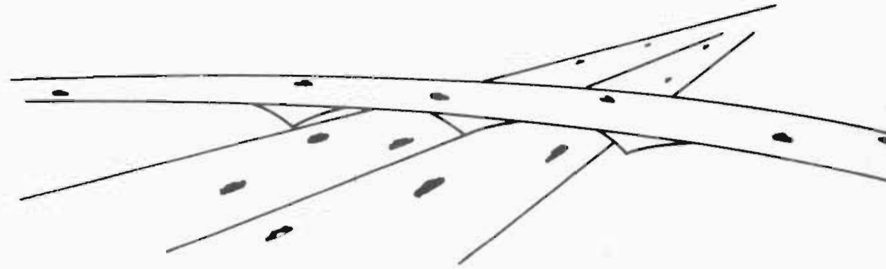

DEPARTMENTAL RESEARCH

Report Number 46-8(F)

PERFORMANCE of CONTINUOUSLY REINFORCED CONCRETE PAVEMENT in TEXAS

TEXAS HIGHWAY DEPARTMENT

PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE

PAVEMENT IN TEXAS

by

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Research Report No. 46-8 (F)
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Performance Study of
Continuously Reinforced Concrete Pavement

Conducted by

Texas Highway Department
Highway Design Division, Research Section

In Cooperation With The
U. S. Department of Transportation
Federal Highway Administration
Bureau of Public Roads

August 1968

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ABSTRACT

This report is a summary of the performance of continuously reinforced concrete pavement in Texas. The performance of pavement test sections with varying subbase, subgrade, and slab thickness characteristics is evaluated in terms of steel strain, deflection, crack pattern, pumping and traffic.

Data collection and analysis techniques, covered in previous reports were essentially the same as used at the AASHO Road Test with appropriate modifications for the continuously reinforced concrete pavement. Empirical equations developed by using regression techniques are modified for use as design tools.

Results indicate that each of the parameters mentioned above affect the performance of continuously reinforced concrete pavement in some way. Pavement type, pavement thickness, subbase type and the subgrade were all found to affect deflection. Pavements with 0.5 percent longitudinal steel are performing satisfactorily as true continuous pavements, i.e. good load transfer is being obtained.

The crack pattern development is related to pavement age, subbase friction, concrete flexural strength and curing temperature. Crack spacing distribution in a given pavement length indicates performance. A normal distribution reflects satisfactory performance while a skewed distribution reflects unsatisfactory performance.

Based on percentage evaluation, twice as many jointed pavements as continuous pavements were found pumping. If not protected, lime-stabilized subbases may pump. Asphaltic concretes with high asphalt content or good surface treatments will protect stabilized subbases. Signs of pumping are not always proof that the subbase is being erroded.

The Present Serviceability Indexes of CRC pavements in Texas follow the trend of the AASHO equations but with a somewhat lower initial PSI. CRC pavements are performing with a significantly higher PSI than are jointed concrete pavements with the same number of equivalent 18 kip axle applications.

NOMENCLATURE

Symbol	
S	Stress in Longitudinal Steel, plus sign denotes tension
E_c	Compressive modulus of elasticity of concrete
α_c	Coefficient of thermal expansion for concrete
ΔT	Change in slab temperature from time in question to time of concrete placement
$\bar{X}$	Average crack spacing, ft.
P	Per cent longitudinal steel
Z	Shrinkage, in/in.
ΔZ	Change in shrinkage, in/in.
ΔX	Crack width in concrete, in.
K_1	Constant equal to 1.468
K_2	Constant equal to 2.553
D_c	Edge deflection at a crack in CRCP, in.
D	Slab thickness, in.
T	Slab temperature differential, degrees F
T_{sg}	Texas triaxial class of subgrade
L	18 Kip single axle load
SS	Soil support

NOMENCLATURE

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Symbol

U_1	Seven day unconfined compressive strength of subbase, psi
U_2	Seven day unconfined compressive strength of lime stabilized subgrade, psi

PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT IN TEXAS

I. INTRODUCTION

Background

Continuously reinforced concrete pavement became a reality in Texas in 1951 when the first of its kind was built. Several other states had used continuously reinforced pavement on a small scale at that time. The pavement was experimental in the sense that it was new to the Texas Highway Department. As the Interstate Highway System got underway, more continuously reinforced pavements were built.

By 1960, research studies were underway on the continuously reinforced concrete pavement. In 1963 a need for the evaluation of the performance of continuously reinforced concrete pavement (CRCP) was realized. At that time the Texas Highway Department initiated a research project in cooperation with the Bureau of Public Roads. In very general terms, the objective of this research effort was to determine the performance characteristics of continuously reinforced pavement under varying conditions of subbase support, subgrade and pavement thickness. This report presents a performance record of CRC pavement in Texas and is the last of a series of reports.

Reports

In addition to this report seven others have been published. A variety of subjects have been covered, but all were intended to cover some portion of the objective.

Report 46-1 entitled, "Development of Equipment and Techniques for a Statewide Rigid Pavement Deflection Study", gave a detailed account of equipment and procedures as developed for the large research study.[1] The method for obtaining temperature differential for concrete pavement by use of a portable slab was covered from both the theoretical and practical standpoint. The development of the basin bean for concrete pavement was presented as well as the development of a formula for radius of curvature. The design of the truck which was used to apply static load to the pavement in deflection studies was presented. Testing procedures for deflection studies on a statewide basis were outlined.

Report 46-2 entitled, "Analysis of Steel Stress and Concrete Movement on an Experimental Continuously Reinforced Concrete Pavement", was a report on an experimental pavement built in 1964.[2] The experimental pavement had varying percentages of longitudinal steel and also varying preformed crack spacings. Data obtained included strains in the longitudinal steel, concrete movement, crack pattern with time, and a recording of the slab temperature. These data were analyzed with theoretical concepts of steel stress and concrete crack width in mind. The analyses resulted in empirical equations for stress in the steel and crack width in the concrete. The parameters exhibited in these equations were coefficient of thermal expansion of the concrete, temperature change in the slab, modulus of elasticity of the concrete, crack spacing in the concrete, percentage of longitudinal steel in slab, and the shrinkage of the concrete.

Report 46-3 entitled, "Evaluation of Single Axle Load Response on an Experimental Continuously Reinforced Concrete Pavement", was a deflection study on the pavement studied in Report 46-2 and another experimental CRCP.[2,3] In this report the response of the pavement was measured by deflection and radius of curvature and compared to percent steel, load, concrete modulus of elasticity and crack spacing.

Report 46-4 entitled, "Determining the Relationship of Variables in Deflection of Continuously Reinforced Concrete Pavement", was a detailed analysis of deflection in terms of slab temperature differential, crack spacing, crack width and soil support.[4] The analysis presented in the report was based on data taken from three different pavements. These data were recorded around-the-clock, i.e. for periods of 24 hours to get all effects of temperature. The analysis was summed up by the development of an empirical equation based on the parameters enumerated above.

Report 46-5 entitled, "A Statewide Deflection Study of Continuously Reinforced Concrete Pavement in Texas", was the climax to the performance study of CRCP in terms of load-deflection studies.[5] Deflection data were gathered from approximately 45 different pavements located throughout the state. Parameters included in the experiment factorial included modulus of elasticity of the concrete, flexural strength of the concrete, subbase type and subgrade support. The parameters that significantly affected deflection and radius of curvature were correlated into empirical equations similar to that in Report 46-4. This report also validates some of the assumptions on which the design of continuously reinforced concrete pavement is based.

Report 46-6 entitled, "A Laboratory Study of the Variables That Affect Pavement Deflection", was a report on a laboratory study aimed at studying the relationship of deflection and modulus of elasticity.[6] The parameters considered in the experiment were load, plate or slab thickness, support and modulus of elasticity of plates or slabs. The results found were similar to those from certain field studies where the relationship of deflection to modulus of elasticity under certain conditions was not always consistent with accepted theory.

Report 46-7 entitled, "Observation and Analysis of Continuously Reinforced Concrete Pavement", was a report which compared data collected, analyzed and reported on this project with theoretical methods of slab analysis now being developed at The University of Texas.[7,8]

II. PERFORMANCE

The performance of a pavement is a measure of its accumulated service or the adequacy with which it serves its purpose. Pavement performance in its most general sense is usually measured or specified with an index value as suggested by Carey and Irick.[9,10]

This research effort was entitled "Performance Study of Continuously Reinforced Concrete Pavement". The pavement performance was evaluated on test sections selected in the experiment factorial. Performance of pavements of varying designs in the experiment factorial were studied for their relative performance and finally their performance in terms of the present serviceability index or PSI.

Relative performance was measured or indicated by steel strain, deflection, crack pattern, pumping, and traffic.

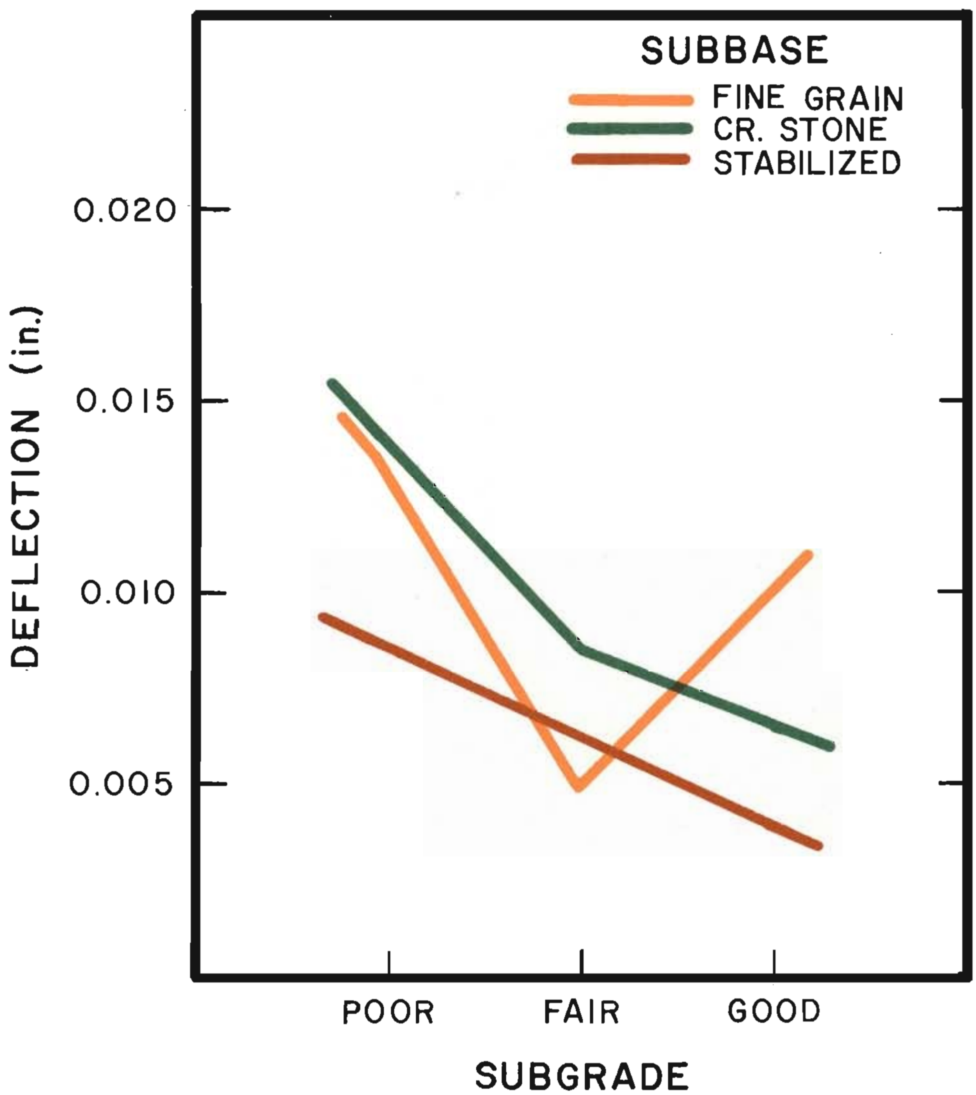
Steel Strain

Based on a very limited amount of steel strain data collected in the investigation, no significant relationship was found between performance and the amount of longitudinal reinforcing steel in pavements of similar design.[2] Investigations prior to this one indicated extremely high stresses in the longitudinal steel when Type III cement is used.[11] Specifications now also limit the fineness of cement.

