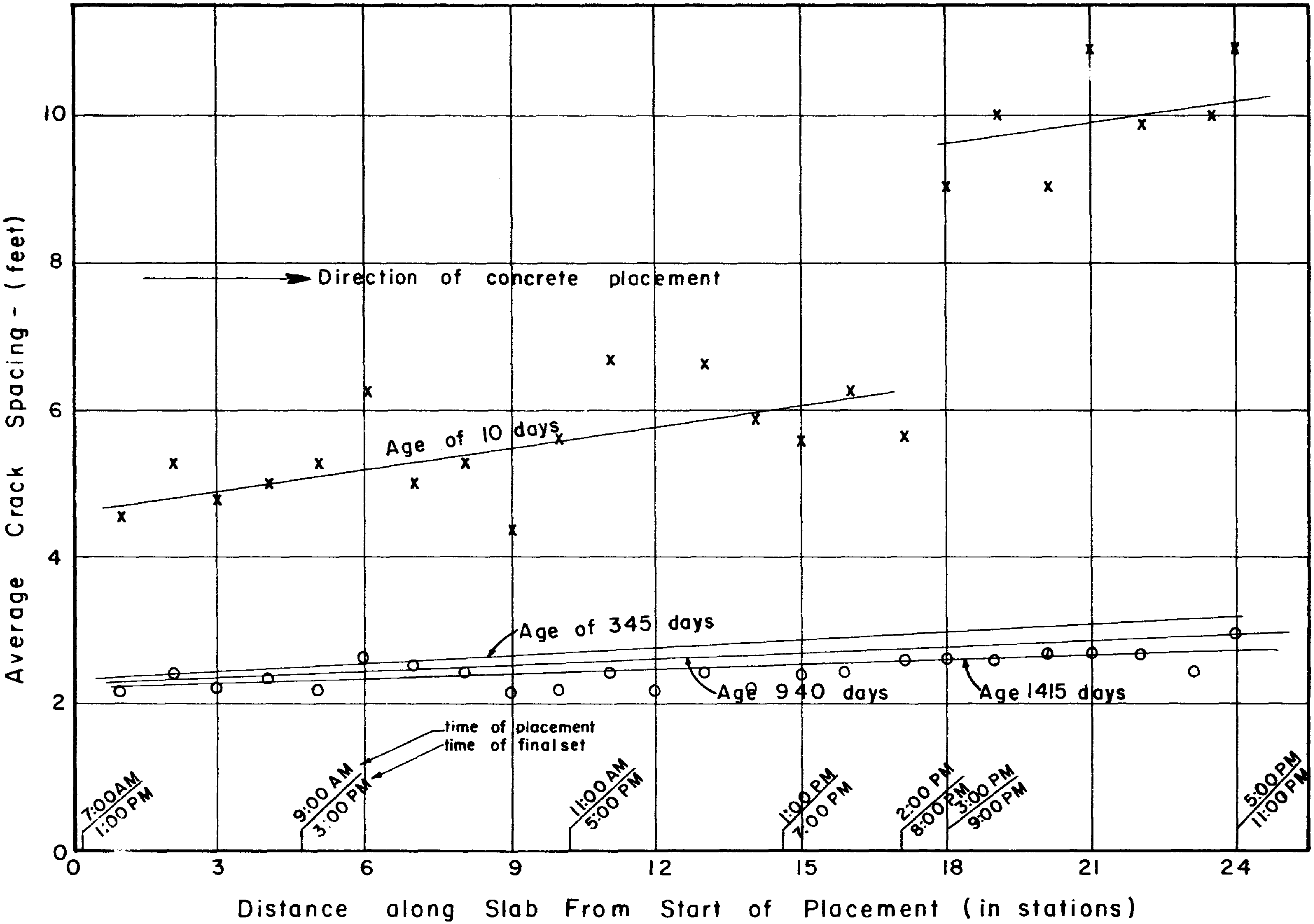
Earlier studies had indicated that the relative position within a slab had an effect on the average crack spacing. The data indicated that for any given day's placement the average crack spacing increased in the direction of concrete placement or paving. Figure 2 shows this relation for four different time periods. It is apparent that time is nullifying the effect of relative position since the trend line is approaching a horizontal position with increasing age.

Figure 3 shows the relation between average crack spacing and approximate curing temperature for eight study sections located at various positions over the project. As was the case previously, age is tending to nullify the inverse relationship in that the trend line is approaching a horizontal.



