TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/TX-88+422-2		

4. Title and Subtitle	5. Report Date
DESIGN RECOMMENDATIONS FOR STEEL REINFORCEMENT OF CRCP	November 1987
	6. Performing Organization Code
7. Author(s) Mohammad F. Aslam, C. L. Saraf, Ramon L. Carrasquillo, and B. Frank McCullough	8. Performing Organization Report No. Research Report 422-2
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin Austin, Texas 78712-1075	10. Work Unit No.
	11. Contract or Grant No. Research Study 3-8-86-422
12. Sponsoring Agency Name and Address Texas State Department of Highways and Public Transportation; Transportation Planning Division P. O. Box 5051 Austin, Texas 78763-5051	13. Type of Report and Period Covered Interim
	14. Sponsoring Agency Code

15. Supplementary Notes

Study conducted in cooperation with the U. S. Department of Transportation, Federal Highway Administration. Research Study Title: "Evaluation of Pavement Concrete Using Texas Coarse Aggregates"

16. Abstract

The primary objective of this study was to evaluate the effects of the variations in properties of concrete mixes composed of limestone and siliceous river gravel aggregates on the design and performance of CRC pavements. Laboratory testing of concrete mixes composed of these two aggregate types was carried out at the Balcones Research Center, The University of Texas at Austin. A statistical analysis was performed on these laboratory measurements to develop models to predict concrete properties for the two aggregate types. These models reflect differences in the properties of concrete mixes composed of limestone and siliceous river gravel aggregate types. Utilizing these concrete property models and formulating a factorial based on environmental conditions, pavement geometry, and steel reinforcement variables, an analysis was performed with the CRCP-4 computer program. The pavement performance predictions from this program were used to develop aggregate based CRC pavement steel reinforcement design models. The variation in concrete properties due to the choice of limestone or siliceous river gravel aggregates was accordingly translated into different steel reinforcement requirements for the two aggregate types. As a further refinement of the CRC pavement design procedure a concept of design reliability based upon the observed field performance of pavements has been developed. This concept, which identifies the aggregate type, should be incorporated into the criteria for developing design recommendations.

17. Key Words	18. Distribution Statement
rigid pavement, continuously reinforced concrete pavement (CRCP), limestone aggregate concrete mix, siliceous river gravel concrete mix, steel design	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	48	

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DESIGN RECOMMENDATIONS FOR STEEL REINFORCEMENT OF CRCP

by

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Ramon L. Carrasquillo
B. Frank McCullough

Research Report Number 422-2

Research Project 3-8-86-422
Evaluation of Pavement Concrete Using Texas Coarse Aggregates

conducted for

Texas State Department of Highways
and Public Transportation

in cooperation with the

U.S. Department of Transportation
Federal Highway Administration

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

November 1987

PREFACE

This is the second report for Research Project 3-8-86-422, "Evaluation of Pavement Concrete Using Texas Coarse Aggregates." The research for this project was conducted at the Center for Transportation Research (CTR), The University of Texas at Austin, as part of the Cooperative Highway Research Program sponsored by the Texas State Department of Highways and Public Transportation (SDHPT) and the Federal Highway Administration (FHWA).

The purpose of this report is to summarize the findings that led to the development and implementation of the revised concrete pavement details of continuously reinforced steel bars for the State of Texas. Work is in progress to test the concrete mixes containing coarse aggregates other than limestone and siliceous river gravel. The results of these future studies will be incorporated into the existing specifications for steel bars.

We are indebted to all the members of the CTR staff and the graduate students who participated in the activities of this project. Thanks are due to Peggy Carrasquillo, who supervised the laboratory testing of samples; Terry Dossey, for his computer analysis of data; Lyn Gabbert, for typing the manuscript of this report; and Michele Mason Sewell, for drafting the figures.

Thanks are extended to the Texas State Department of Highways and Public Transportation personnel for their cooperation, in particular Mr. James Brown and Mr. Jerry Daleiden.

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B. Frank McCullough

LIST OF REPORTS

Report No. 422-1, "Coarse Aggregates for PCC Pavements—Pilot Study Evaluation," by William J. Green, Ramon L. Carrasquillo, B. Frank McCullough, and C. L. Saraf, presents the laboratory measurements of concrete properties for Texas coarse aggregates, siliceous river gravel, and crushed limestone, determines their respective pavement performance, and develops a set of predictive equations which can forecast concrete property behavior by coarse aggregate type.

Report No. 422-2, "Design Recommendations for Steel Reinforcement of CRCP," by Mohammad F. Aslam, C. L. Saraf, Ramon L. Carrasquillo, and B. Frank McCullough, describes the development of steel design for CRC pavements using concrete mixes containing limestone or siliceous river gravel aggregates.