Deflection

Pavement deflection or response to load indicated relative performance of pavements of different design. Different subgrades were evaluated in terms of deflection.[5] In Figure 2.1 the relative performance of poor, fair and good subgrades is shown in terms of deflection. Also indicated in Figure 2.1 is the relative performance of the three subbases shown. The low deflection of the fine grain subbase on the fair subgrade was a case where the subgrade was stabilized with lime. Lime stabilization of the subgrade was not part of the experiment factorial.[5] Stabilization of the subgrade with lime is a widely used technique for developing a construction platform in areas of the state where subgrades are very wet.

Because of results with lime stabilized subgrades as mentioned relative to Figure 2.1 it is believed that the lime treated subgrade actually performs like an additional subbase layer in terms of response to static load. In reference (4) a soil support term was developed to correlate deflections on different foundations.



RELATIVE PERFORMANCE OF SUBBASES AND SUBGRADES

FIGURE 2.1

$$SS = \frac{U_1 + U_2}{T_{sg}}$$

Where SS = Soil support

U_1 = Seven day unconfined compressive strength of subbase

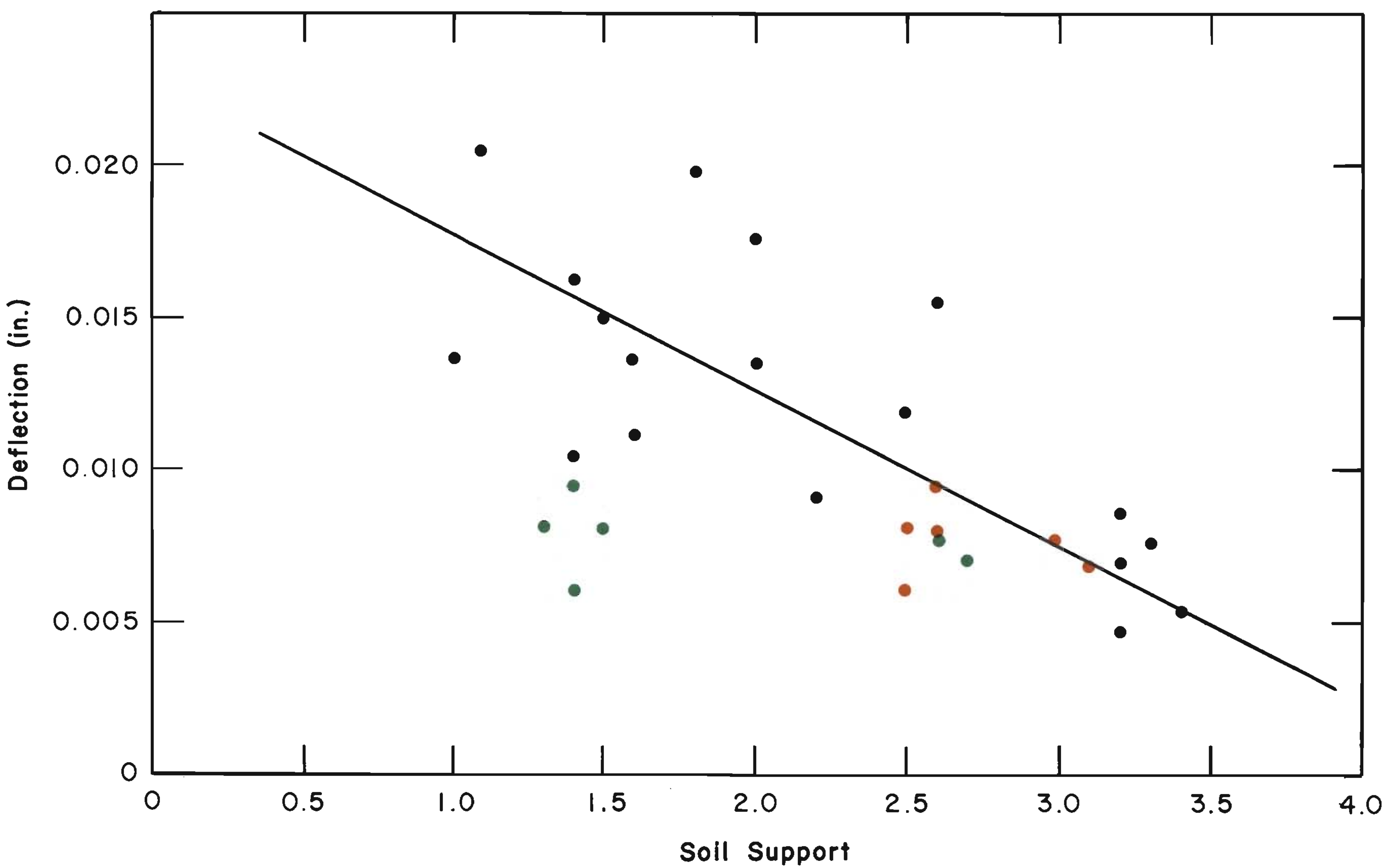
U_2 = Seven day unconfined compressive strength of lime stabilized subgrade

T_{sg} = Texas Triaxial class of subgrade

In Figure 2.2 the green data points represent pavements which have lime treated subgrades. The soil support value for these pavements was modified to include the treated subgrade layer as a second subbase. After inclusion in the soil support, the green data points moved to the positions shown in red. Now the red and black form the data set which displays the relation of deflection to soil support shown. Thus, the lime treated subgrade layers do strengthen the pavement system. However, this additional strength is usually not considered in the design stage. Usually the stabilization with lime is primarily for developing a workable subgrade or a construction platform.

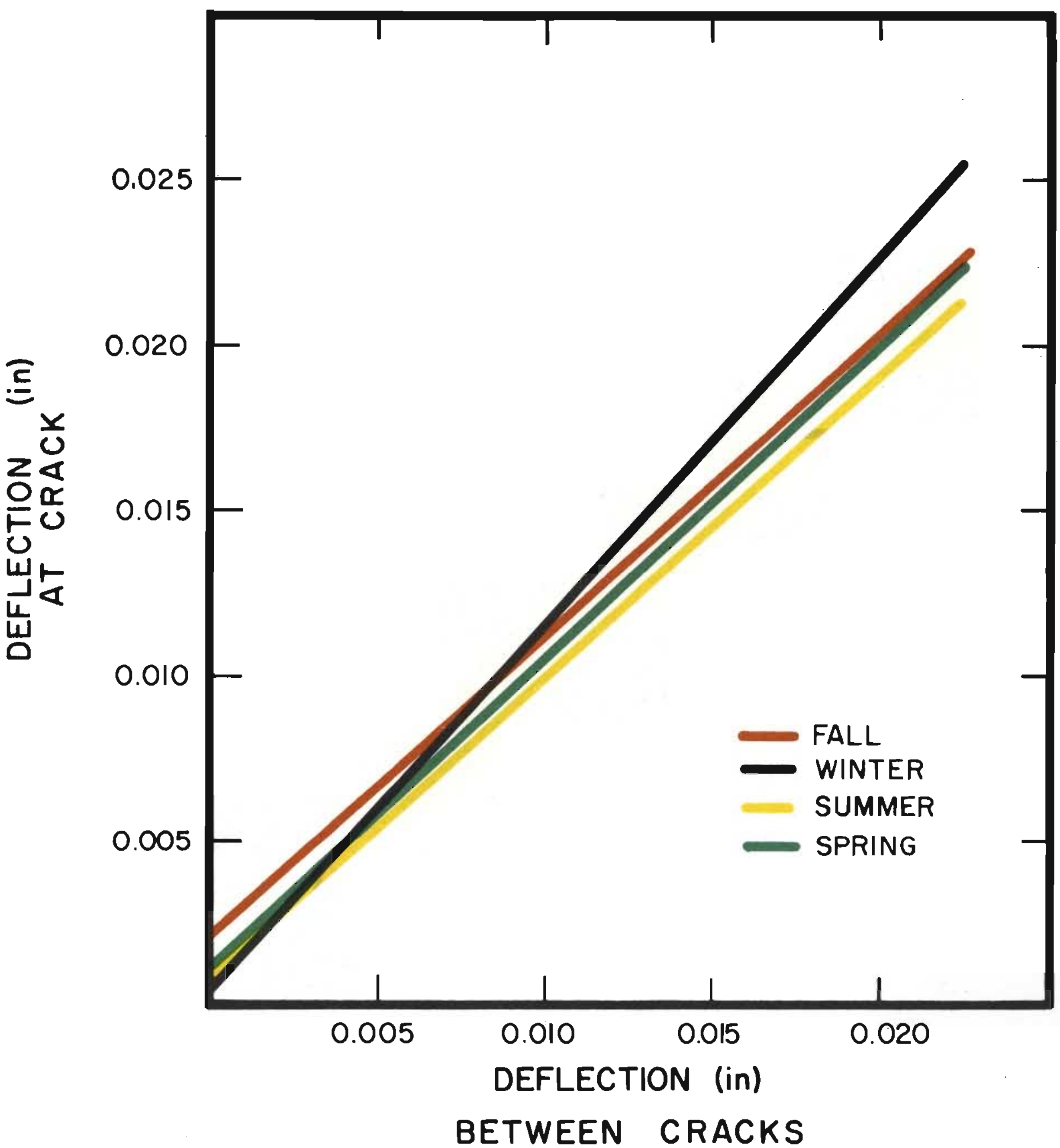
Another use of deflection measurements was a comparision of measured responses at crack positions and midway between cracks on the continuously reinforced concrete pavement. The continuity in the CRCP or load transfer across the volume change cracks is measured by the comparison shown in Figure 2.3. The lines shown in Figure 2.3 are regression lines for data taken at and between volume change cracks. Each of the four lines represent data recorded in one of the four respective seasons of the year.[5] Note that the lines are very near what would be a 45 degree line. Also the intercept is very small, smaller than the resolution of the Benkelman Beam. The design of the longitudinal reinforcing steel was identical in all of the pavements from which the data were gathered for the graphs in Figure 2.3. It is apparent from a deflection standpoint that the 0.5% steel design was performing satisfactory at the time of this investigation.

