

# DEPARTMENTAL RESEARCH

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## FACTORS INFLUENCING THE DESIGN AND PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT



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# FACTORS INFLUENCING THE DESIGN AND PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

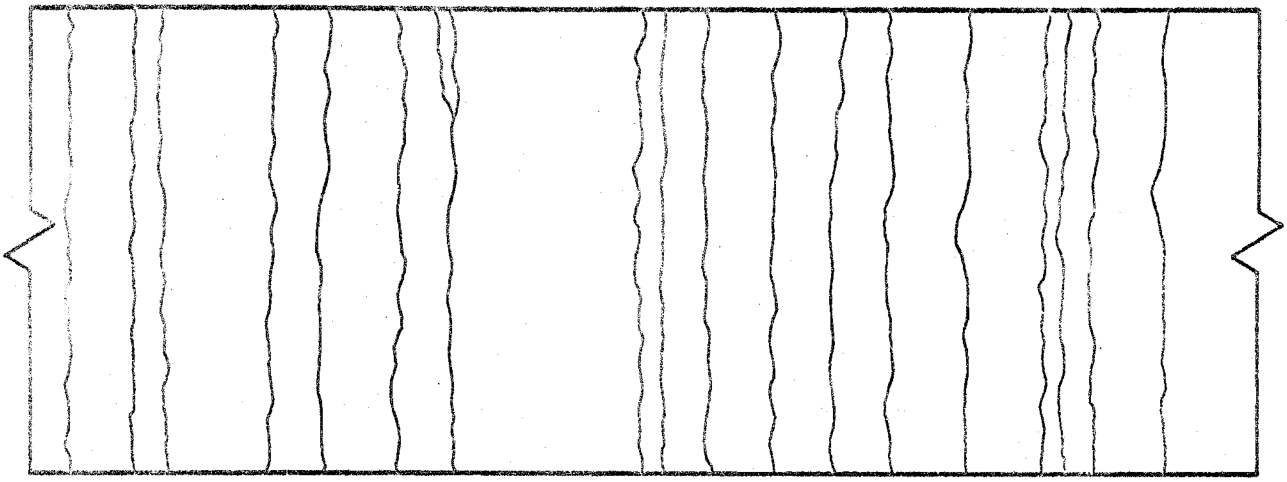
## I. INTRODUCTION

Continuously reinforced concrete pavement is a rather startling type of pavement to many highway engineers. Transverse contraction joints, long considered essential in the construction of concrete pavement to prevent concrete volume changes from causing excessive damage to the pavement, have been eliminated. In their place, so to speak, in continuously reinforced concrete pavement, is a series of seemingly uncontrolled, apparently randomized, cracks which generally run transversely across the pavement. These cracks are hairline in width and are generally spaced at closer intervals than common joint spacing. Figure 1 gives a diagrammetric view illustrating this difference in appearance between jointed and continuously reinforced concrete pavements.

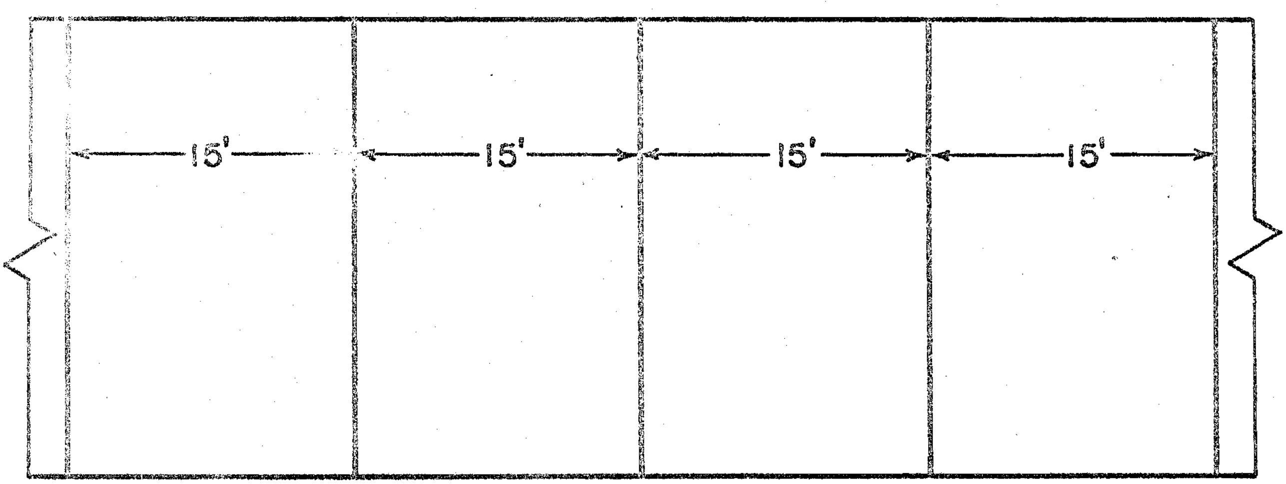
Figure 2 portrays a typical contraction joint in a jointed pavement used in Texas and elsewhere in the country. It has been sealed and is about 1/4 to 3/8 inch wide. Figure 3 shows a typical crack in continuously reinforced concrete pavement (note that a scale is laying across the crack). This is dramatic evidence of the hairline width of cracks in continuously reinforced concrete pavement. These cracks are quite a bit closer and tighter than volume control joints in a jointed pavement.