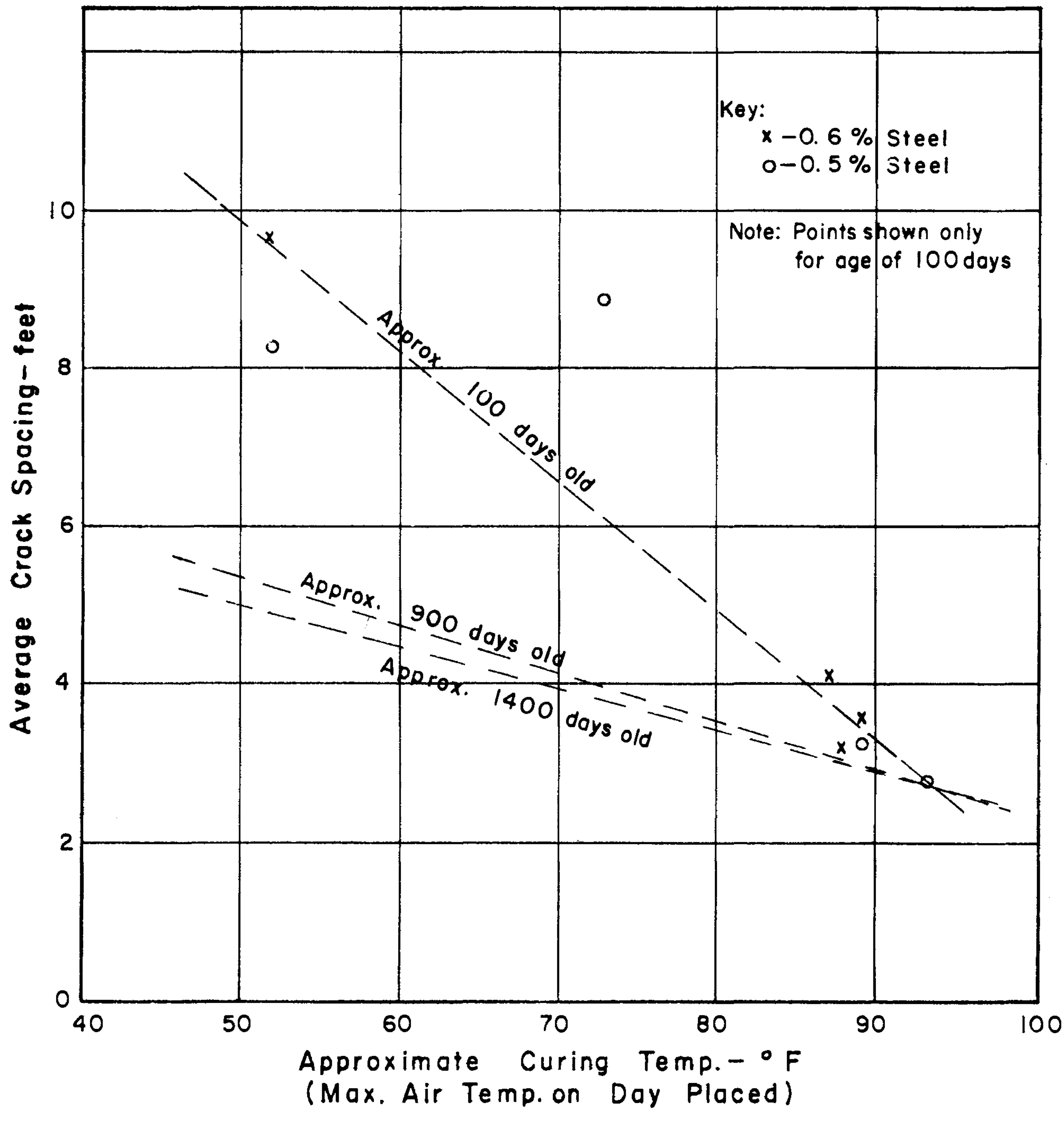
RELATIONSHIP BETWEEN AGE AND THE AVERAGE CRACK SPACING ON THE TEST SECTIONS

Figure 1



EFFECT OF RELATIVE POSITION WITHIN A SLAB ON THE AVERAGE CRACK SPACING - SBL (0.6%)

Figure 2



AVERAGE CRACK SPACING VERSUS APPROXIMATE CURING TEMPERATURE FOR STUDY SECTIONS

Figure 3

IV. DEFLECTION

Deflection studies have been used on this project for two different purposes. One study in 1962 was to investigate the irregularities on the project. The other phase consisted of determining the effect of percent longitudinal steel on deflection.

Shortly after this project was open to through traffic, several irregularities were noted in the vicinity of the construction joints at numerous locations over the project. A survey of the project revealed that on the down placement side of the construction joint raveling was occurring at the crack and edge deposits were forming on the shoulder at several locations. An extensive testing program was initiated to investigate the cause of these irregularities. This study showed that the deflection in the problem areas was considerably higher than at other locations which were performing satisfactorily. In comparing the data to the AASHO Road Test deflection data[7], it was found that the irregular sections were deflecting similar to a 5.5 inch Road Test Pavement; whereas, the satisfactorily performing sections were deflecting similar to a 9.5 inch Road Test Pavement. These findings led to an extensive sub-surface investigation; the results of which will be discussed later in this report.

In regard to determining the effect of percent steel on deflection, four test sections were selected over the project to give a range of support conditions. Two test sections, one of each percentage of steel, were selected on a relatively good subgrade (Triaxial Classification 4.5). The other two sections were selected on a relatively poor subgrade (Triaxial Classification 6.0). The data for this study is presented in Table 1. Note that the percentage of longitudinal steel has a slight effect in that a greater percentage of steel has a smaller deflection, but the differences are small. There is no apparent trend for edge deflection between cracks. This data shows that subgrade support has as much influence on deflection as the range of longitudinal steel used on this project. (Caution should be used in extrapolating this data below 0.5 percent longitudinal steel.)