The deflection-percent steel relationship has been investigated on two experimental paving projects where more than one steel design was used. Figure 2.4 shows the relationships of per cent steel to deflection on two projects. The subbases and subgrades for these two projects were entirely different. This accounts for the vertical placement of the two data sets, i.e. data from two different populations. From Figure 2.4 it is believed that increases in per cent steel beyond the 0.5 will probably not decease deflection a significant amount. Thus, a design with more than 0.5 per cent steel will have a small amount of built-in insurance.



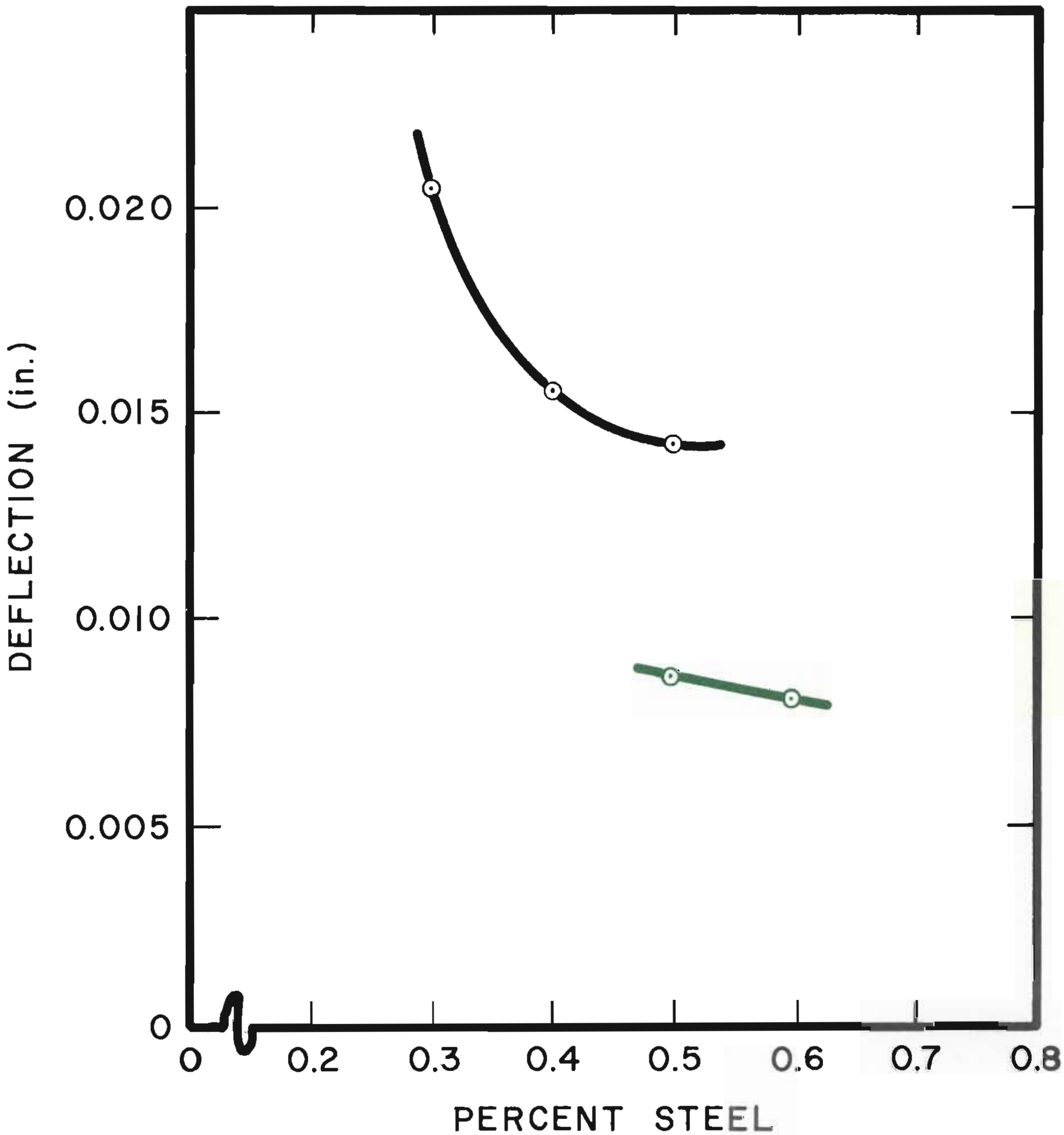
DEFLECTION AS A FUNCTION OF SOIL SUPPORT

Figure 2.2



COMPARISON OF DEFLECTION
AT AND BETWEEN CRACKS

Figure 2.3



RELATION OF PERCENT STEEL TO DEFLECTI

Figure 2.4

The response of pavements with different slab thickness was also measured by deflection. As expected, thinner slabs deflected more than thicker ones.[5] The data shown in Figure 2.5 were collected from pavements of identical design except for the slab thickness. In Figure 2.5 where six and eight inch pavements are compared, the red line represents the data. The AASHO and Westergaard analyses are shown in green and black respectively. Based on the limited amount of data shown, a satisfactory comparison exists between the data shown, other research and theory.

Crack Pattern

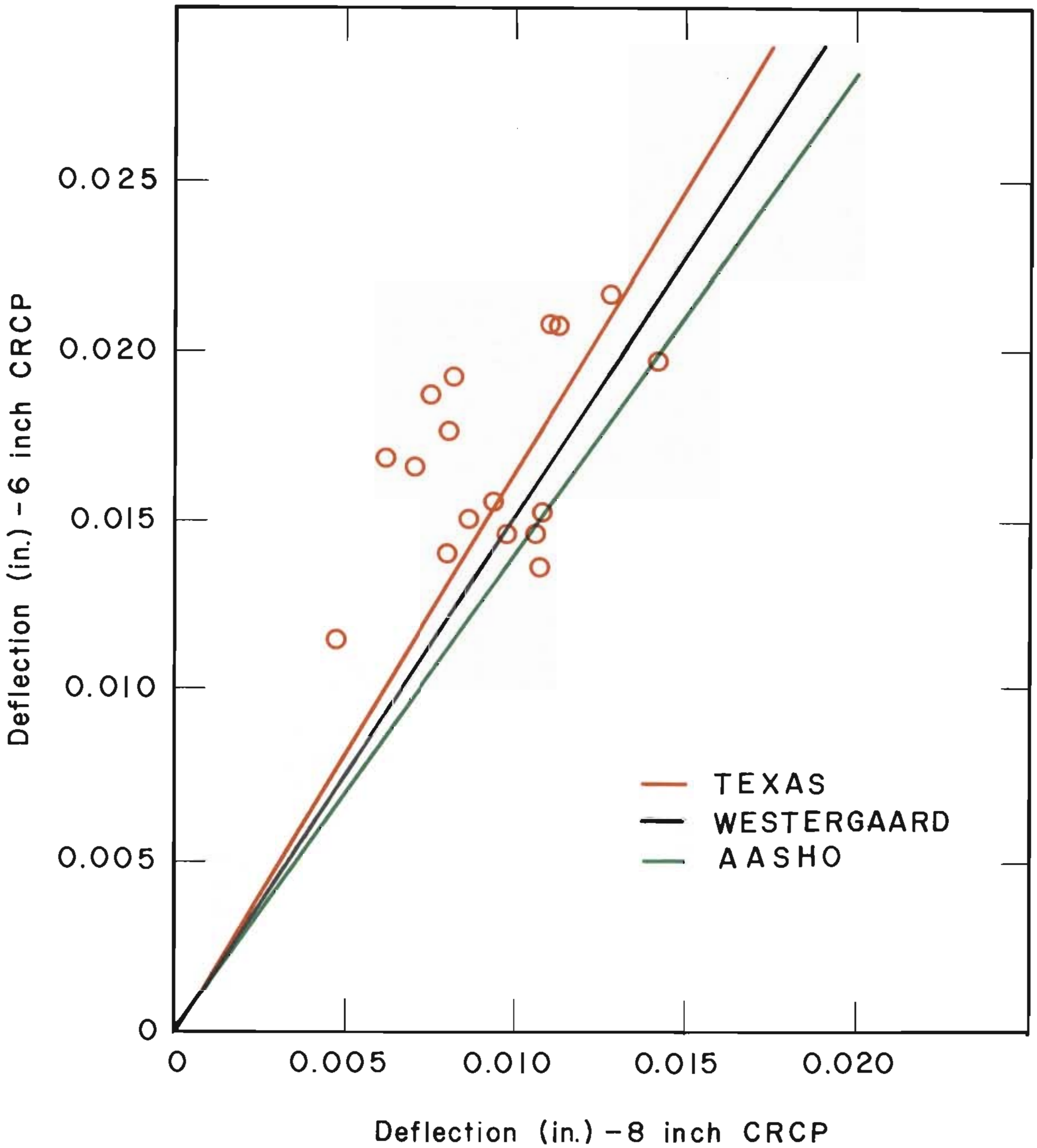
The development of the random crack pattern in CRCP is very complex. It is affected by the tensile strength of the concrete, percent steel shrinkage, curing temperature, friction between concrete and subbase, and the uniformity and homogeneity of the paving concrete.

Crack pattern development with time is displayed in Figure 2.6. The two pavements shown are identical in design except for the subbases. Both pavements were built in the same contract and are adjacent to one another. The red line represents a CRCP with 0.4 per cent longitudinal steel (deformed wire mat) on a lime stabilized gravel subbase. The black line shows the same pavement on a subbase of crushed limestone stabilized with asphalt emulsion. Here at an age of 600 days the pavement on the asphalt treated subbase has a crack spacing 23 per cent less than the pavement on the lime stabilized gravel. This difference in crack spacing can be attributed to the difference in subbase friction or resistance to movement.

The concrete tensile strength also affects the cracking pattern. A tensile test is not used for job control in Texas. Mid-point loading flexural strengths are determined and there is a correlation between the tensile and flexural strengths of concrete. Thus, in Figure 2.7 for two different curing conditions the seven day flexural strength is related to the average crack spacing as determined at a pavement age of 200 days.[12] As concrete strengths go up and curing temperatures go down, wider crack spacings result.