### History of Continuous Pavement

Figure 4 gives a list of some of the early projects using continuously reinforced concrete pavement throughout the country. The use of this



CONTINUOUSLY REINFORCED CONCRETE PAVEMENT WITH TYPICAL TRANSVERSE CRACK PATTERN




UNREINFORCED CONCRETE PAVEMENT WITH TRANSVERSE CONTRACTION JOINTS ON 15 FOOT CENTERS

FIGURE 1


FIGURE 2: VIEW OF A TYPICAL CONTRACTION JOINT IN JOINTED CONCRETE PAVEMENT

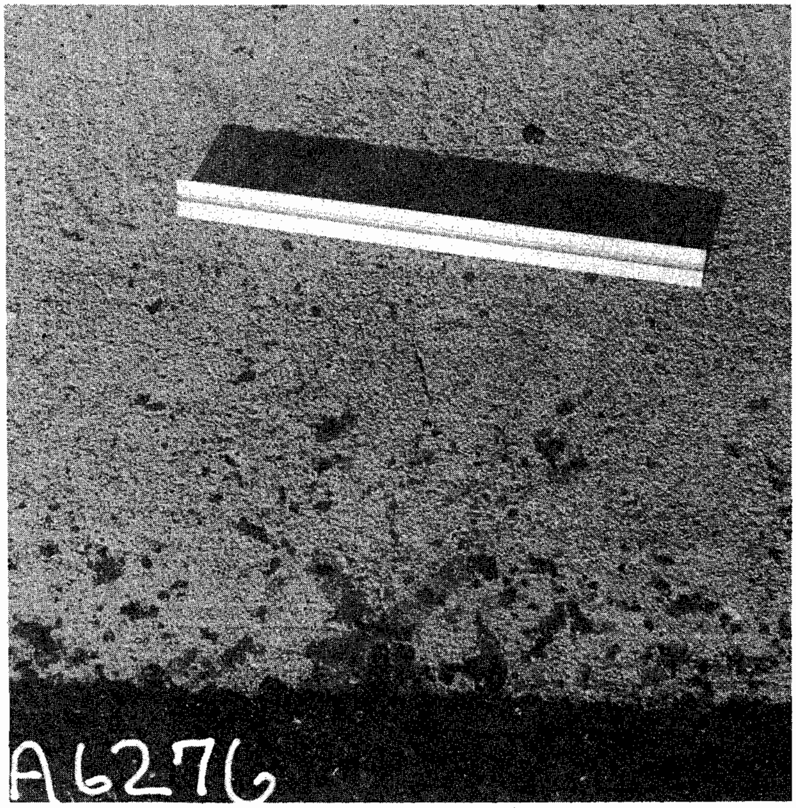



FIGURE 3: VIEW OF A TYPICAL TRANSVERSE CRACK IN CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

FIGURE 4

SUMMARY OF EARLY CONTINUOUSLY REINFORCED CONCRETE

PAVEMENT PROJECTS IN THE UNITED STATES

| YEAR BUILT | STATE | THICKNESS (Inches) | PERCENT LONGITUDINAL STEEL |
|---|---|---|---|
| 1938 | Indiana | 9-7-9 | 1.82 |
| 1947 | Illinois | 7 and 8 | 0.30 to 1.00 |
| 1947 | New Jersey | 8 and 10 | 0.90 and 0.72 |
| 1949 | California | 8 | 0.63 and 0.50 |
| 1951 | Texas | 8 | 0.70 |

type of pavement started back in 1938 with Indiana using a pavement thickness of 9-7-9[1] and 1.82 per cent longitudinal steel.[2] Note the percentage of longitudinal steel has ranged from as little as 0.30 per cent to as much as 1.82 per cent. The 0.70 per cent steel employed in the first Texas project was based largely upon the incomplete findings (at that time) from the Indiana and Illinois projects. Since that time, a theoretical analysis of the design of continuously reinforced concrete has been developed and will be discussed in general terms.

(1)Nine inches thick along each edge of the pavement and six inches thick in the center portion.

(2)Per cent of steel is a term referring to the area of steel in any given area of concrete times 100.

## II. TEXAS DESIGN APPROACH

### Determining Pavement Thickness

Any theoretical approach to this type of design results in a rather complicated problem. Most of the existing theories of concrete pavement thickness design are based upon the concrete being a homogeneous, isotropic, elastic solid in equilibrium over a subgrade whose reactions are vertical. Of course, it is evident these assumptions do not fit the actual conditions, but the unfortunate thing is that no umcomplicated rational design approach using more realistic assumptions has been developed, to the authors' knowledge, for the last 25 years. Therefore, engineers are in the position of having to use existing theories developed for thickness determinations of unreinforced jointed concrete pavement by Westergaard(3) and modified by Kelley(4) and others.

Now on continuously reinforced concrete pavement, these same theories for thickness design can be used, if it is first considered that the concrete and the steel are going to act separately. The purpose of the concrete, from an initial design standpoint, is to carry the wheel load, and the purpose of the steel is to keep the volume change cracks tightly closed so that there is 100 per cent load transfer across the cracks. If the cracks are kept tightly closed so that no water will seep through them to harm the subgrade then pumping and other detrimental

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(3) Westergaard, H.M., "Stresses in Concrete Pavements Computed by Theoretical Analysis", Public Roads, Vol. 7, No. 2, April 1926.
(4) Kelley, E. F., "Application of the Results of Research to the Structural Design of Concrete Pavements", Public Roads, Vol. 20, 1939.

effects are prevented.

Figure 5 shows a comparison of thickness design boundary conditions for jointed and continuous pavements. The wheel load is represented by a line load in the figure. The joint in jointed pavement requires the concrete to be designed for a corner loading or a modified corner loading depending upon the degree of load transfer across the joint. In continuous pavement, even without considering the steel as aiding in carrying the imposed load, it can be seen from Figure 5 that an interior condition exists as long as there is enough steel to keep a crack from opening up. Therefore, the slab from a structural standpoint, acts as a continuous uncracked slab. Using the interior loading design there is an approximate 20 per cent savings in concrete over the regular or heretofore considered standard type of pavement design.

Determining the Steel Percentage

Now as far as steel percentage is concerned in continuously reinforced concrete pavement, the steel is not used to prevent cracking because to effectively do so would be prohibitively expensive if not impossible to accomplish. Therefore, enough steel is used to prevent the cracks, which will form, from opening excessively, keeping them tightly closed so that there will be practically 100 per cent transfer of load across each crack.