Deflection Type	Percent Steel	Character Of Subgrade Support: Poor	Good
Crack	0.5	10.6[a,b]	8.7
	0.6	8.8	8.7
Edge	0.5	9.6	8.0
	0.6	9.6	8.1

(a) Deflection in inches $\times 10^{-3}$

(b) Deflection corrected to a zero degree slab temperature differential. Each figure shown is the average of 20 or more seperate measurements.

Data taken in November, 1963

EFFECT OF PERCENT LONGITUDINAL STEEL AND SUBGRADE SUPPORT ON DEFLECTION

Table I

V. PERFORMANCE

The roadway has excellent riding qualities in comparison to the jointed concrete pavement projects to the north and south of it,[a] but several alarming developments have been experienced with the pavement. In the 22 miles of roadway under observation, a total of 35 failures have occurred. The term, failure, is being used to describe a serious disintegration of the pavement structure; whereas, deterioration refers to an area that is beginning to show performance characteristics different from adjacent sections, i.e. crack raveling, edge deposits, etc. For the purpose of classification, the failures have been designated as either "construction joint failures" or "between construction joint failures". Of the 35 failures, 16 have occurred at the construction joint and 19 between the construction joints. Photographs of the majority of the failures may be examined in Appendix A.

At the first experience of failure, there was a tendency among many engineers to attribute the failure to either the difference in percentage of longitudinal steel or unequal distribution of traffic on the roadway. Table 2 shows that both steel percentages have identical performance records since the failures are equally distributed between

(a) The present serviceability index for the sections as determined by the Chloe Profilometer ranges from 3.8 to 4.1.

roadways. It is pointed out here that all four sections of the roadway are equal in length.

Table 2
Steel Design vs. Number of Failures

Steel Design	Northbound Lane	Southbound Lane
0.5%	13 Failures	4 Failures
0.6%	4 Failures	14 Failures

Construction Joints Failures

Shortly after the project opened to traffic in 1961, failures were detected at a considerable number of construction joints. As mentioned earlier, deflection studies indicated that these problem areas had deflection characteristics similiar to a five inch pavement, and subsequent repair of these areas found the lower three to four inches of the pavement were honeycombed.[8,9] (For a detailed description of the findings at one of these repairs, refer to Appendix B.) Hence, the effective pavement depth was from four to five inches. Since vibrators were not required on this project, serious honeycombing was found to be present in the first 20 feet from the construction joint. Due to severe longitudinal stresses experienced at these locations as a result of concrete volume changes, these failures showed up quickly after traffic was placed on the pavement.

Since 1961, a pictorial history has been kept on every construction joint in the northbound roadway. In 1961, numerous engineers inspected the project and stated that over 70 percent of the joints would give trouble, but based on a pictorial record, this magnitude of failure has not been experienced. Table 3 shows how many more deteriorations, failures and repair areas have occurred at construction joints in the northbound roadway since the preceding record photographs were taken. As can be seen, rate of deterioration has decreased markedly since 1961. There are a total of 38 construction joints in the northbound lane, and photographs for each year are available.

Table 3

Breakdown of Increases in Deteriorations, Failures, and Repair Areas

Year	Deteriorations	Failures	Repair Areas
1961	4	1	2
1963	1	2	3
1964		2	6

(The total of failure and repair area represent total construction joint failures.)

Intermediate Failures

Shortly after fully investigating the construction joint problem, failures began to appear at intermediate locations between construction joints.[10] Subsequent investigation and repair of these failures found the lower three or four inches of the concrete to be honeycombed, as was the case with the construction joint failures.[11]

These failures were also attributed mainly to the absence of mechanical concrete vibrators on this job. There are 19 of these failure types at the present time, and this number had been progressively increasing with each survey. A distinct feature of these failures has been the presence of extremely large transverse cracks directly over the transverse steel bars which are spaced at two foot centers. Since the longitudinal steel did not function as intended, the concrete simply flexed over the transverse bars and cracked at these areas; hence, exceptionally large cracks were noted. In essence there is a four inch slab supported by a steel mat.

An immediate question that comes to mind is the reason why numerous failures occur at certain locations on the project and none occur at other locations. Numerous factors such as roadway grade, direction of placement, concrete flexural strength, etc. were studied in terms of the failures, but the only positive correlation was in terms of the maximum temperature on the day of concrete placement. Table 4 reveals the bulk of the failures occurred where the maximum air temperature was 90 degrees or greater on the day of placement. This is certainly a plausible correlation since the time from placement to initial set of the concrete decreases as the curing temperature increases. Therefore, on cool days the concrete probably received full benefit of the limited vibration derived from the paving equipment before the concrete set, whereas on hot days the concrete took its initial set before full vibration could be received from the paving train. (It is pointed out again that mechanical vibrators were not required or used on this project.)