ABSTRACT

The primary objective of this study was to evaluate the effects of the variations in properties of concrete mixes composed of limestone and siliceous river gravel aggregates on the design and performance of CRC pavements. Laboratory testing of concrete mixes composed of these two aggregate types was carried out at the Balcones Research Center, The University of Texas at Austin. A statistical analysis was performed on these laboratory measurements to develop models to predict concrete properties for the two aggregate types. These models reflect differences in the properties of concrete mixes composed of limestone and siliceous river gravel aggregate types. Utilizing these concrete property models and formulating a factorial based on environmental conditions, pavement geometry, and steel reinforcement variables, an analysis was performed with the CRCP–4 computer program. The pavement performance predictions from this program were used to develop aggregate-based CRC pavement steel reinforcement design models. The variation in concrete properties due to the choice of limestone or siliceous river gravel aggregates was accordingly translated into different steel reinforcement requirements for the two aggregate types. As a further refinement of the CRC pavement design procedure, a concept of design reliability based upon the observed field performance of pavements has been developed. This concept, which identifies the aggregate type, should be incorporated into the criteria for developing design recommendations.

KEYWORDS: Rigid pavement, continuously reinforced concrete pavement (CRCP), limestone aggregate concrete mix, siliceous river gravel aggregate concrete mix, steel design for CRCP.

SUMMARY

This report describes the development of steel design algorithms based upon aggregate type. Laboratory testing was performed on concrete mixes containing limestone (LS) and siliceous river gravel (SRG) aggregates. A statistical analysis of these laboratory measurements allowed the development of prediction models for concrete mix properties containing LS and SRG aggregates. These models were later utilized in an analysis with the CRCP-4 computer program to develop steel reinforcement design models for the pavements to be built with limestone or siliceous river gravel aggregates. The design models predict different steel requirements for the two aggregate types. A probabilistic approach based upon the observed performance of the pavements in the field is introduced in this report. This approach, when incorporated in the CRC pavement design procedure, provides a method for comparing the expected performance of different steel reinforcement designs.

IMPLEMENTATION STATEMENT

Preliminary design recommendations based upon the models introduced in this report had already been prepared by the Texas SDHPT at the time this report was written. It is anticipated that the results of further studies will provide appropriate information to prepare specifications for reinforcement steel in pavements using various types of coarse aggregates found in Texas.

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CHAPTER 1. INTRODUCTION

BACKGROUND

Design of steel reinforcement for continuously reinforced concrete (CRC) pavement has often been a problem for the engineer. The complexity in this problem arises from the number of variables involved and the difficulty in quantifying those variables.

Past practice for the design and construction of these pavements has not considered the variation in concrete properties that may be attributed to the use of different coarse aggregates. A large volume of the concrete mix is occupied by the coarse aggregates. Accordingly, variations in the properties of the coarse aggregate types influence the material properties of the concrete mix. These differences in concrete properties should ideally be reflected in different design requirements, such as variation in steel reinforcement due to the choice of a particular coarse aggregate. A rational design approach analyzing the factors influencing CRC pavement performance was issued in 1981 after suggestions presented in CTR Report 177-22F, "Summary and Recommendations for the Implementation of Rigid Pavement Design." Although the design process can recognize performance differences of coarse aggregate types, the selection of the coarse aggregate type used during construction is left to the contractor. Hence, as long as the aggregate meets gradation and physical requirements, the basic assumption is that all aggregates are equivalent in performance and thus acceptable. Field observation has strongly refuted this hypothesis, since pavements built with different coarse aggregate types have shown significant variation in performance (Ref 1).

OBJECTIVES OF THE PROJECT

In Texas most concrete pavements are constructed with either limestone or siliceous river gravel coarse aggregates. Although this project will involve other aggregates in its later phases, present work involves only these two aggregates. The primary objective of Phase 1 of this project is to provide a comparison of the two aggregates in terms of their respective design algorithms.