The design of the steel is based on the basic relationship that was proposed several years ago by Mr. Vetter (1933), and published in ASCE Transactions.[5] The relationship is simply that the per cent of steel

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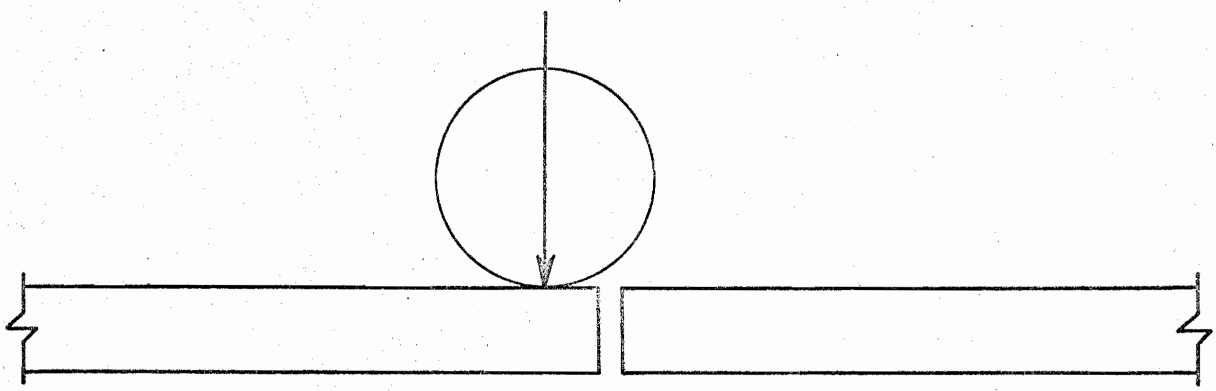
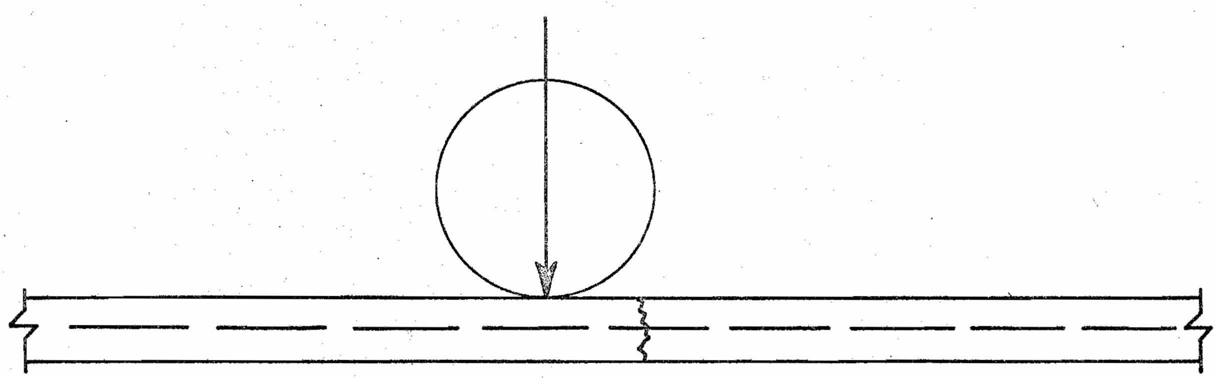
(5)Vetter, C.P., "Stresses in Reinforced Concrete Due to Volume Changes", Transactions, ASCE, Vol. 98, 1933.



JOINTED PAVEMENT
Designed for Corner Loading



CONTINUOUS PAVEMENT
Designed for Interior Loading (Approximately 20% Savings in Concrete)

COMPARISON OF THICKNESS DESIGN FOR JOINTED AND CONTINUOUS PAVEMENTS

FIGURE 5

required to control volume changes is equal to the tensile strength of the concrete divided by the tensile strength of the steel. This design approach has been reported elsewhere by the authors,[(6)] and therefore will not be discussed in detail in this paper. It might be appropriate to point out an important concept while considering this basic relationship of tensile concrete strength to tensile steel strength. The stronger the concrete, the more steel required, because the steel will continue to increase its stress as the concrete increases its strength. This can best be illustrated by assuming a temperature drop where the concrete is attempting to contract. This restrained contraction throws the steel into tension at the cracks, building up tensile stresses in the concrete until the tensile strength of the concrete is reached. At this point, instead of more stress going to the steel at an existing crack, the concrete cracks again, thereby reducing the stress in the steel at an existing crack. Therefore, the lower the tensile strength of the concrete, the lower the resulting stress in the steel at the cracks. With this concept, quite a bit less steel than heretofore considered as necessary would be needed, because only enough steel is needed to insure that the concrete tensile strength is reached before the steel stress at the cracks reaches yield.

Optimum CPCR Design

Figure 6 is a design comparison based on the above discussed theories, using different types of steel and design loads for various flexural

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(6) McCullough, B. F. and Ledbetter, W. B., "LTS Design of Continuously Reinforced Concrete Pavement", *Journal of the Highway Division*, Proceedings, ASCE, Vol. 86, HW4, December 1960.