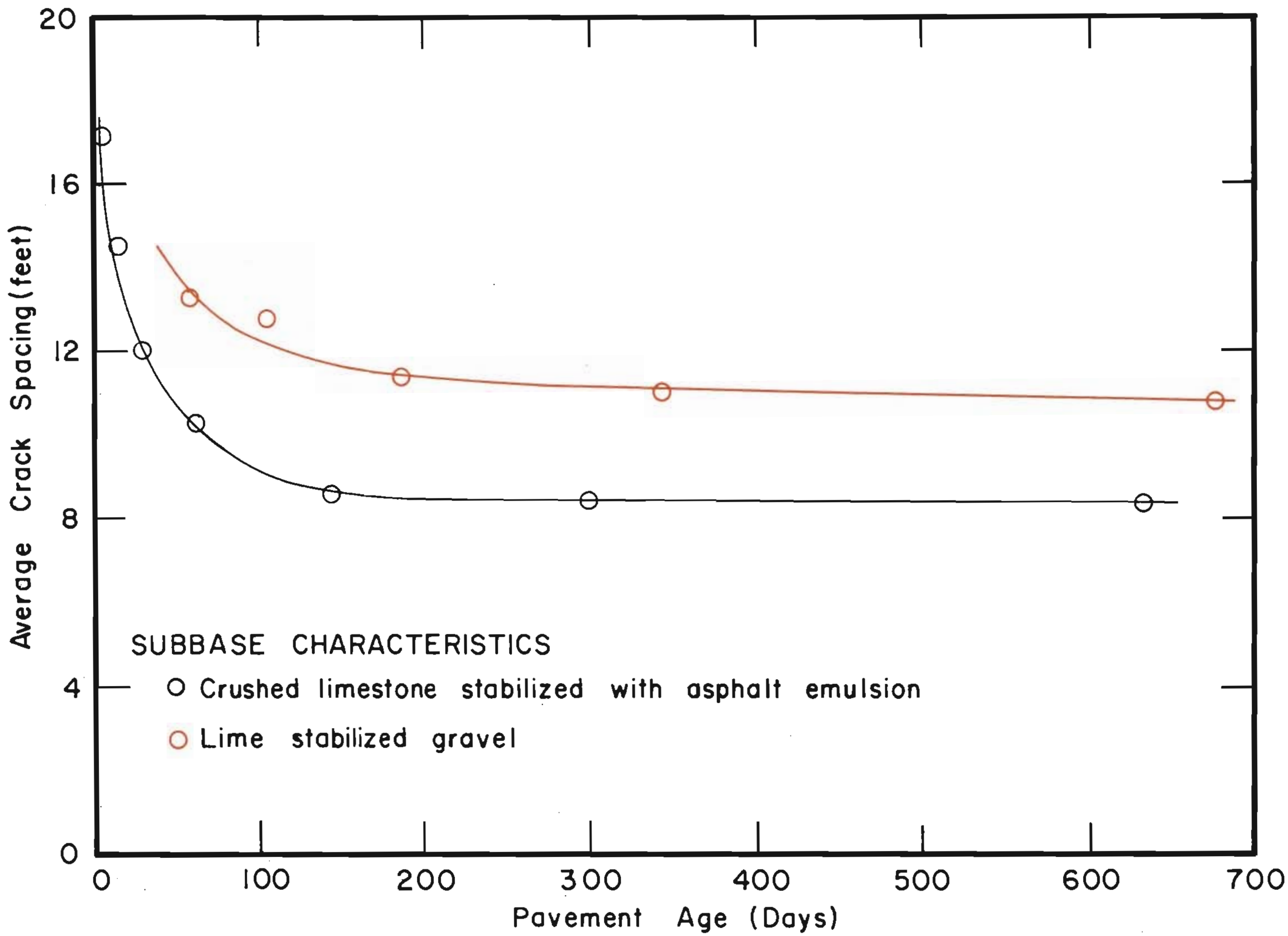
The ambient temperature, i.e. curing temperature of the pavement also affects the crack spacing significantly as shown in Figure 2.7 where the slope of the lines was a measure of the curing temperature. In Figure 2.8 the relation of curing temperature to crack spacing is shown. As temperature of pavement reduces, the crack spacing approaches infinity, i.e. spacings much larger than the optimum of five to eight feet. Data in Figures 2.7 and 2.8 are from one construction project.

The distribution of crack spacings in a given length of pavement is a good indicator of performance. A distribution such as shown in Figure 2.9 reflects satisfactory performance. Note that the distribution approaches a normal one. Cracking patterns such as this are representative of



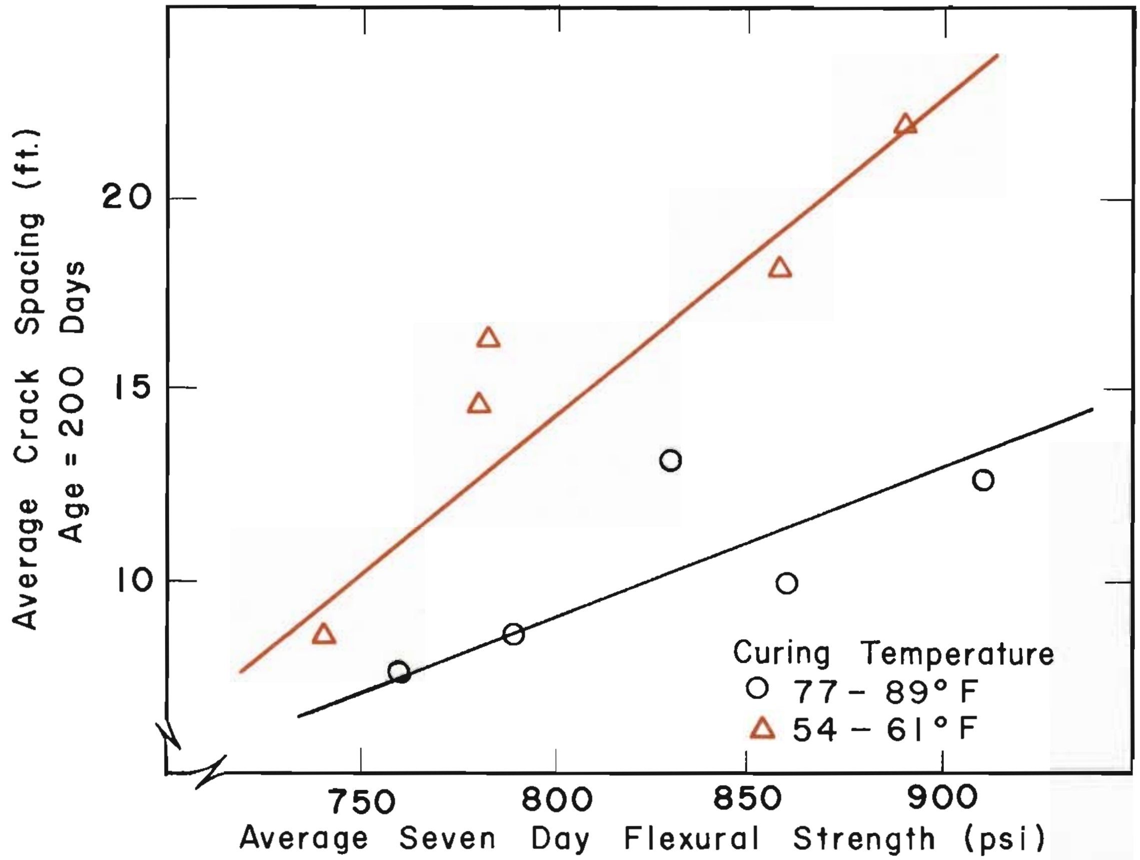
DEFLECTION COMPARISON —
SIX AND EIGHT INCH CRCP

Figure 2.5



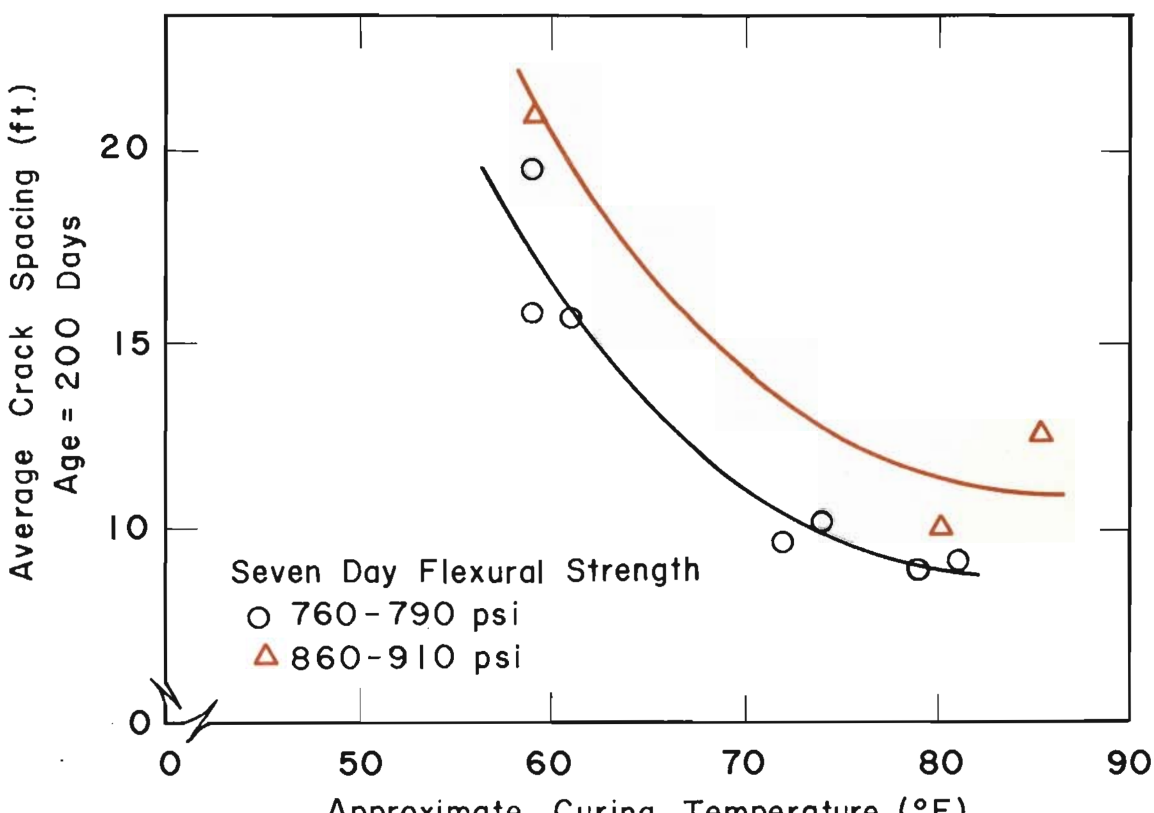
RELATION OF PAVEMENT AGE TO AVERAGE CRACK SPACING