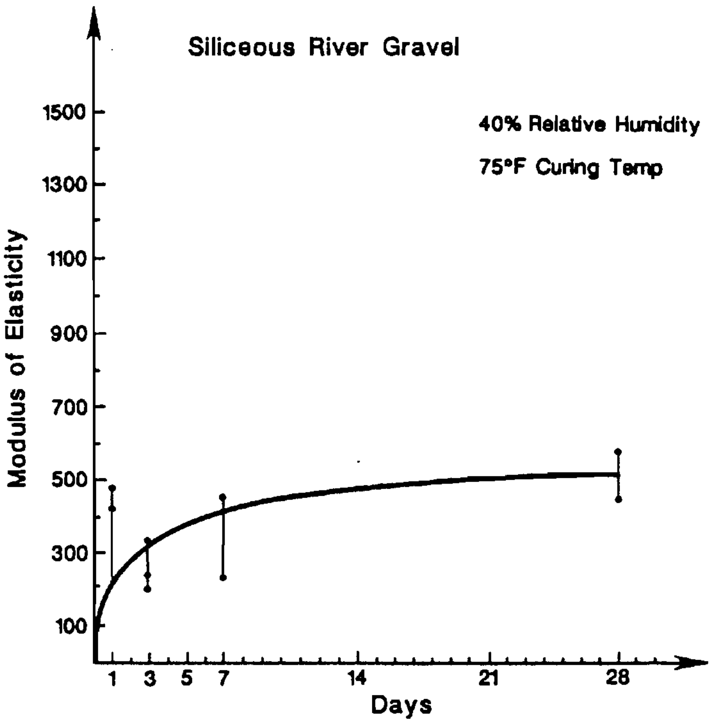
Up to the current stage there were two major aspects of the project: first, the determination of concrete properties in the laboratory using limestone and siliceous river gravel aggregates, and, second, an analysis of pavement performance predictions provided by the CRCP–4 computer program, based on the concrete property inputs for the two aggregates. The CRCP–4 computer program developed at The University fo Texas at Austin served as the primary analysis tool. The program's capabilities include the prediction of a time history for crack spacing, crack width, and steel stress for a range of concrete properties, environmental conditions, and pavement structure geometry. Concrete properties of drying shrinkage strength and stiffness are allowed to vary with time. These are important factors since they are key factors affecting performance and consequently the design of continuously reinforced concrete pavements. This capability of the CRCP–4 computer program was the reason for its being the basis of the analysis procedure. Major goals of this project may be summarized as follows:

(1) Create an understanding regarding variations in concrete properties with the use of different coarse aggregate types.
(2) Develop an analysis procedure for comparison of aggregate types based on their concrete properties. This procedure would be a performance based analysis utilizing the CRCP–4 computer program.
(3) Develop performance based predictive models and design charts leading to design details and guidelines for specifications to be used with continuously reinforced concrete pavement design and construction in Texas.

SCOPE OF THE STUDY

A major aspect of the work done under Phase 1 of Project 422 was the determination of concrete properties using limestone and siliceous river gravel aggregates. These properties were determined in the Ferguson Structural Engineering Laboratory at the Balcones Research Center, The University of Texas at Austin. Specimens used in testing were composed of aggregates similar to those being currently used in field designs. Once the laboratory data had been compiled, concrete properties models were developed on the basis of a statistical analysis. These models served as input to the CRCP–4 computer program. An analysis of performance predictions from the CRCP–4 program provided a comparison and development of performance based design models for limestone and siliceous river gravel aggregates.

This report is the second in a series of reports recording work accomplishments of Project 422. The primary objective of Report 422-1, "Coarse Aggregates for PCC Pavements—Pilot Study Evaluation," was to report a comparison of limestone and siliceous river gravel on the basis of their engineering properties. The stage at which Research Report 422-1 was written involved primarily the preparation of input models for CRCP–4. Two different methodologies were adopted in the initial period of laboratory data analysis. As an interim approach, until the availability of 90–day test results, concrete property inputs were generated by visually plotting a smooth curve through the laboratory observations. An example of such a curve is illustrated in Fig 1.1. The curves drawn by this method were made to pass through the observed variation in each value. This variation occurred



Summary of Selected Values

Day	1	3	5	7	14	21	28
Modulus of Elasticity	200	325	375	415	480	505	520

Fig 1.1. *A characteristic curve used in the preliminary analysis to determine CRCP–4 input.*

because three measurements were recorded for each combination in the testing factorial (Chapter 2). Specific values from these plotted curves were back calculated and input into the CRCP–4 computer program. This process allowed the computer program to utilize a smooth concrete property versus time relationship for its analysis. A more rational approach was later adopted by developing regression models for concrete properties. These regression models provided the basis for developing design algorithms to be introduced in this report.

A comparison of aggregate type cannot be made only on the basis of concrete properties. Various concrete characteristics affect CRC pavement design in diverse ways. A gain in concrete strength may result in higher tensile strength and modulus of elasticity. However, while higher tensile strength requires an increase in steel reinforcement, higher modulus may necessitate lowering the steel percentage (Ref 2).

This report introduces models for design of steel reinforcement for CRC pavements based upon aggregate type. Steel reinforcement requirements for continuously reinforced pavements are determined by several variables, including concrete properties. Thus, the requirements of steel for pavement design can serve as a good basis of comparison for two aggregates in terms of construction cost. An aggregate resulting in a lower steel percentage could be considered a better aggregate in terms of initial cost. The ultimate comparison, however, must involve pavement life cycle costs related in terms of performance and reliability.