Table 4

Showing Number of Days Pavement Laid in a Particular Temperature Range and The Number of Failures in the Range

Temperature Range	Number of Days	Number of Failures
40°F. to 50°F.	6	2
50°F. to 60°F.	6	1
60°F. to 70°F.	13	2
70°F. to 80°F.	14	2
80°F. to 90°F.	21	4
90°F. to 100°F.	21	24

Another factor that would tend to compound the effect of air temperature on initial set of the concrete was the use of Type III Cement (High Early Strength) on this project. Earlier in the report reference was made to the fact that the use of Type III Cement resulted in extremely high steel stresses, consequently, its use was barred on future projects. The cement on this project had a specific surface area of 2300 cm^2/gm as measured by the Wagner Test. The greater the specific surface area, the greater the cement hydration rate with a resulting decrease in time to initial concrete set. Consequently, the combined effect of a Type III Cement and high air temperature probably resulted in numerous "flash settings" of concrete.

VI. CONCLUSIONS

On the basis of the observations thus far on the project, the following conclusions are warranted:

1. In terms of the original experimental objective of this project, it has been found that both the 0.5 percent and 0.6 percent longitudinal steel designs have preformed equally well. Therefore, a continuously reinforced concrete pavement with 0.5 percent hard grade longitudinal steel is adequate for conditions existing on this project.

2. Factors such as temperature placement conditions, cement type, air temperature, etc. were found to have more influence on steel stress, deflection, and average crack spacing than the difference in longitudinal steel used on this project.

3. Numerous failures have occurred on this project, but thus far the failures can be attributed to problems that developed during construction. It appears that the failures investigated thus far are due to a deficiency in density of the slab. Mechanical vibration was not required

on this project as it had not been needed or desired on our previous jointed concrete pavements. However, experience on this project indicates that a judicious use of vibration is very desirable on a continuous concrete pavement.

4. Although numerous failures have occurred on this project, there are many long sections (a mile or more in length) where no problems have been experienced. These sections have excellent riding qualities, and especially in comparison to the jointed concrete pavements to the south and north which are approximately the same age.

5. There is an excellent correlation of failures with concrete placement days experiencing high air temperatures.

6. Type III Cement (High Early Strength) will result in much higher steel stresses than Type I Cement (Normal) during the early life of the pavement.

7. The use of Type III Cement in combination with high air temperatures possibly resulted in some "flash sets" before complete consolidation of the concrete could be achieved, hence concrete honeycombing. This observation gives further support to the previous decision to eliminate the future use of Type III Cement on basis of its resulting high steel stresses.

8. Experience has shown the continuity of a continuous pavement that has a localized failure can be fully restored

with proper maintenance procedures.

9. Adequate concrete vibration by mechanical methods is an absolute neccessity, and this is especially true in the area of a construction joint at the beginning of a run.

10. For additional conclusions, see end of Appendix B.

REFERENCES

1. Report No. 1, "Background on Research Project in Walker County". September, 1960.

2. Report No. 3, "Description and Layout of Experiment Conducted In Walker County". October, 1960.

3. Report No. 6, "Crack Survey Data in Walker County". September, 1960.

4. Report No. 9, "Flexural Strength Study in Walker County". March, 1961.

5. Report No. 10, "Crack Survey Report in Walker County". March, 1961.

6. Shelby, M. D. and McCullough, B. F., "Experience in Texas with Continuously Reinforced Concrete Pavement." HRB Bulletin 274, 1-29 (1960)

7. Highway Research Board, <u>The AASHO Road Test</u>, (Report Number 5 Pavement Research). Washington, D. C.: National Academy of Sciences, 1962.