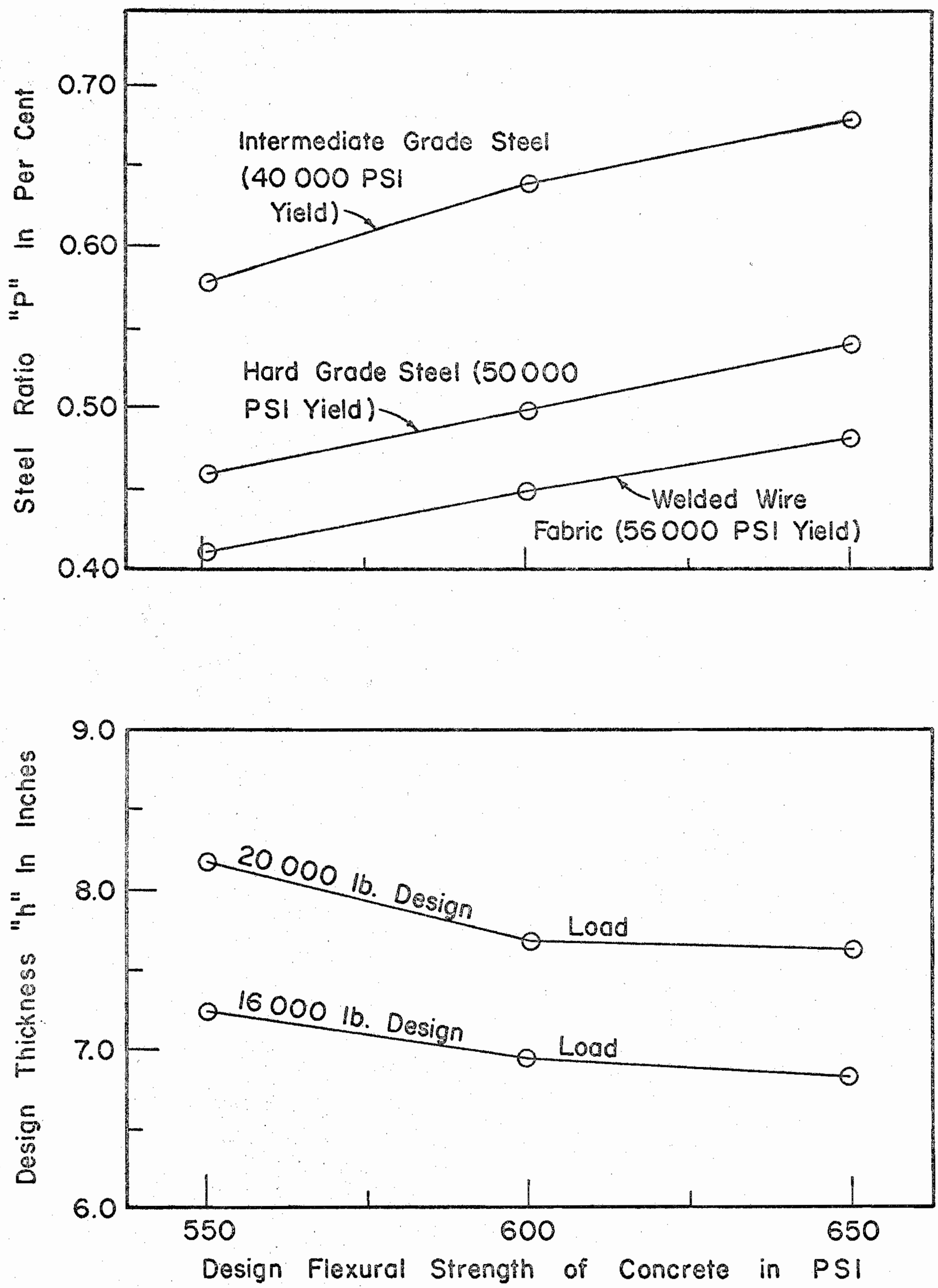


FIGURE 6 - DESIGN COMPARISONS USING DIFFERENT TYPES OF STEEL AND DESIGN LOADS FOR VARIOUS FLEXURAL STRENGTHS

strengths. On the lower part of the figure there are three design strengths portrayed - 550, 600 and 650 psi. This figure is based on the assumption that the tensile strength of concrete is, roughly, directly proportional to the flexural strength. It can be seen from the figure that a reduction in concrete strength does not have too great of an effect on the thickness required (note that a reduction in flexural strength from 650 psi to 550 psi only results in a 1/4 inch thicker pavement). On the upper portion of the figure the effect of a change in flexural strength on the per cent of steel can be seen. A flexural strength of 650 psi would require about 0.55 per cent steel, whereas if the flexural strength is lowered 50 psi, only 0.5 per cent steel is needed, representing about a 10 per cent savings in steel. With the cost of steel being a sizable portion of the total cost of continuously reinforced concrete pavement, by lowering concrete flexural strength less steel is required thereby reducing the overall cost of the pavement. Conversely, if the amount of steel is determined for a particular flexural strength, and a higher concrete strength is obtained in the field, the steel will be stressed beyond yield before a new crack is developed. The existing cracks will then open a detrimental amount, resulting in a reduction in the load transfer capability which in turn will cause a failure in the pavement due to traffic load.

Figure 7 is a comparison of pavement types and costs. The three types of concrete pavement in use today - the jointed reinforced, the jointed unreinforced, and the continuously reinforced - are shown in the figure. The first two types are designed for a corner load condition

FIGURE 7

COMPARISON OF CONCRETE PAVEMENT TYPES

| ITEM | JOINTED Unreinforced | JOINTED Reinforced | CONTINUOUS |
|---|---|---|---|
| Thickness | 10" | 10" | 8" |
| Steel Percentage | None | 0.1% to 0.2% | 0.5% to 0.6% |
| Contraction Joint Spacing | 15' | 30' to 120' | None |
| Cost per Square Yard | 3.40 to 4.60 | 4.70 to 5.30 | 4.30 to 5.70 |

and, based on one set of boundary conditions, a thickness of 10 inches would be required. The required thickness of continuously reinforced concrete under the same boundary conditions would only be 8 inches. The steel percentage for the unreinforced would be 0 per cent steel whereas the jointed reinforced would range from 0.1 to 0.2 per cent and the continuously reinforced would run from 0.5 to 0.6 per cent.

On the bottom of the figure are some average costs per square yard on some recent projects in Texas. Note that in cost per square yard of the various types of concrete pavement, there is a considerable overlap. The unit prices for the earlier continuously reinforced concrete pavement projects ran fairly high - in the neighborhood of $5.70 per square yard. They are now in the range of $4.30 to $4.50 per square yard in Texas. This price places continuously reinforced concrete pavement in the same range as jointed reinforced concrete pavement (on a square yard basis). The authors are of the opinion that, for high volume, heavy traffic highways, continuously reinforced concrete pavement is ideal in that its initial cost is not too much greater than other types, and from a maintenance and traffic service standpoint, it is unexcelled in over ten years of service.