CHAPTER 2. THE ANALYSIS PROGRAM

INTRODUCTION

The CRCP-4 computer program, developed in CTR Research Report 177-9 (Ref 3), provides a complete analysis and design procedure for continuous pavements. This computer program predicts time history of crack spacing, crack width, and steel stress for a range of concrete properties, environmental conditions, and pavement structure geometry. Crack spacing, crack width, and steel stress provide the limiting criteria for the design of CRC pavements. It was for this purpose that this program was selected as the primary analytical tool for this project. The flow chart shown in Fig 2.1 outlines the various activities leading to the development of design algorithms for limestone and siliceous river gravel aggregates. As indicated in the chart, there were two major aspects of the selection and development of input parameters for the CRCP-4 computer program. The first aspect was the determination of concrete properties. Secondly, for the program to provide realistic results, reasonable values of other parameters were required. These included temperature values to model the environment, steel reinforcement properties, external load characteristics, slab subbase friction relationship, and soil support conditions. The purpose of this chapter is to describe the CRCP–4 computer program, its required inputs, and the analysis factorial design for this phase of the project. There is one exception to the discussion of input parameters in this chapter. Since the concrete properties require extensive detail, they are described in Chapter 3 of this report.

THE CRCP–4 COMPUTER PROGRAM

It is not necessary for the reader to be involved with the intricate details of the CRCP–4 computer program. However, it is important to understand the basic concept of the program in order to appreciate the design models introduced in later chapters. For the purpose of analysis, the program utilizes a typical slab segment to represent the pavement system. This segment is based on the behavior of continuous pavement and its response to external and internal stresses. A summary of the basic procedure utilized by the program has been adopted from Research Report 177-2 (Ref 4). Although the actual model includes wheel load modelling as a variable, for the sake of simplicity it has not been included in the following stepwise description.

(1) At any time t_1, the program determines the tensile strength of concrete from a strength time relationship [Fig 2.2(a)].