8. Report No. 14, "Pavement Structure Irregularities in Walker County". November, 1961.

9. Report No. 16, "Pavement Repairs in Walker County". February, 1962.

10. Report No. 17, "Field Trip to Walker County". March, 1963.

11. Report No. 18, "Pavement Repairs in Walker County". May, 1964.

APPENDIX A

Photographs of Failure Areas

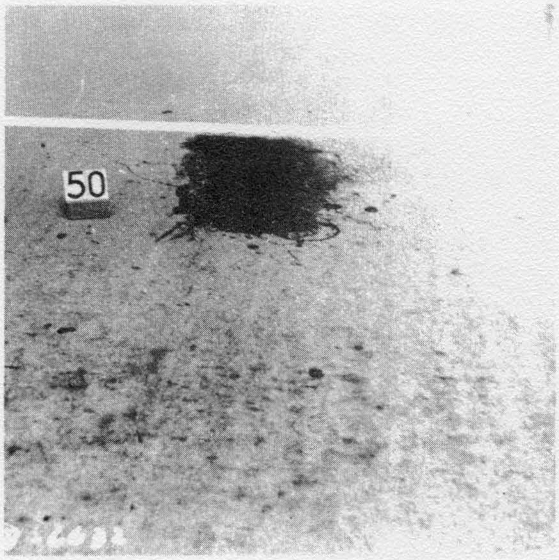


Figure A-1. Repair Area with Cold Mix. SBL Station 560+

Figure A-2. Failure at Construction Joint SBL Station 530+

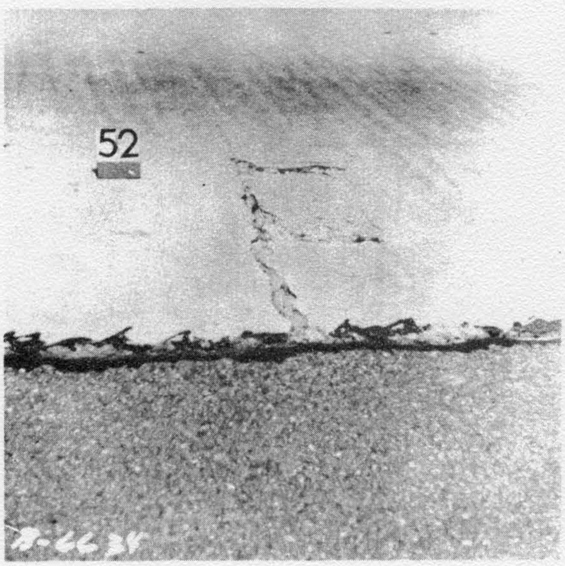


Figure A-3. Failure SBL Station 524+

Figure A-4. Failure Near Construction Joint. SBL Station 511+

Figure A-5. Repair Area. SBL Station 508+

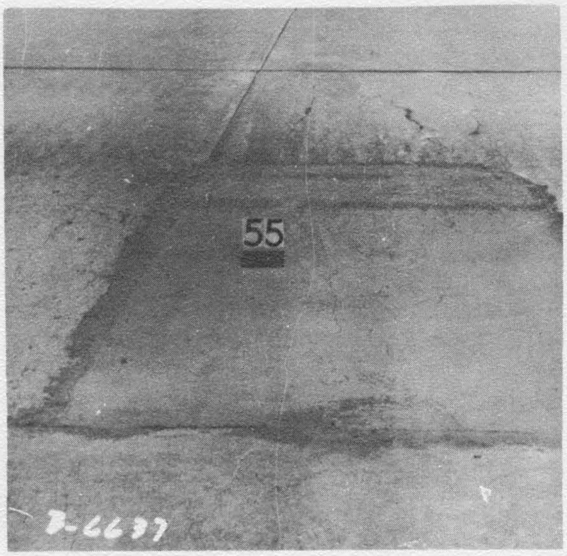


Figure A-6. Repair at Construction Joint. SBL Station 495+

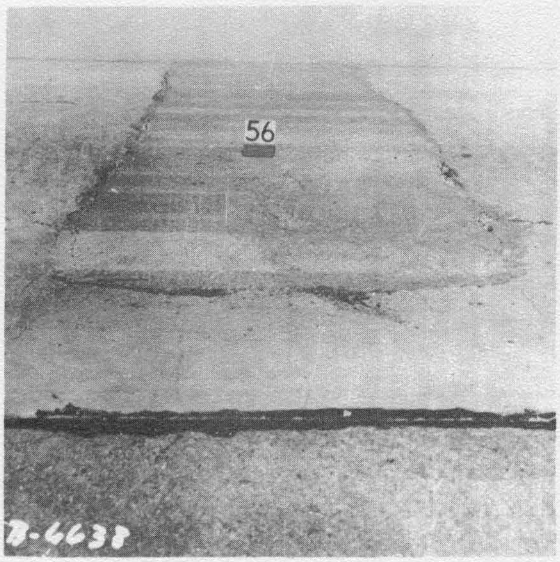


Figure A-7. Repair Area Deteriorating. SBL Station 485+

Figure A-8 Repair Area. SBL Station 445+

Figure A-9. Failure. SBL Station 441+

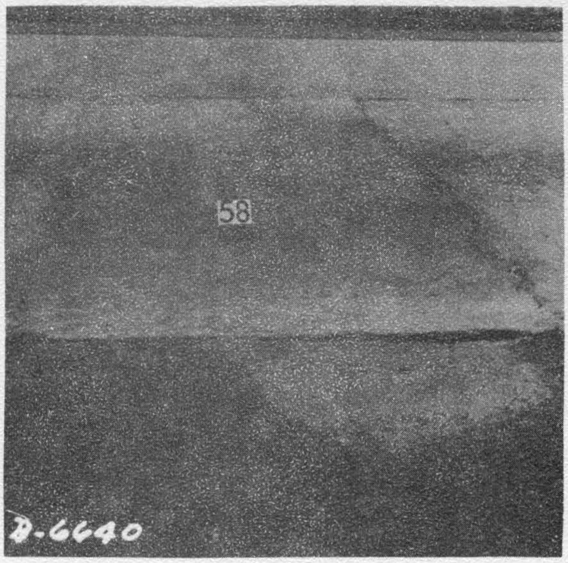


Figure A-10. Repair Area. SBL Station 441+

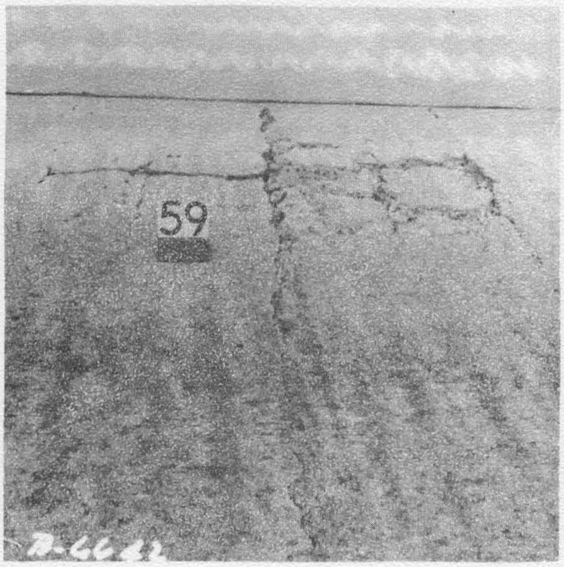


Figure A-11. Failure Near Longitudinal Joint. SBL Station 438+