## III. FACTORS INFLUENCING THE DESIGN AND PERFORMANCE OF CPCR

Since the initial construction of the continuously reinforced concrete pavement in Fort Worth, numerous observations and studies have been made of these pavements.[7] In essence most investigations have been an attempt to correlate theory with practice in an effort to accurately evaluate the performance of this type of pavement. Following is a discussion of several factors which influence the design and performance of continuously reinforced concrete pavement. Some of these factors can be explained with existing theories and some bring up concepts heretofore not considered, while others involve construction control.

### Crack Width

As mentioned previously, the amount of steel needed is determined by the crack width desired, since the concept of design is to keep the cracks as tight as possible. Figure 8 shows the relation between crack width and average crack spacing for a typical project. It is evident from this figure that there is direct relationship between crack width and crack spacing. In other words, the further apart the cracks, the greater the width of the cracks for a given set of conditions. Carrying this observation further, the greater the width of the crack the more shear stresses developed in the steel from a wheel load and the greater the infiltration of foreign matter, both of which are prospective factors for developing future trouble.

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(7)Shelby, M.D. and McCullough, B.F., "Experience in Texas with Continuously Reinforced Concrete Pavement", Highway Research Board Bulletin 274, 1960.

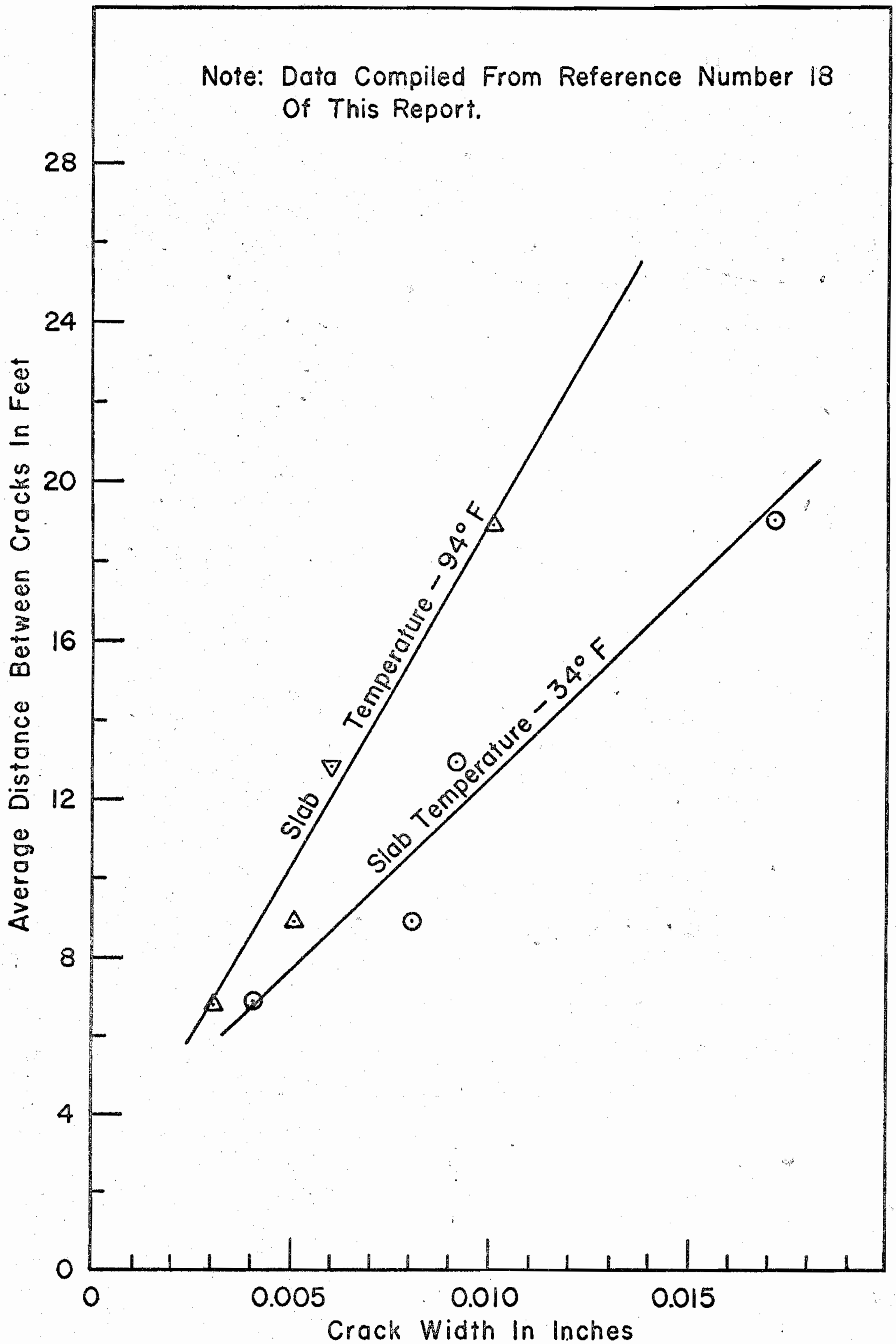


FIGURE 8 – RELATIONSHIP BETWEEN CRACK WIDTH AND CRACK SPACING FOR TWO SLAB TEMPERATURES

## Per Cent Steel

Figure 9 portrays the relationship between the steel per cent, or steel ratio and the average crack spacing. Note that the factors are indirectly related - as the per cent steel increases the crack spacing decreases. Keeping in mind that crack spacing is directly related to crack width, it is apparent that the higher the steel percentage the better the design. However, the smallest steel percentage which will obtain satisfactory results is to be desired from an economical standpoint. From Figure 9 it may be tentatively concluded that a steel percentage of around 0.4 to 0.3 per cent is about as low a percentage as can be used and still keep the distance between cracks small enough for a minimum crack width. Conversely, it may also be concluded that any steel percentage over 1.0 per cent does not materially alter the average crack spacing and hence does not aid the performance of the pavement. From this figure it is apparent that from an experience standpoint, the optimum steel percentage is in the range of 0.3 per cent to 1.0 per cent. The theory presented earlier herein gives the optimum to be from 0.5 per cent to 0.6 per cent for concrete pavements generally used in Texas.