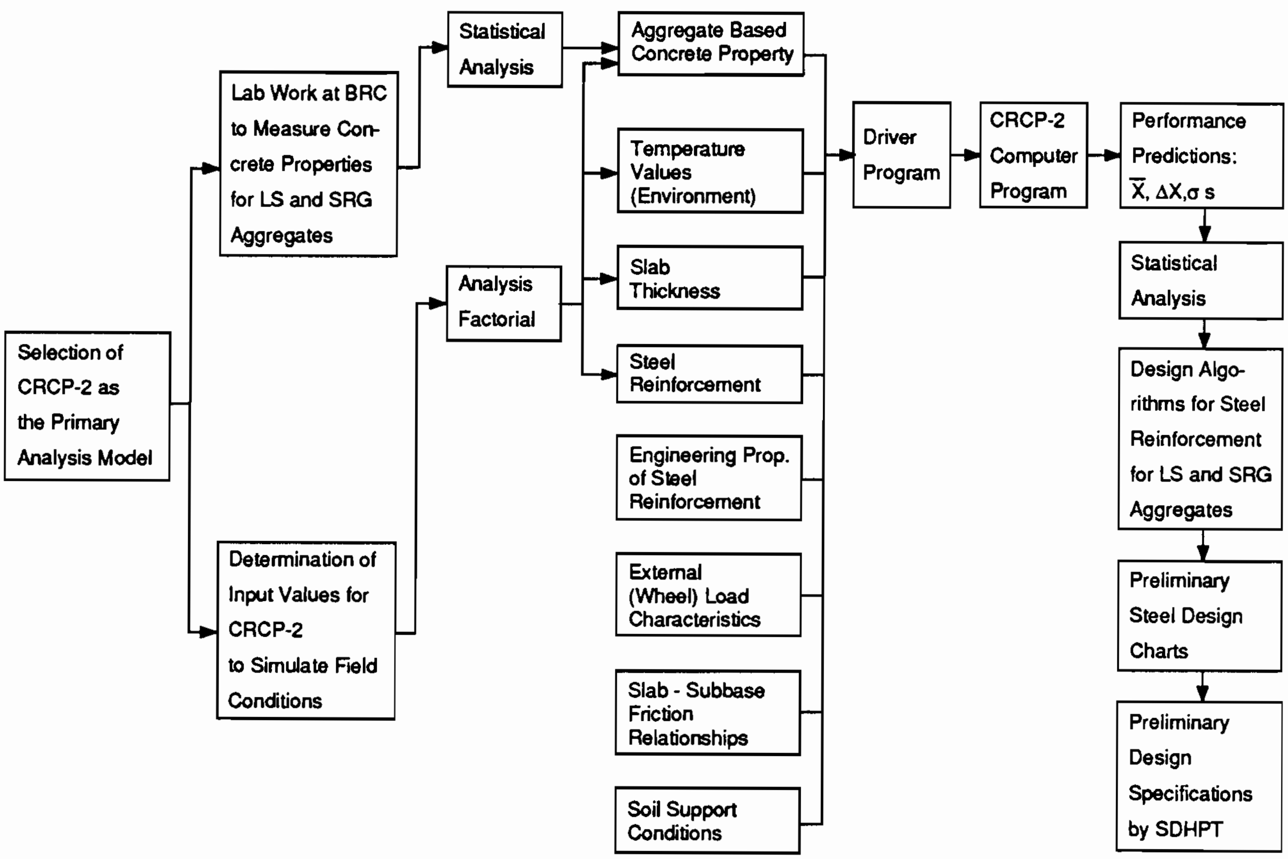


Fig 2.1. *Activity chart for Phase 1 of Project 422.*

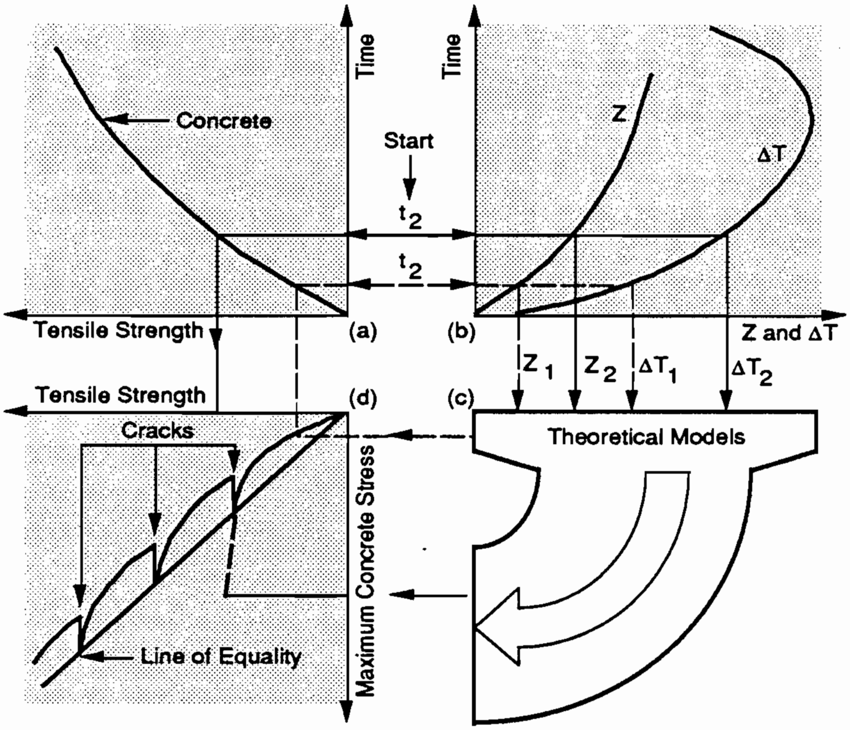


Fig 2.2. ***Simplified approach as applied to the continuous pavement system by the CRCP-4 program.***

(2) It then computes drying shrinkage z_1 and temperature drop ΔT_1 corresponding to time t_1 [Fig 2.2(b)].

(3) With mathematical models, it calculates the maximum concrete tensile stress [Fig 2.2(c)].

(4) It compares the concrete strength with the concrete stress [Fig 2.2(d)]. If the strength is higher than the stress, then no cracking occurs.

(5) It then increments the time to t_2 and repeats Steps 1 through 4. If the stress is higher than the strength, as shown in Fig 2.2(d), a crack occurs between t_1 and t_2.

(6) It solves for the time (somewhere between t_1 and t_2) and the corresponding state of stress at which the cracking occurred.

(7) It increments time and searches for additional cracks as they develop.

The concrete strength and shrinkage models used by the computer program for this analysis were provided by the inputs developed in the laboratory for this project. By virtue of this process the computer model was simulating those pavements built by using aggregates tested in the laboratory. The choice of aggregates tested in the laboratory, in turn, was made so as to sample those being currently used in the field. Furthermore, all inputs, such as temperature and pavement structure properties, were developed to provide the program a representative model of the conditions in the state of Texas.

INPUT PARAMETERS FOR CRCP-4 PROGRAM

The flow chart in Fig 2.1 illustrates the input requirements for the CRCP-4 computer program. An important aspect of the input was concrete properties, which will be discussed in the following chapter. As for the other inputs, they were classified into two categories from the point of view of the analysis. These were (1) inputs which served as variables in the analysis program and (2) inputs which were input one time to provide the computer model a reasonable basis for determining performance predictions. Temperature variations due to different environmental conditions, pavement thicknesses, aggregate based concrete properties, and steel reinforcement were variables in the analysis factorial. However, the other four inputs, engineering properties of the steel reinforcement, wheel load characteristics, slab–subbase friction relationships, and soil support conditions, were constant values for the analysis. These constant values are significant due to the fact that the design models to be introduced later in this report are based on these values.