Figure 2.6



RELATION OF FLEXURAL STRENGTH TO CRACK SPACING

Figure 2.7



RELATION OF CURING TEMPERATURE TO CRACK SPACING

Figure 2.8

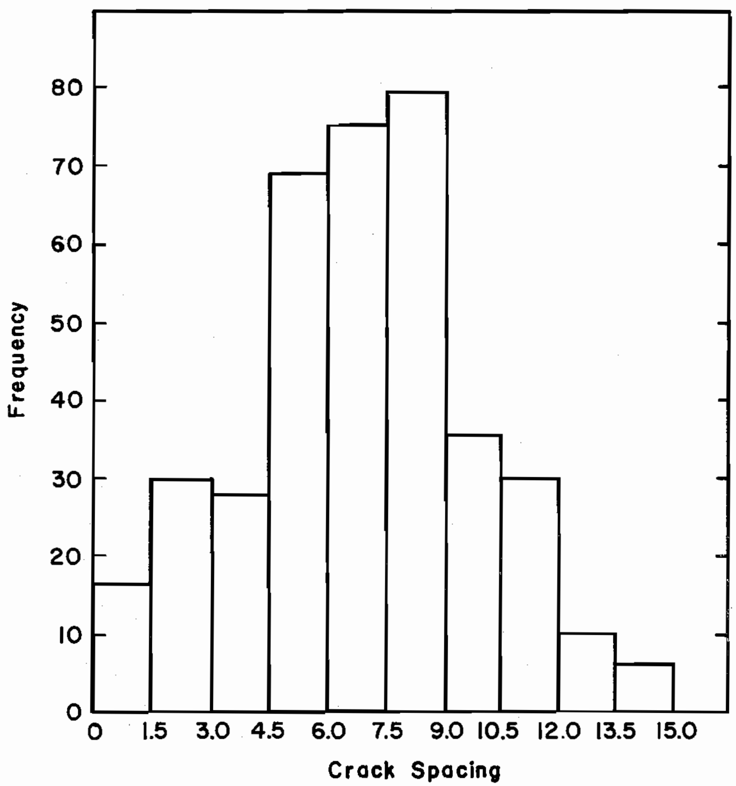
the random crack pattern desired in CRC pavement. The data shown in Figure 2.9 is from a pavement test section 2500 feet long. A distribution like this can only be obtained with uniform, homogenous, well consolidated concrete. Figure 2.10 shows a crack spacing distribution on a pavement test section also 2500 feet long where the concrete was neither uniform, homogenous, or well consolidated. Extensive areas of unconsolidation were present and most of the cracks formed over the transverse steel which is spaced on two foot centers. This is reflected in the skewed distribution. The frequency is very high for the range from 1.5 to 3.0 feet. In general, the normal distribution indicates satisfactory performance while the skewed distribution reflects unsatisfactory performance.

Pumping

Loss of uniform support under a rigid pavement usually causes distress in a pavement slab when heavily loaded. A survey of tests sections selected for this investigation indicated that pumping was not a problem on pavements with stabilized subbase layers. Figure 2.11 shows a qualitative evaluation of the pumping found on some of the test pavements selected for the overall experiment, both jointed and continuously reinforced.

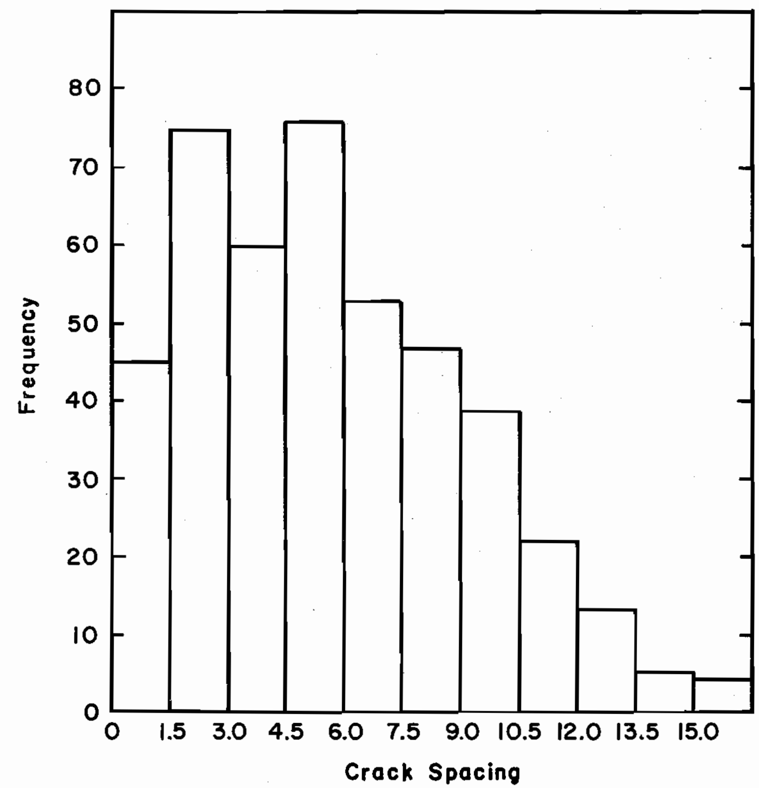
Other experience in Texas with lime stabilized subbases directly under CRCP has been somewhat unsatisfactory. Lime treated soil apparently loses some of its integrity when it becomes wet. After wetting it errodes and pumps like a fine grained material.

Signs of pumping such as water movement and material deposited on a paved shoulder are not always true pumping, i.e. removal of foundation from beneath the portland cement concrete pavement slab. Some pavement with sound, stabilized subbases have shown signs of pumping. For example, an eight inch CRC pavement with a asphalt stabilized subbase showed severe signs of pumping. The pavement had cement stabilized shoulders with one inch of asphaltic concrete as shoulder surfacing. The pavement edge was cored. It was found that when the shoulder base material was stabilized with cement, a small portion in the corner between the blanket subbase and the edge of the slab was either not mixed with cement or not compacted. It was this material that was being forced out by the pumping action of water in the joint between the shoulder and the slab. Figure 2.12 is a sketch of what was found. A careful examination of the core holes showed the small channel where the loose material had all been removed.



DISTRIBUTION OF CRACK SPACINGS IN A PAVEMENT WITH SATISFACTORY PREFORMANCE

Figure 2.9



DISTRIBUTION OF CRACK SPACINGS IN A PAVEMENT WITH UNSATISFACTORY PREFORMANCE

Figure 2.10

FIGURE 2.11

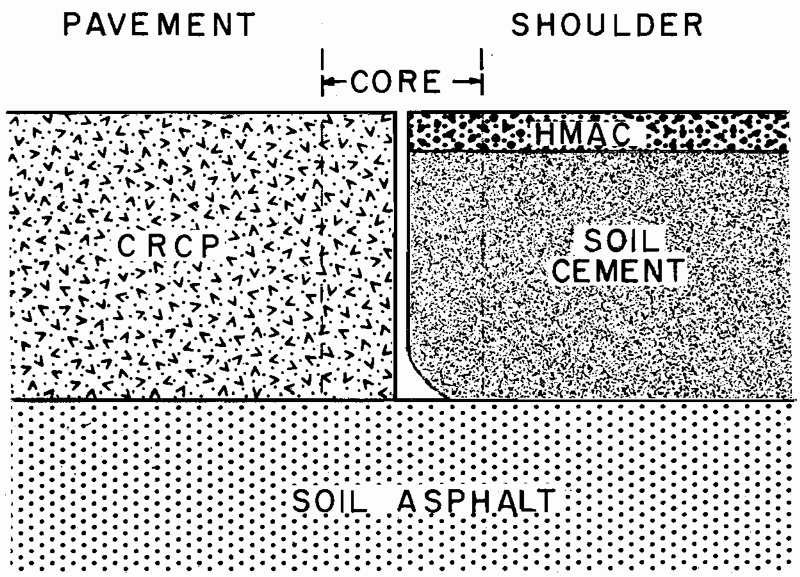
CONCRETE PAVEMENT PUMPING DATA

Numbers indicate degree of pumping: 1-heavy, 2-moderate, 3-trace, 4-clear water, 5 – no pumping

SUBGRADE SUPPORT	SUBBASE TYPE	SUBBASE MATERIAL	3,500,000 PSI LOW 580 PSI(–)	3,500,000 PSI MEDIUM 580-690 PSI	3,500,000 PSI HIGH 690 PSI(+)	5,500,000 PSI LOW 580 PSI(–)	5,500,000 PSI MEDIUM 580-690 PSI	5,500,000 PSI HIGH 690 PSI(+)
POOR 5.5 +	NON-STABILIZED	FINE GRAIN		C-5 J10-2			C–2	
		CRUSHED STONE		C-2,C-2 J9-3,J10-5			C-5 J8-1,J9-5,J10-3	C-2
	STABILIZED	CEMENT					C-2 ,C-5	C-3
		ASPHALT		C-3				
		LIME	C-5	C-5				
FAIR 5.0 – 5.5	NON-STABILIZED	FINE GRAIN		C-5			C-5 J8-1	
		CRUSHED STONE		C-5 J10-3			J9-3,J10-1	
	STABILIZED	CEMENT					C-5,C-5	
		ASPHALT		C-5				
		LIME		C-5				
GOOD 4.0 – 5.0	NON-STABILIZED	FINE GRAIN		C-1 J10-3			C-2,C-2 J9-5	
		CRUSHED STONE		C-3 J9-5,J10-1			C-5 J8-1, J9-5	C-2
	STABILIZED	CEMENT		C-5			C-5,C-5 J9-3,J10-2	C-5
		ASPHALT		C-5				
		LIME		C-5				

MODULUS OF ELASTICITY / CONCRETE FLEXURAL STRENGTH

C – CRCP
J10, J9, J8 - JCP



CROSS SECTION OF PAVEMENT- SHOULDER JOINT

Figure 2.12

Traffic

Since the test sections for this project were selected at the inception of the project, the traffic load applications have been estimated several times. The data collected was for one direction only from the date of completion to the time of data collection.

The traffic data, total for one direction, is in reality all forced into one lane as far as the number of 18 kip applications is concerned. This may be reasonable since almost all the test sections were on roadways with two traffic lanes in each direction and heavily loaded vehicles usually travel the outside lane where all test sections are located.

The pavement serviceability-performance concept developed at the AASHO Road Test relates the traffic load application to a performance index known as the Present Serviceability Index or PSI.[13] The CHLOE profilometer was used to determine the PSI of each test section in the experiment. Only one value of PSI was obtained for each section, i.e. the PSI's immediately after construction were unknown.

The test sections are in general all about the same age, thus a relative comparison of PSI's would be reasonably valid. The relation of PSI to traffic is exhibited. The data is limited, however, it is all that exists.