## Bond Area

Figure 10 shows the relationship between bond area and average crack spacing. Note that bond area is expressed in terms of bond area per volume of concrete. As was the case for per cent steel, these two factors are inversely related in a linear fashion. This indicates a smaller crack spacing could be obtained by using smaller bars at a smaller spacing in lieu of the more expensive alternative of increasing the steel percentage.

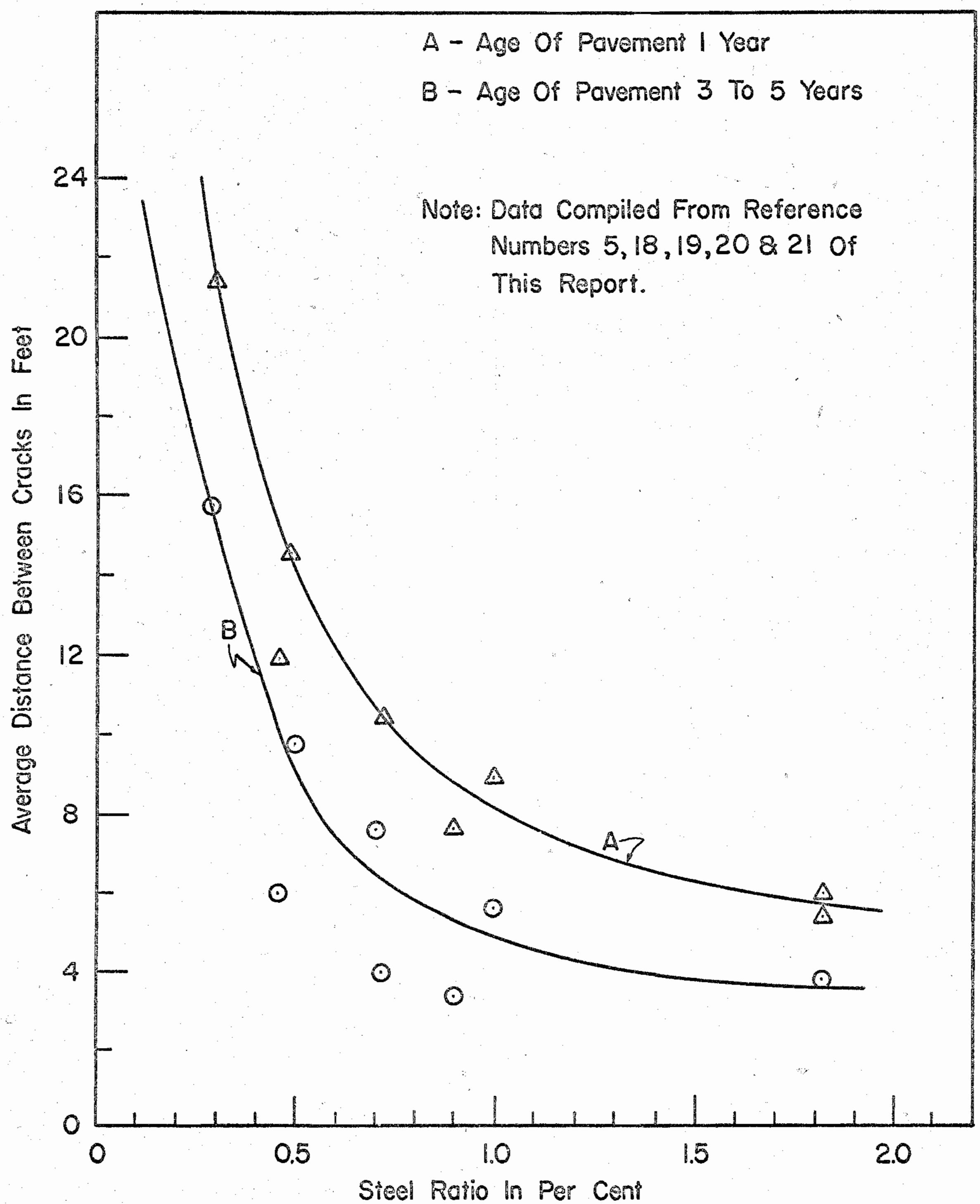


FIGURE 9 - RELATIONSHIP BETWEEN STEEL RATIO AND CRACK SPACING

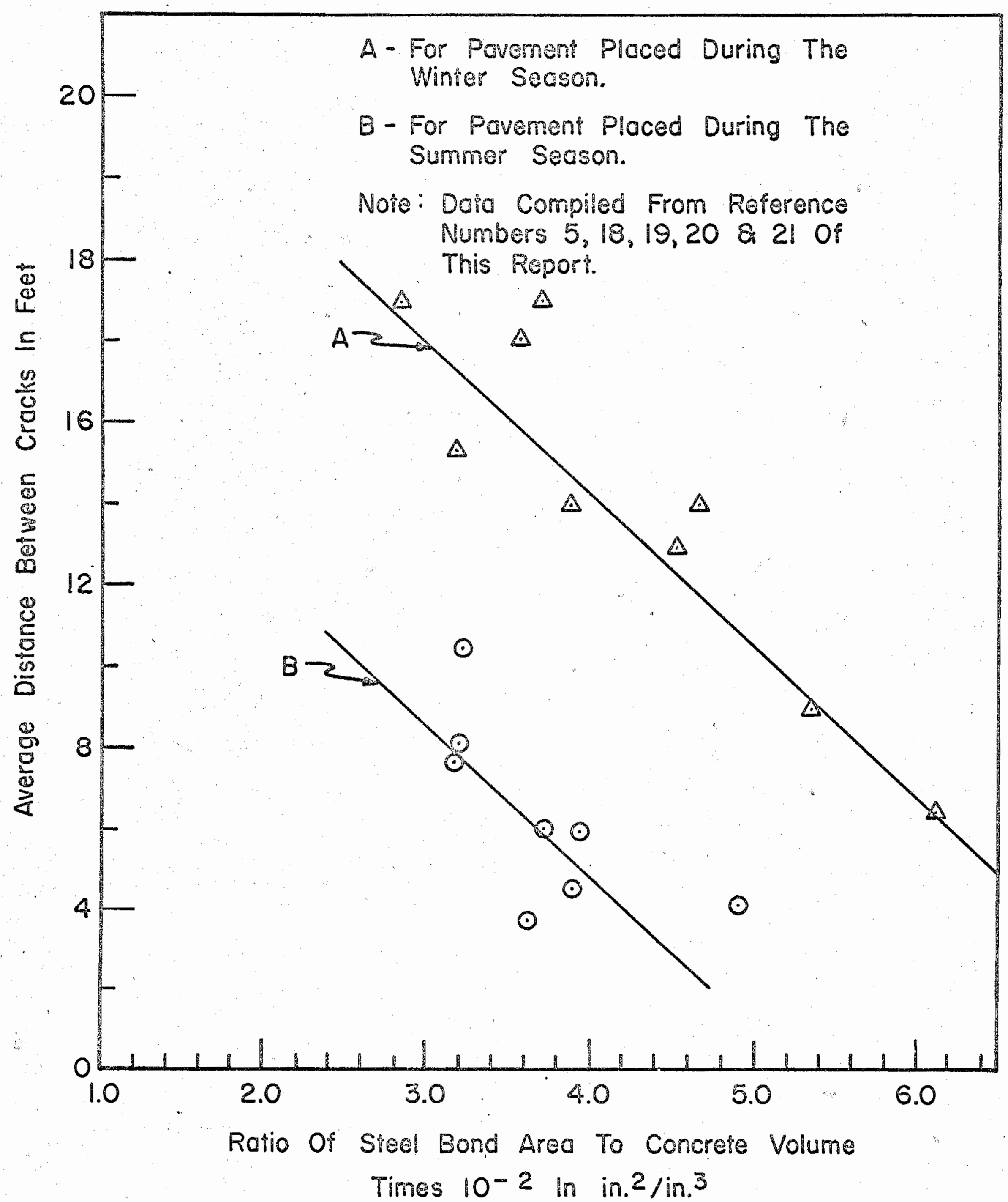


FIGURE 10-RELATIONSHIP BETWEEN BOND AREA PER CONCRETE VOLUME AND CRACK SPACING

Although theory does not cover this feature, it is a logical deduction. Until theories are developed experience alone must be relied upon to supply the needed information.

## Season of Placement

Figure 10 also brings out another interesting point that influences performance - the season the concrete is placed. From observations, it can be seen that this is a very important factor in determining the crack spacing which in turn determines the stress on the steel. The upper line (Line A) is for pavements placed in the winter season, whereas, the bottom (Line B) is for pavements placed in the summer season. Pavements placed during the hot weather actually had a closer crack spacing and consequently a smaller crack width than those placed in cool weather. This observation can be explained by relying on theory, which shows that temperature drop is the primary condition producing cracks. If the pavement is placed in $100^{\circ}$ weather and the temperature drops to $30^{\circ}$, $70^{\circ}$ of restrained volume stresses are built up in the concrete. Consequently, this condition results in a closer crack spacing than if the concrete was placed in $50^{\circ}$ weather and the temperature drops to $30^{\circ}$ where only $20^{\circ}$ of restrained volume stresses are built up in the concrete.

## Crack Strength

Figure 11 shows a relationship between concrete strength and the average crack spacing. Since flexural strength is generally used in Texas as the job control test, the crack spacing is expressed here in terms of flexural strength, (although a much better correlation could probably be obtained with tensile strength). Note that as the concrete