Figure A-12. Repair Area. SBL Station 405+

Figure A-13. Repair Area. SBL Station 387+

Figure A-14. Repair Area. SBL Station 331+

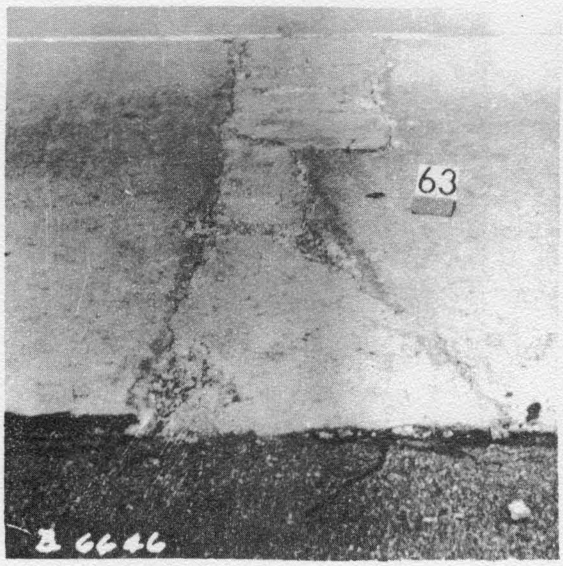


Figure A-15. Repair Area Deteriorating. SBL Station 206+

Figure A-16. Repair Area with Cold Mix Construction Joint SBL Station 98+

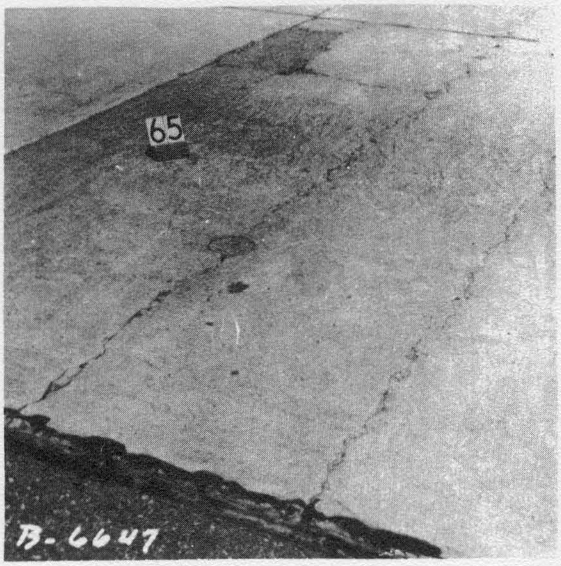


Figure A-17. Repair Area at Construction Joint. SBL Station 88+

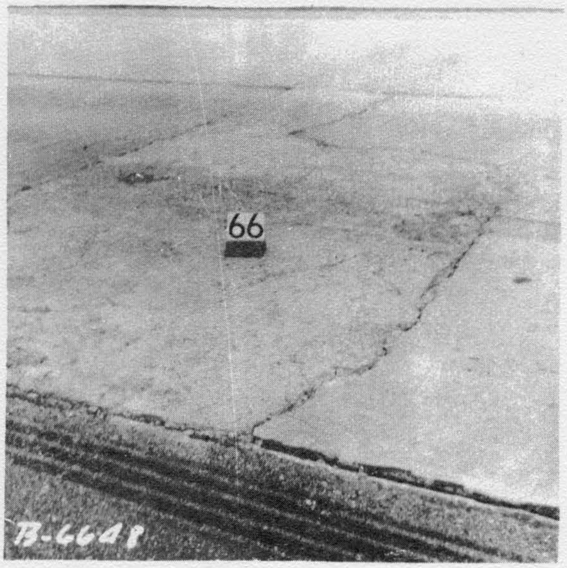


Figure A-18. Repair. NBL Station 138+

Figure A-19. Repair Area at Construction Joint. NBL Station 310+

Figure A-20. Failure. NBL Station 351+

Figure A-21. Repair Area with Cold Mix and Concrete. NBL Station 353+

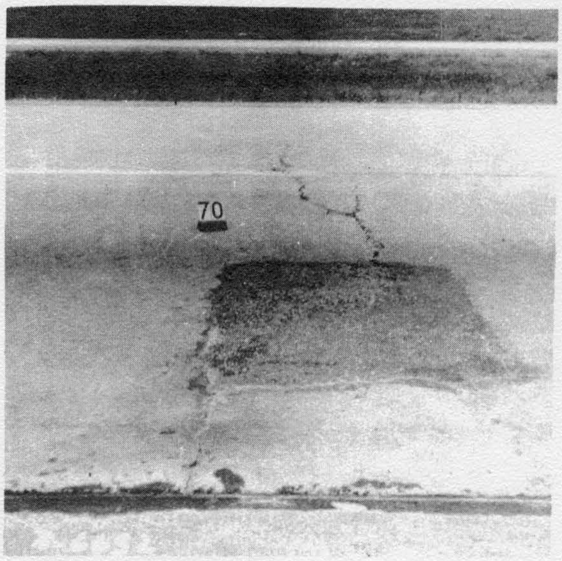


Figure A-22. Repair Area with Cold Mix and Failure. NBL Station 377+

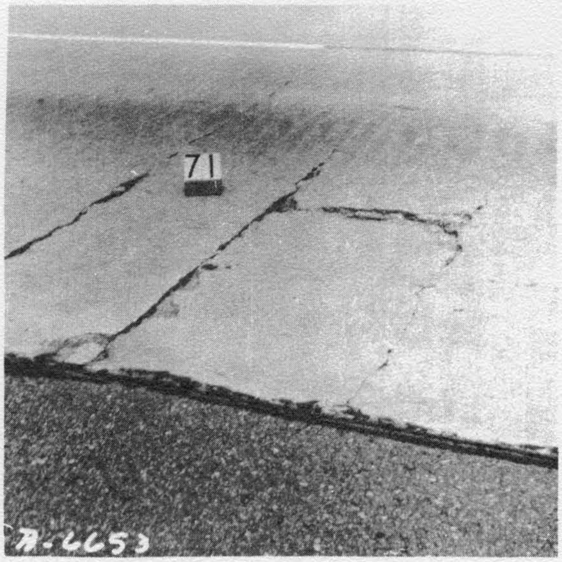


Figure A-23. Failure. NBL Station 419+

Figure A-24. Repair Area and Failure due to Honeycombing. NBL Station 425+

Figure A-25. Repair Area and Failure due to Honeycombing. NBL Station 425+

Figure A-26. Repair Area and Failure due to Honeycombing. NBL Station 520+

Figure A-27. Repair Area and Failure due to Honeycombing. NBL Station 521+

APPENDIX B

Pavement Repairs