The data indicates that the average for the sections was in the range of 4.0 to 4.2. These pavements were all three to six years old at the time of measurement. Although these pavements were not very old, this does indicate the ability of continuously reinforced concrete pavement to maintain a high level of serviceability.

For the test sections in this experiment, corresponding PSI and traffic data were assembled.

This assemblage of data is shown graphically in Figure 2.13. In Figure 2.13 all pavements in the experiment factorial are shown with no attempt to differentiate between pavements of different design. The line shown on the graph represents the AASHO equation which related the PSI of a 10 inch jointed reinforced concrete pavement to traffic. The initial PSI of rigid pavements at the AASHO Road Tests was 4.5, thus the line passes through 4.5 on the vertical axis on the semilog graph of PSI and traffic.

Figure 2.14 shows the same data as Figure 2.13 with an attempt to differentiate between pavements with stabilized subbases and pavements with unstabilized subbases. Data are inconclusive.

Figure 2.15 compares the performance of jointed and continuously reinforced pavements. The sample of jointed pavements is smaller than that of the continuously reinforced. The observations do indicate that the continuously reinforced is performing with a higher serviceability index than are the jointed concrete pavements as built in Texas. The lines on this figure again represent the AASHO correlation of traffic to serviceability index for eight and ten inch jointed reinforced concrete pavements.

In Figure 2.16 the Texas performance data is compared to that of several Illinois eight inch continuously reinforced concrete pavements.[14] The Illinois data are from pavements having a range of 0.3 to 1.0 percent longitudinal steel.

The foregoing figures indicate that the AASHO equations cannot be applied directly as they predict a higher level of performance than has been determined from in-service pavements. This same finding was in an investigation on pavements in Illinois.[14] The trend of loss in serviceability with increased traffic is correct and valid. However, the PSI's measured for this investigation indicate that the average initial PSI after construction is something less than 4.5 as built at the AASHO Road Test.

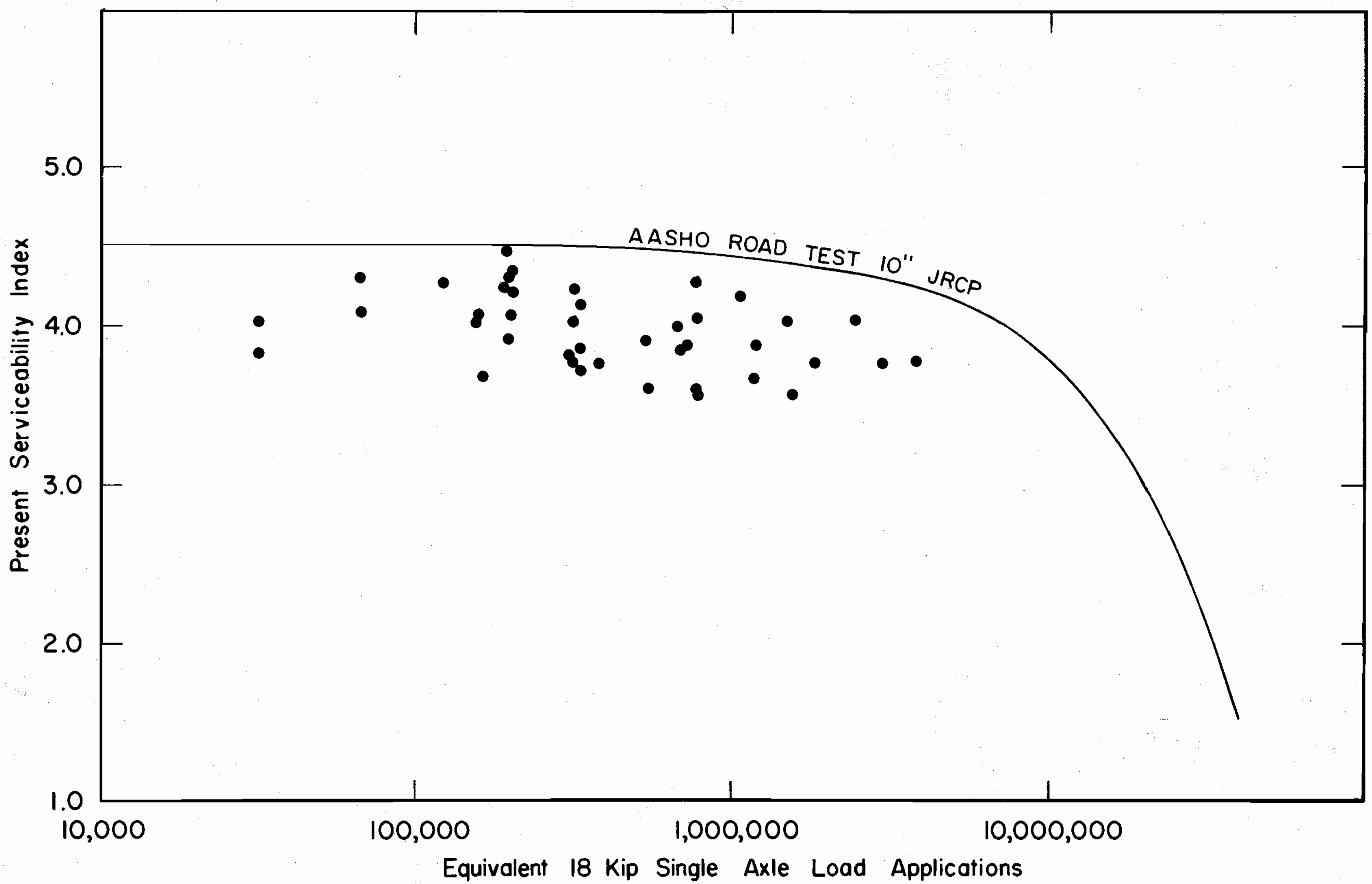


Figure 2.13 — PERFORMANCE OF EIGHT INCH CRCP AS COMPARED TO THE AASHO PERFORMANCE EQUATION FOR TEN INCH JOINTED RIGID PAVEMENT