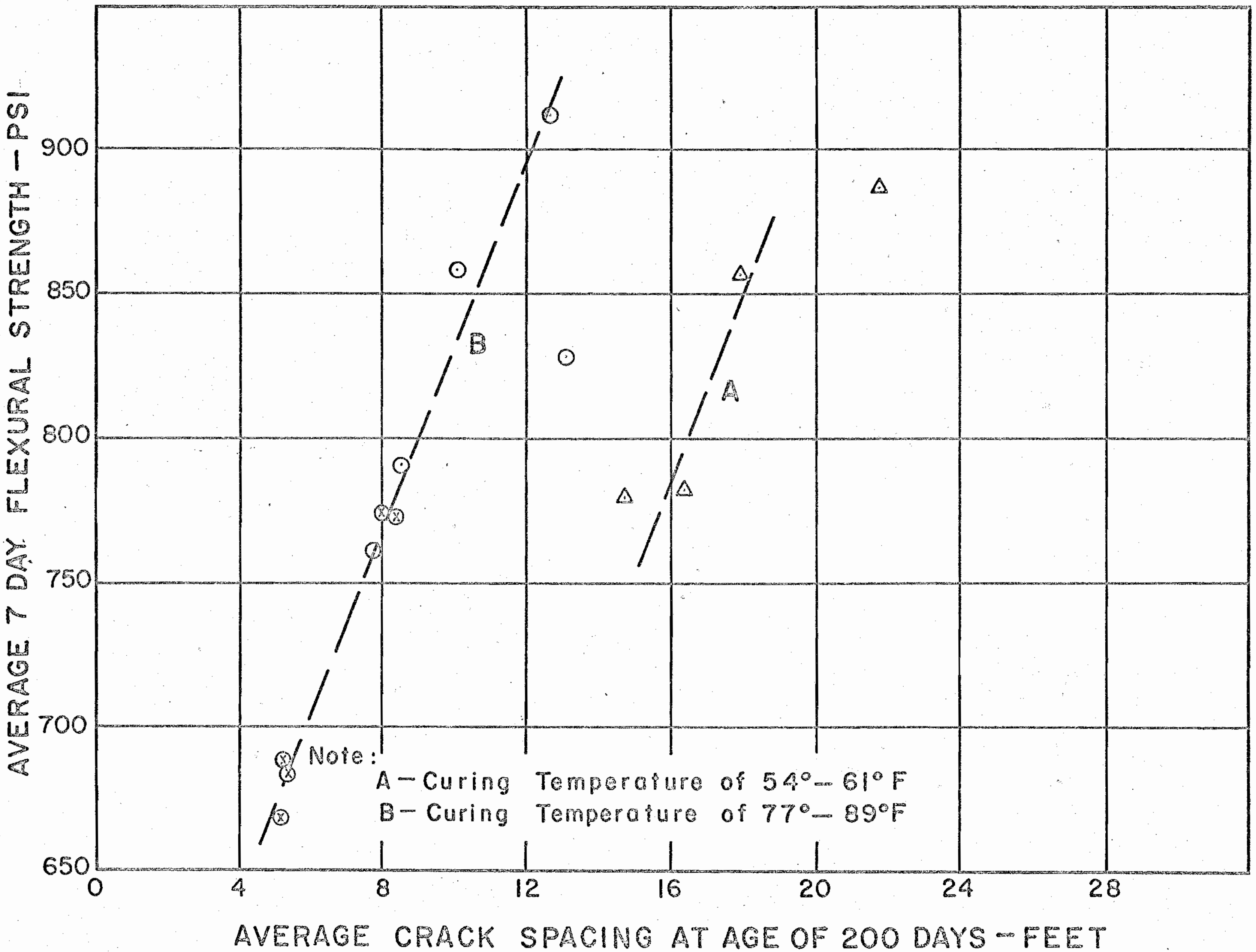
AVERAGE 7 DAY FLEXURAL STRENGTH — PSI
900
850
800
750
700
650
0
4
8
12
16
20
24
28
AVERAGE CRACK SPACING AT AGE OF 200 DAYS — FEET
B
A
Note:
A — Curing Temperature of 54°— 61° F
B — Curing Temperature of 77°— 89°F

FIGURE 11

strength increases the average crack spacing also increases. This observation is in agreement with theory, but is contrary to the popular notion "the stronger the slab the better it would be". This concept of the effect of concrete strength is importatnt when considering that a pavement with a flexural strength of 900 psi would have a crack spacing of twelve feet, whereas, as reduction to 650 psi would result in a five foot crack spacing. All the benefits derived from increased steel percentage and bond area could conceivably be cancelled out by a relatively high concrete strength.

Bar Lap

Another factor influencing the performance of continuously reinforced concrete pavement is the longitudinal bar lap. The ACI code and most of the other codes specify a 20 bar diameter lap. This is suitable when dealing with stresses in the steel to only one-half the steel's yield point (a safety factor of 2), such as, a working stress of 20,000 psi stress on a 40,000 psi yield steel. However, with continuously reinforced concrete pavement, the steel stresses are designed to approach the yield point (very low safety factors) and consequently a 20 bar diameter lap has been found to be insufficient.

On some early continuous pavements a 20 bar diameter was specified and considerable trouble was experienced. Figure 12 is a picture of a bar lap failure that resulted from insufficient lap at a construction joint. The older slab is to the left and the younger slab to the right. The slabs were connected by 20 bar diameter laps which were all located along a transverse line. Due to the high stresses imposed, a bar slip

FIGURE 12: VIEW OF A BAR LAP FAILURE AT THE CONSTRUCTION JOINT IN CONTINUOUSLY REINFORCED CONCRETE PAVEMENT

occurred along these laps. Then cracks formed over each one of the longitudinal bars with a resulting corner loading condition instead of the interior loading condition as designed for. The corner loading condition would require a 10 inch slab instead of an eight inch slab, therefore, the slab was overstressed due to wheel load, and required maintnenace repair to correct the situation.

Rather than resort completely to an increase in bar lap to cure this problem a lap staggering requirement was specified. Figure 13 portrays the staggered lap technique used on all continuous projects today. Specifications now require that in any two feet of pavement length only 1/3 of the bars can be lapped and, in addition, an increased bar lap of 25 diameters is required. At the present time, all projects using this specification have performed satisfactorily.


FIGURE 13: VIEW OF THE REINFORCING STEEL FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENT SHOWING THE STAGGERED BAR LAP

## IV. CONCLUSIONS AND RECOMMENDATIONS

Continuously reinforced concrete pavement is receiving widespread attention from highway engineers throughout the country. The ideas proposed through its use are considered by some to be radical, but its performance record in Texas, to date, has been outstanding. Texas' experience with continuous pavement dates back 10 years as of this writing, and concerning the design and performance of this type of pavement the following recommendations and conclusions are suggested.

1. Proper design requires strict control of concrete construction and a lower strength concrete than heretofore considered to be adequate.

2. The variables influencing the design and performance of this type of pavement are many, including such things as temperature extremes expected, season of placement, steel bond area per volume of concrete, per cent steel, concrete strength, steel bar lap, and many others not mentioned in this report.

3. The cost of this type of pavement is becoming less as more is being built, and today it is not much more expensive than jointed reinforced or jointed unreinforced concrete pavement on a square yard basis.

4. The high level of performance of this type of pavement indicates it to be a desirable type of pavement on high volume, heavy traffic highways where an interruption for even the most routine type of maintenance would be dangerous and aggrevating to the traveling public.

5. A more widespread use of this type of pavement is predicted in the future.

6. The questions raised upon the validity of established concepts

through the use of continuously reinforced concrete pavement are healthy, emphasizing the fact that no established concept is necessarily entirely correct in every instance. This points out that engineers should always explore every avenue of possibility when approaching a problem.