VARIABLE INPUTS

As shown in Fig 2.1, there were four variable inputs formulating the analysis factorial. These were steel reinforcement, slab thickness, environmental conditions, and aggregate based concrete properties. Discussion of concrete properties is provided in the next chapter. As for steel reinforcement and slab thickness, their input simply required the selection of values that would bracket the field designs. These values are described in the following section.

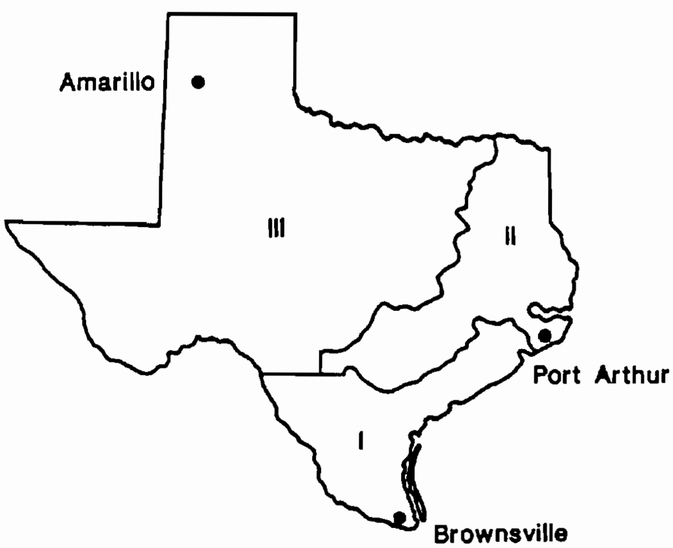
Extensive work was done on the selection and preparation of temperature values to model the environment. Details of this process have been discussed in Research Report 422-1 (Ref 6). A brief discussion of this aspect is provided in this report.

Design temperature drop, along with effects of shrinkage, is the contributing factor that causes the pavement slab to move. Restraint from this movement is the primary cause of stresses in the pavement. The CRCP-4 computer program requires specific temperature values to model environmental effects on the pavement. Input requirements include curing temperature, minimum temperature expected after the concrete gains full strength, number of days after the concrete is set, and minimum daily temperature.

This study has required the formulation of design models applicable for the whole state of Texas. In terms of environmental inputs this has meant simulating a tremendous variation in climatological conditions. CTR Research Report 249-6, "Design Charts for the Design of ACHM Overlays on PCC Pavements Against Reflection Cracking" (Ref 5), had defined the prevailing climatological regions in the state of Texas. That report provided the basis for dividing the state into three representative regions, which are illustrated in Fig 2.3. The three locations chosen to determine the minimum daily temperatures were Brownsville (Zone I), Port Arthur (Zone II), and Amarillo (Zone III). *Local Climatological Data Summary, 1984 Monthly Summary*, compiled by the National Oceanic and Atmospheric Administration, was used to determine temperature values for each geographical location.

The specific time in the year at which concrete is placed determines the placement temperature, as well as the length of time after which the maximum temperature drop is going to occur. Since the CRCP–4 program utilizes time history models of concrete strength parameters, it requires such information. The calendar year was, accordingly, divided into four seasons. Table 2.1 illustrates the division of the four seasons and lists the number of days prior to the maximum temperature drop for that season.

CLIMATOLOGICAL REGIONS



ZONES	COMBINED DISTRICTS	SITES
I	Gulf Coast / Lower Valley	Brownsville
II	East Texas - South Central	Port Arthur
III	North and West Texas	Amarillo

Fig 2.3. *Climatological district assignment (Ref 6).*

TABLE 2.1. ***NUMBER OF DAYS BEFORE MAXIMUM TEMPERATURE DROP FOR EACH SEASON (REF 6)***

Season	Months	Number of Days Before Maximum Temperature Drop
Winter	Dec/Jan/Feb	360
Spring	Mar/Apr/May	270
Summer	Jun/Jul/Aug	180
Fall	Sep/Oct/Nov	90

TABLE 2.2. ***REVISED SEASONAL DAILY TEMPERATURE DROP VALUES (REF 6)***

	Season			
Location	Winter	Spring	Summer	Fall
Brownsville	41	36	23	32
Port Arthur	34	30	23	28
Amarillo	47	45	41	39

The minimum daily temperature drop for each day of the season was calculated by considering the difference between the high and low temperatures for that particular day. The largest differential of all of the days for that particular season determined the seasonal minimum daily temperature drop. A summary of the revised values used in this analysis is provided in Table 2.2.