Pavement Repairs In Walker County

March 13, 1962

The purpose of this report is to describe the observations made during the repairs of a failure in the experimental continuously reinforced concrete pavement on IH 45 South of Huntsville. The repairs were made adjacent to a construction joint in the northbound lane at Station 188. Figures B-1 and B-2 show the area prior to the beginning of repairs. A section 8 to 10 feet wide in the outside lane was removed on this date. An investigation of the slab removed and of the base materials revealed several interesting observations.

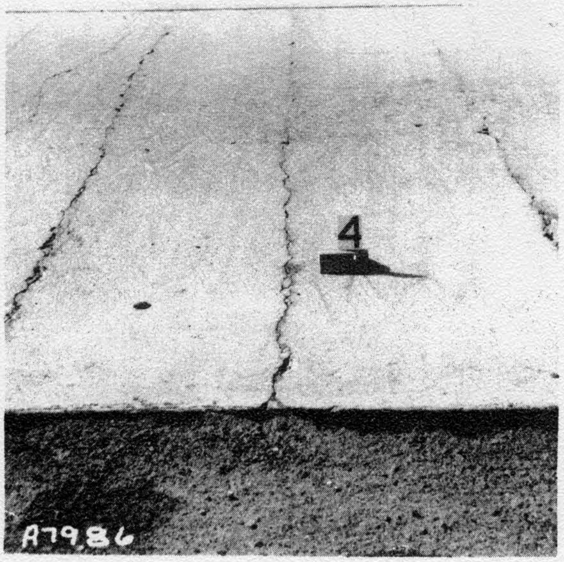


FIGURE B-1

FIGURE B-2

Area Removed for Repairs

Observations

Upon removing the upper four inches of concrete to expose the steel around the perimeter of the area to be removed, it was observed that a dry, sandy batch had been placed in the area immediately adjacent to the construction joint (Figure B-3). The material in the area of the dry batch crumbled under a light blow with a pick as shown in the picture. It was also noted that the transverse cracks in the failure area had formed directly above the transverse bars as shown in Figure B-4.