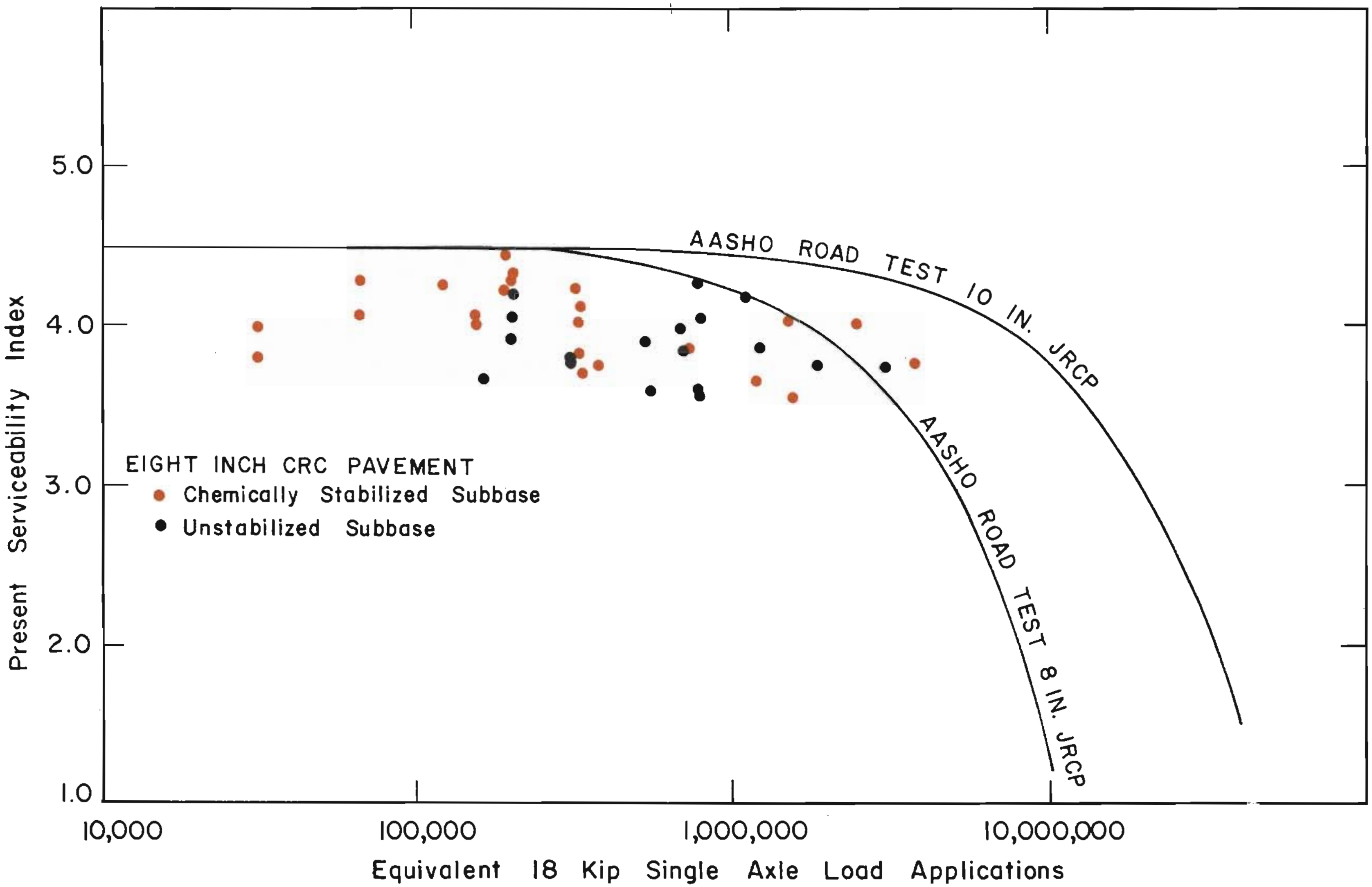


Figure 2.14 — PERFORMANCE COMPARISON OF PAVEMENTS WITH STABILIZED AND UNSTABILIZED SUBBASES

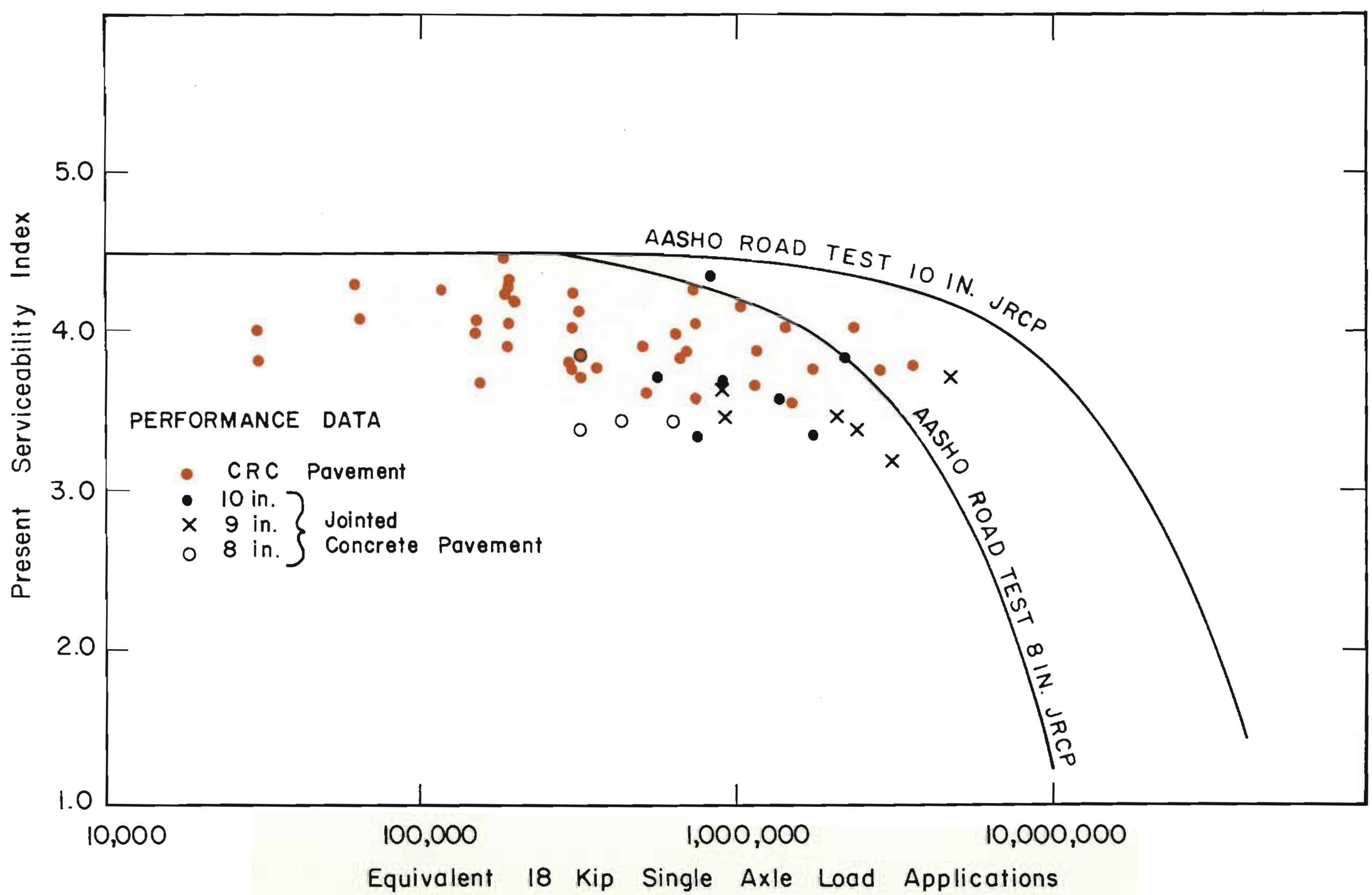


Figure 2.15 – COMPARISON OF PERFORMANCE DATA FROM JOINTED AND CONTINUOUS PAVEMENTS WITH AASHO PERFORMANCE EQUATIONS

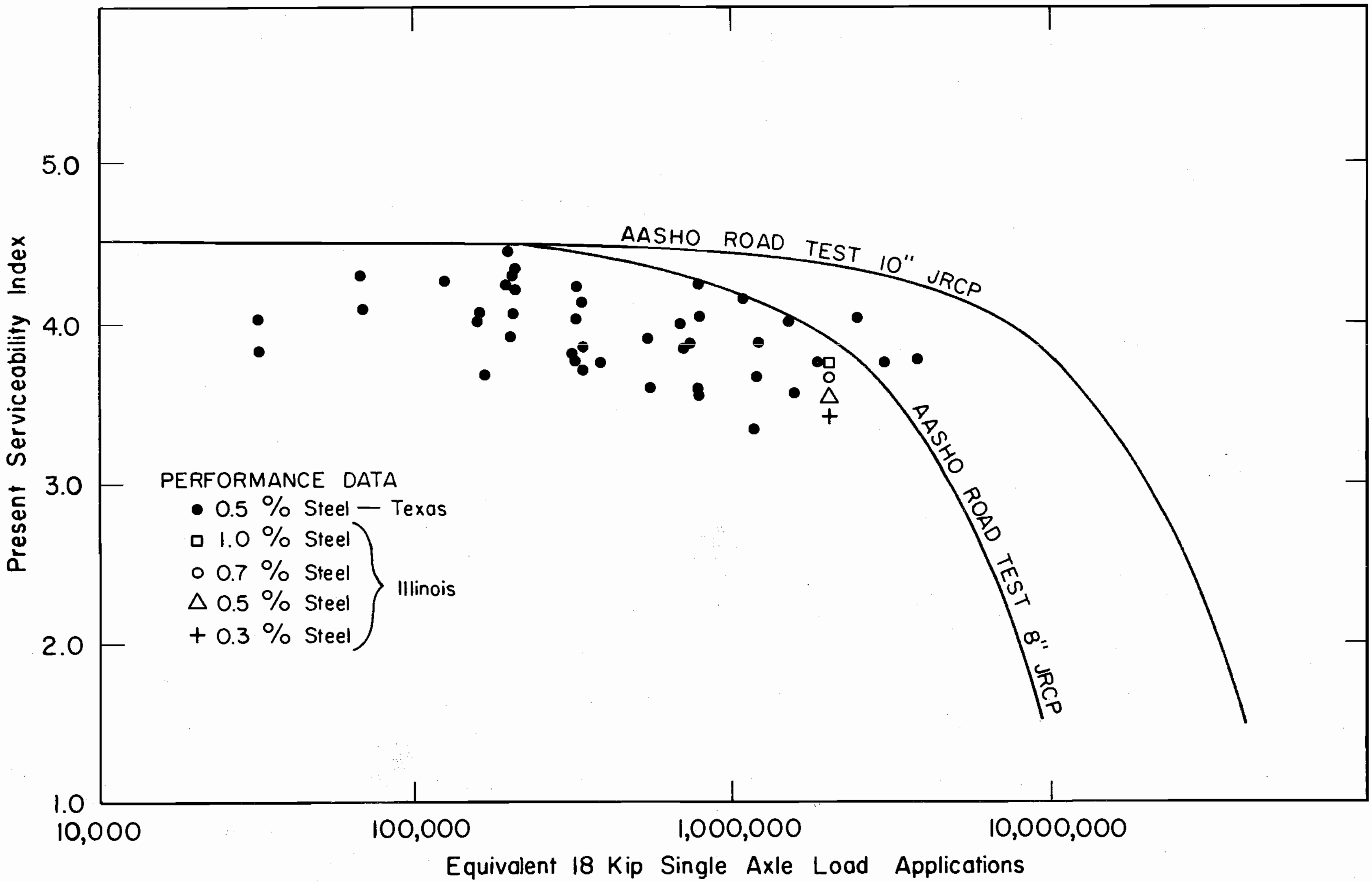


Figure 2.16 — COMPARISON OF PERFORMANCE OF EIGHT INCH CRCP IN TEXAS AND ILLINOIS WITH AASHO PERFORMANCE EQUATIONS

III. DESIGN

From the research summarized in Chapter II several empirical equations were developed. In each case the equations were statistical models which correlate the parameters involved. From the study of steel strain the following model was developed for stress in the longitudinal steel.[2]

$$S = A_1 + A_2E_cZ_c + A_3E_cZ_c\left(\frac{1}{P}\right)^2 + A_4E_c\alpha_c \Delta T + A_cE_e\alpha_c \Delta T\left(\frac{\bar{X}}{P}\right)^2 \quad \ldots \quad 3.1$$

Where A_1, A_2, A_3, A_4, A_5 are constants which were determined from the data and all other terms are as defined in the NOMENCLATURE.

The above equation can be solved for P, the percent longitudinal steel. Thus it takes the form

$$P = \left[\frac{A_3E_cZ_c + A_5E_c\alpha_c \Delta T\bar{X}^2}{S - (A_1 + A_2E_cZ_c + A_4E_c\alpha_c \Delta T)}\right]^{0.5} \quad \ldots \quad 3.2$$

Also a result of the stress and concrete movement investigation was an empirical equation for crack width in the concrete.[2] Model number DK1 has been selected from reference (2) for this exhibit.

$$\Delta X = \frac{\bar{X}(\Delta Z + \alpha_c \Delta T)}{e^{K_1P}} \quad \ldots \quad 3.3$$

Where K_1 is a constant determined from the data and all other terms are as defined in the NOMENCLATURE.

From the analysis of the load-deflection data collected in this research study, the final result was an empirical equation or model which correlated the parameters in the study to the deflections measured in the field.[4,5]

The first such study evaluated the effects of temperature and crack spacing, while the second, a statewide investigation determined the effect of subbase, subgrade and materials properties. The final model for predicting the edge deflection of a CRC pavement was

$$D_c = \frac{A_o L\, 10^{B_5 \Delta X}\, \bar{X}^{B_4}\, Tsg^{0.25B_3}}{D^{1.75}\, E^{B_2} U^{0.25B_3}\, 10^{0.0147T}} \quad \ldots\ldots\ldots\ldots 3.4$$

Where A_o, B_2, B_3, B_4, and B_5 are constants determined from the data.

Equation 3.4 could be rearranged such that a solution for slab thickness, D could be obtained. Thus it follows that the equation would take the form

$$D = \left[\frac{A_o L\ 10^{B_5\, \Delta X_{\bar{X}}}\ {}^{B_4}\ U^{0.25B_3}\, Tsg^{0.25B_3}}{D_c\ E^{B_2} U^{0.25B_3}\ 10^{0.014T}}\right]^{0.571} \quad \ldots\ldots\ldots 3.5$$

Thus two empirical equations have been formulated which may serve as design aids. First, equation 3.2 displays the percent of longitudinal steel in terms of the material properties of the concrete, environment and the stress. The values for the constants are shown in Equation 3.6

$$P = \left[\frac{E_c(2.46\ Z_c\ -\ 0.002\ {}_c\ \Delta T\ X^2)}{S\ -\ (3400\ -\ E_c(18.22\ Z_c+\ 27.79\,\alpha_c \Delta T))}\right]^{0.5} \quad \ldots\ldots\ldots 3.6$$

Now by having the concrete modulus of elasticity, coefficient of thermal expansion for concrete, range of temperature from construction to extreme cold temperature, shrinkage of concrete and a desired crack spacing, the necessary percentage of steel can be computed. See Appendix A for solution of example problem.