OTHER INPUT CRITERIA

This section describes the values chosen for the constant inputs shown in the flow chart (Fig 2.1). The subsequent sections provide a discussion explaining the basis of selecting these input values.

Engineering Properties for Steel Reinforcement

ASTM Grade 60 steel was considered for this analysis; accordingly, a 60–ksi value was used as the steel yield stress, and 29,000 ksi was input as the steel elastic modulus. Based on the recommended value in the AASHTO Guide (1986) (Ref 7), a thermal coefficient value of 5.0×10^{-6} in./in./°F was used for this study.

External Load Characteristics

The CRCP–4 computer program has the capacity to analyze the effects of a wheel load based on its time of application since the concrete placement. A wheel load of 9,000 pounds (for an 18–kip single axle) and a duration of 14 days were provided as program inputs. This meant that the computer program would apply the appropriate wheel load at a concrete age of 14 days. Two weeks used to be the minimum time in Texas before traffic is allowed on a concrete pavement.

Subbase Friction Relationship

Since the primary purpose of this report is to provide a comparison between the use of limestone and siliceous river gravel aggregates, a constant subbase friction relationship was assumed for this analysis. Figure 2.4 illustrates the relationship used for this analysis. This relationship was first reported in "Report on a Mechanistic Analysis at King Fahd International Airport, Kingdom of Saudi Arabia" (Ref 8).

Soil Support Conditions

Again, since the objective of this project was to provide a comparison between the two aggregate types, a constant value was assumed for the soil support condition. The value of k (soil support constant) chosen for this analysis was 300 pci.

ANALYSIS FACTORIAL

A factorial for analysis with the CRCP–4 program was formulated on the basis of the variable inputs. As shown in Fig 2.1 there were four categories of these variables. These categories may be further broken up in the following manner:

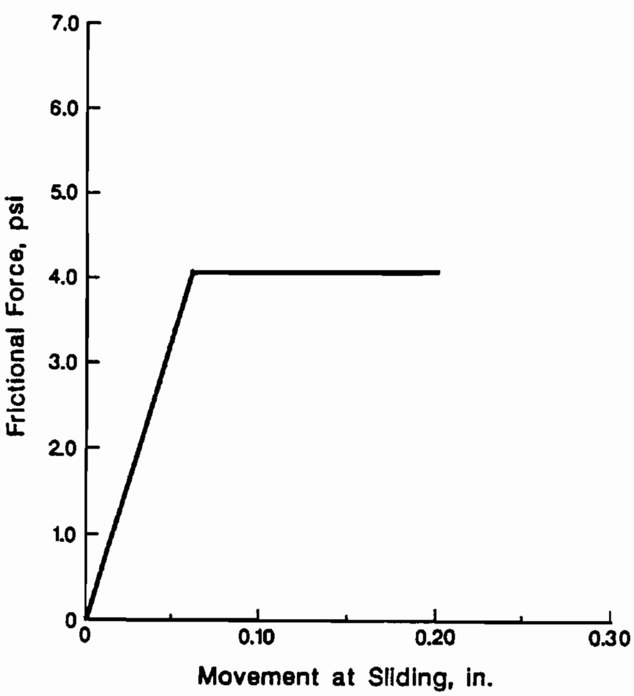


Fig 2.4. ***Slab subbase friction relationship.***

Aggregate	Pvmt Depth	% Steel 0.4%				0.5%				0.6%				0.7%			
	Bar #	4	5	6	7	4	5	6	7	4	5	6	7	4	5	6	7
SILICEOUS RIVER GRAVEL	8																
	9																
	10																
	11																
	12																
	13																
	14																
	15																
CRUSHED LIMESTONE	8																
	9																
	10																
	11																
	12																
	13																
	14																
	15																

Fig 2.5. ***Model analysis factorial for one season at any one location.***

AGGREGATE TYPE (Concrete properties)
- 2 types (LS and SRG)

ENVIRONMENT (Temperature values)
- 4 placement seasons (winter, summer, autumn, and spring)
- 3 geographic locations (Brownsville, Port Arthur, and Amarillo)

STEEL REINFORCEMENT
- 4 reinforcement ratios (0.4, 0.5, 0.6 and 0.7 percent steel)
- 4 bar diameters (nos. 4, 5, 6, and 7)

PAVEMENT THICKNESS
- 8 thicknesses (8 to 15 inches)

A sample factorial is shown in Fig 2.5. This factorial is for one placement season at any particular location. Therefore, for four seasons at three locations there were 12 such factorials. Since each cell in the factorial requires a computer run, 3,072 runs were necessary for the complete analysis procedure. The CRCP–4 program requires an elaborate data file for each run; to execute so many runs posed the increased possibility of error and omissions. To circumvent this aspect, a computer program was formulated to execute the CRCP–4 computer program in accordance with the analysis factorial. As a result of this procedure massive amounts of output data were generated. This output data provided pavement performance predictions by the CRCP–4 model based upon the variables considered in the analysis. Further statistical analysis of this data provided the steel design algorithms introduced in Chapter 4 of this report.