FIGURE B-3
Dry, Sandy Batch of Concrete Below Steel

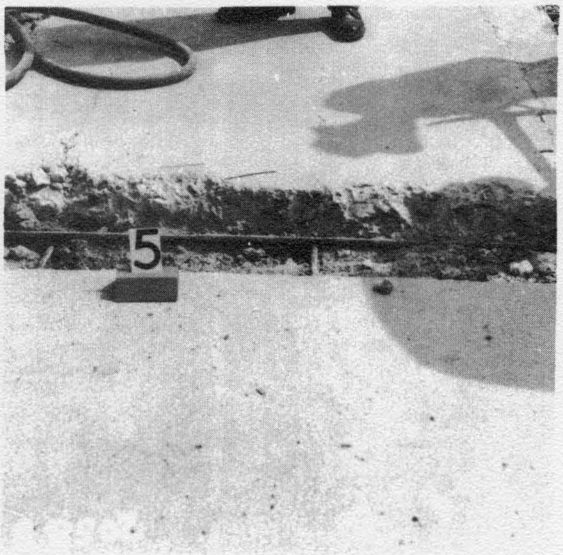


FIGURE B-4
Transverse cracks above Transverse Steel

After the slab had been lifted from the location, the subbase material was inspected. The upper 1 to 2 inches of the crushed sandstone base was wet, with some free water present on the surface as shown in Figure B-5. The lime stabilized subgrade appeared to be very dense although the upper portion was damp relative to the lower portion.

Figure B-6 shows the slab (upside down) after removal. It appears that some of the mortar from the fresh concrete combined with a thin layer of the crushed sandstone had adhered to the bottom of the slab. This figure also reflects the composition of the slab. The upper part of the slab (lower portion in figure) is sound while the bottom 3 to 4 inches of the slab is sandy and honeycombed.

FIGURE B-5
Base Immediately after Removing Slab

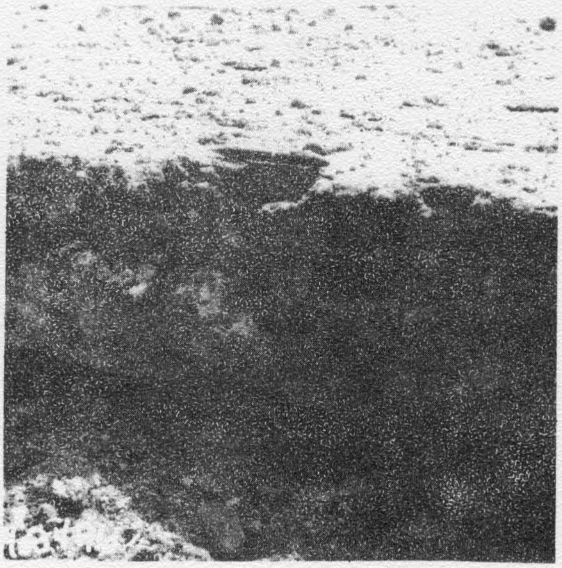


FIGURE B-6
Slab Removed from Pavement

Poor bond between the concrete and steel or complete absence in some cases resulted in wide crack openings as shown in Figure B-7. The steel removed in the vicinity of these open cracks also show rusty spots as a result of the water filtering through the crack.

FIGURE B-7
Crack in Bottom of Slab

Discussion

The honeycombing in the lower portion of the pavement in the failure area resulted in two slabs - a sound layer and a honeycombed layer - each approximately four inches thick. In essence, the upper part of the pavement structure consisted of a four inch pavement with the bar mat beneath it. Due to this condition of no bond of concrete to the steel, the longitudinal steel was not serving its design function of keeping the cracks tightly closed. Hence, the crack opened excessively resulting in complete loss of load transfer. This tended to compound the condition of inadequate slab thickness (lower four inches of "sandy" concrete). With these conditions present, the pavement quickly disintegrated when subjected to the heavy wheel loads experienced on this roadway.

The transverse bars acted as support for the thin slab of sound concrete and the longitudinal bars. The slab, unable to support the applied loads, failed above the supports resulting in the transverse deflection cracks shown in Figure B-4. The "banging" sound detected during previous investigations can be attributed to the thin slab hitting against the steel mat.

Inadequate vibration adjacent to the construction joint caused the honeycombing observed in the lower portion of the slab.

Several possibilities exist which might explain the dry batch of concrete. One theory is that the first batch mixed each day is lean as a result of cement adhering to the sides of the mixer. Other sources feel that the Type III Cement used on this project caused "sand balls", i.e., sand surrounded by cement, to form.

Conclusions and Recommendations

First, it should be pointed out that there was no evidence of inadequate pavement design. The failure must be attributed to the composition of the concrete in the vicinty of the construction joint.

It is believed that the honeycombing resulted from the inability of the paving equipment to properly consolidate the first 10 to 20 feet of the pavement. This deficiency can be overcome by hand vibration of the first 10 to 20 feet of placement. Furthermore, special attention should be given to consolidation of the concrete below the steel.

The special provision to Item 360 should eliminate the possibility of lean batches or sand "balling up" in the mix. However, it may be desirable to add cement to the first batch each day in order that the resultant mix contain sufficient cement after coating the mixer surfaces. Since this is a construction problem it is felt that this should be left to the discretion of the project engineer.

As a matter of future reference, it should be pointed out that there is a possibility that the slab removed from the pavement did not encompass all of the unsound concrete present in this area. This location should be observed for any unusual developments.