Slab thickness may be determined from Equation 3.5. The slab thickness would be based on a static load condition and a maximum allowable edge deflection. Equation 3.7 is Equation 3.5

with constants substituted in

$$D = \left[\frac{0.378L\ 10^{6.341}\ \Delta X_{\bar{X}}^{0.027}\ Tsg^{0.163}}{D_c\ E^{0.168}\ U^{0.163}\ 10^{0.0147\ T}} \right]^{0.571} \qquad \ldots\ldots\ldots\ 3.7$$

The value of crack width (ΔX) may be computed from Equation 3.3 which is rewritten here as Equation 3.8.

$$\Delta X = \frac{\bar{X}\ (\Delta Z + \alpha_c \Delta T)}{e^{1.468P}} \qquad \ldots\ldots\ldots\ldots\ 3.8$$

or it may be computed from the following equation which was also developed along with Equation 3.8.[2]

$$\Delta X = \frac{\bar{X}(\ Z + \alpha_c\ \Delta T)}{1 + K_2\ P} \qquad \ldots\ldots\ldots\ldots\ 3.9$$

Where $K_2 = 2.553$

Either Equation 3.8 or Equation 3.9 will give a satisfactory answer; however, Equation 3.8 is recommended.

The load term (L) in Equation 3.7 was replaced by 18 since the load used in the experiment was 18 kips. For design it may be desirable to use a temperature differential (T) equal to Zero. Thus, other than the allowable deflection, only strength parameters need to be determined.

The allowable deflection (D_c) is a subject which may be conjectured. A deflection of 0.012 in. has been suggested for design purposes where the average of the single-axle loads is in the neighborhood of 15,000 lb.[15] Based on AASHO Road Test Work, a reinforced jointed concrete pavement after being fatigued with 7,000,000 18 kip equivalencies should have an edge deflection of about 0.014 in. at PSI of 2.5. It is believed that CRCP performs equally well and better than JCP in Texas, thus based on the fatigue relationship the 0.014 in. may be considered for CRC pavement.[13]

For design purposes a maximum allowable deflection 0.012 in. is recommended at this time. This is the most conservative from the available literature. The 0.012 in. is also satisfactory from the standpoint of experience in having measured deflections on many CRC pavements of varying design and service level.

Figure 3.1 is a graphical solution of Equation 3.4. The nomograph may be used to solve for slab thickness with the following parameters being known.

1. Subgrade triaxial classification
2. Subbase unconfined compressive strength at seven days
3. Maximum allowable deflection

In the development of the nomograph in Figure 3.1 the parameters in Equation 3.4 not listed above were assigned the following values.

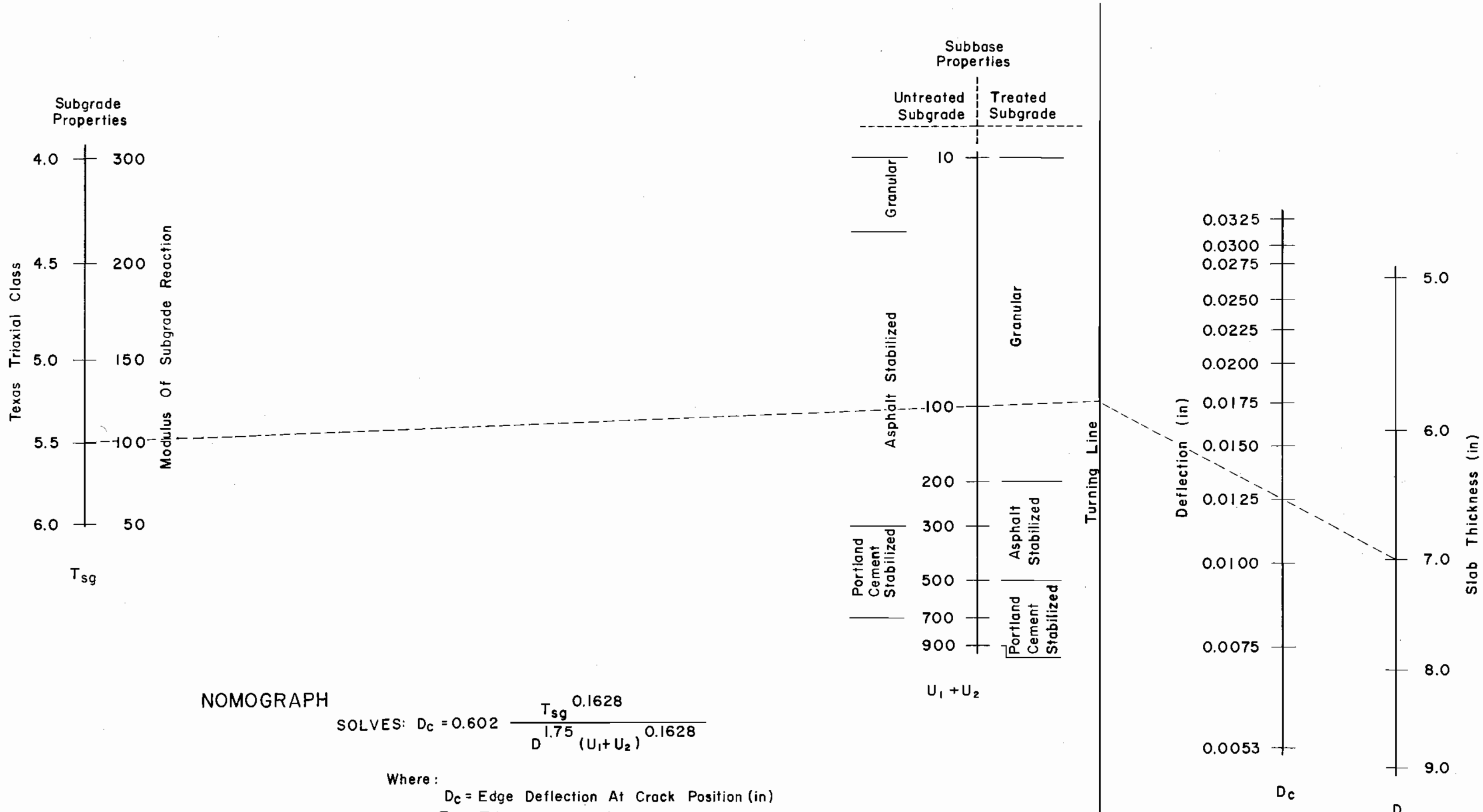
$\overline{X}$ (crack spacing) = 5.0 feet

ΔX (crack width) = 0.010 in

T (temperature differential) = 0

L (Load) = 18 kips

E = 5.5×10^6 psi



NOMOGRAPH

SOLVES: $D_c = 0.602 \dfrac{T_{sg}^{\;0.1628}}{D^{1.75}\,(U_1 + U_2)^{0.1628}}$

Where:

D_c = Edge Deflection At Crack Position (in)

T_{sg} = Triaxial Class Of Subgrade

D = Slab Thickness (in)

U_1 = Subbase unconfined compressive Strength At Seven Days (psi)

U_2 = Unconfined Compressive Strength Of Treated Subgrade (psi)

DESIGN CHART

CRC PAVEMENT

Figure 3.1

EXAMPLE PROBLEM

T_{sg} = 5.5

$U_1 + U_2$ = 100

D_c = 0.0125

Ans. D = 7.0 in.

IV. SUMMARY

This performance study of continuously reinforced concrete pavement encompassed a wide variety of the environmental elements. The pavements studied were located throughout the state. This research effort was conducted over a period of five years during which much data was collected, many analyses made and numerous reports written. The following conclusions are for the entire research study, some of which are being repeated from earlier reports on this project. In general they summarize the relative performance of continuously reinforced concrete pavement as evaluated in this investigation.

1. Steel Strain
   a. Temperature stresses are a direct function of the concrete's modulus of elasticity and coefficient of thermal expansion.
   b. Stress is an inverse function of percent longitudinal steel.
   c. Stress is a direct function of the spacing of volume change cracks.
   d. Stress is a direct function of concrete shrinkage.
2. Deflection
   a. Pavements with good subgrades deflected less than those with poor subgrades.
   b. Pavements with lime stabilized subgrades performed better than identical pavements with no subgrade treatment.
   c. For pavements with 0.5 steel over a wide variation of support and environmental conditions the transverse cracks were small enough to retain sufficient aggregate interlock to maintain approximately 100 percent load transfer.
   d. Pavements with more steel respond more favorably, however beyond the 0.5 or 0.6 percent point additional steel will not decrease deflection significantly.
   e. The relationship of slab thickness to deflection determined from this research is in agreement with accepted theory and other research.
   f. Pavements with stabilized subbases are superior to those with non-stabilized subbases.
3. Crack pattern
   a. There is a relation between pavement age and crack pattern on an individual project basis.
   b. The subbase friction affects the average crack spacing.
   e. The flexural strength of the concrete has a direct effect on the crack spacing.
   d. The crack spacing is related to the curing temperature.

e. The distribution of crack spacings in a known length of pavement is an indicator of performance.

4. Pumping

a. Percentage wise, twice as many jointed concrete pavements as CRC pavements with similar subbases were found pumping.

b. Lime stabilized subbases will pump if they are not protected with a non-errosive surface.

c. Signs of pumping are not always proof that a pavement's subbase is being erroded.

5. Traffic

a. The performance data shows the trend set forth by the AASHO equations, but with a somewhat lower initial serviceability index.

b. CRC pavements show a significantly higher serviceability index than jointed concrete pavements as built in Texas.

c. CRC pavements in Texas are performing equally well to others in this country.

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APPENDIX A

Example Problem - Solution for Per Cent Steel

Example Problem - Solution for Percent Steel

The percent steel for a given set of conditions can be completed by using the empirically developed Equation 3.6.

$$P = \left[\frac{E\ (2.46Z_c - 0.002 \quad _c\ \Delta T\ \bar{X}^2)}{S - \left[3400 - E_c\,(18.22Z_c + 27.79 \quad _c\ \Delta T)\right]} \right]^{0.5}$$

For the following set of conditions the required longitudinal steel percentage is computed.

$E_c = 4.0 \times 10^6$ psi

$Z_c = 100.0 \times 0^{-6}$ in/in

$\alpha_c = 5.0 \times 10^{-6}$

$\Delta T = 50^oF$

$\bar{X} = 10$ feet

$S = 33{,}000$

Substituting the above values

$$P = \left[\frac{4.0 \times 10^6 \left[2.46\ (100\text{x}10^{-6}) - 0.002\ (5.0\text{x}10^{-6})\ (-50) \times 10^2\right]}{33{,}000 - \left[3400 - 4.0\text{x}10^6 \left[18.22\ (100\text{x}10^6) + 27.79(5.0\text{x}10^{-6})(-50)\right]\right]} \right]^{0.5}$$

Simplifying

$$P = \left[\frac{4.0 \times 10^6\,(296\text{x}10^{-6}\ )}{33{,}000 - \left[3400 + 20{,}504\right]} \right]^{0.5}$$

$$P = \left[\frac{1184}{9096} \right]^{0.5}$$

$$P = 0.4\% \qquad \text{(Answer)}$$