CHAPTER 3. PREDICTIVE MODELS FOR CONCRETE PROPERTIES

INTRODUCTION

Input requirements for the CRCP–4 computer program were discussed in Chapter 2. This program requires specific inputs of concrete properties to develop pavement performance predictions. These properties for two types of concrete mixes, comprised of limestone and siliceous river gravel aggregates, were determined in the Ferguson Structural Engineering Laboratory at the Balcones Research Center, The University of Texas at Austin. Aggregate sources, mixing proportions, and testing procedures were determined by the CTR staff with approval from the Highway Design Division and the Materials and Test Division of the SDHPT.

The laboratory testing included measurements of concrete elastic modulus, flexural strength, drying shrinkage, and coefficient of thermal expansion. A factorial developed for the purpose of laboratory testing is shown in Fig 3.1. Concrete specimens were cured at relative humidities of 40 and 100 percent and curing temperatures of 50, 75, and 100°F. All specimens were tested at five different concrete ages: 1, 3, 7, 28, and 90 days. Furthermore, three specimens were tested for each cell shown in the factorial. This was done so that the statistical analysis of the data could be performed and variability associated with the tests could be estimated.

Initial work done for the preparation of concrete property inputs for the CRCP–4 computer program was reported in CTR Research Report 422-1 (Ref 6). The laboratory data were plotted manually to generate the concrete property curves. These curves were generated by visually determining the best fit to the plotted data. A typical plot of the laboratory test data is shown in Fig 1.1. As mentioned earlier, three values were measured for each cell of the factorial. The dispersion of these values defines the statistical variation of the test data.

		MOISTURE CONDITIONS					
		40% REL HUM.			100% REL HUM.		
		CURING TEMPERATURES					
COARSE AGGREGATE TYPES	CURING TIMES	50°F	75°F	100°F	50°F	75°F	100°F
Siliceous River Gravel	1						
	3						
	7						
	28						
	90						
Limestone	1						
	3						
	7						
	28						
	90						

Fig 3.1. ***Factorial for laboratory testing.***

This preliminary procedure served until the availability of the 90-day data. There were many problems with this approach. First, several visual interpretations could be made from the data set, and as a result the existence of any trends in the data could not be determined. Second, this approach could not provide an interpolation of the data obtained in the laboratory. When the laboratory testing factorial was designed its main purpose was to bracket a wide range of variables and values rather than to test each conceivable combination. For example, in the case of relative humidity, laboratory testing was performed at 40 and 100 percent humidities. Neither value was considered to model the field conditions. For the purpose of design a value of 75 percent was considered appropriate for simulating field conditions.

The development of mathematical models for concrete properties allowed flexibility to model any intermediate field condition and analyze the data with respect to the desired variables. Concrete property predictive models were developed by a computer based regression analysis of the data. This chapter describes and analyzes the models in subsequent sections.

PREDICTIVE EQUATIONS OF CONCRETE PROPERTIES

Predictive models were developed for concrete elastic modulus, tensile strength, drying shrinkage, and thermal coefficient. Although laboratory measurements were also taken for concrete flexural strength, a model was not developed for this property. This was due to the fact that flexural strength is not a required input for the CRCP–4 computer program. Determination of flexural strength was required to provide a quality control check, because this property is used in the specifications of the Texas State Department of Highways and Public Transportation.

Concrete property predictive models were obtained by a multiple regression analysis of laboratory data for all samples. Laboratory data were analyzed to determine the effects of the following variables on the properties of concrete mixes:

(1) test age (days),
(2) curing temperature (°F),
(3) relative humidity at which specimens were cured (percent), and
(4) coarse aggregates used in the mix (SRG or LS).

The subsequent sections describe the measurement procedures for determining concrete properties and analyze the significance of their models.

Elastic Modulus of Concrete

Concrete modulus of elasticity values were measured from specimens tested in third point loading according to

ASTM C-78 test method (Ref 9). Modulus data values were determined using the slope of the chord connecting the 20 and 50 percent ultimate stress values from the stress–deflection curve. A regression analysis of this data provided the following model:

$$E = \left(e^{(5.260 + 0.104x)}\right)\left(t^{0.097}\right)\left(H^{0.152}\right) \qquad (3.1)$$

where

- E = modulus of elasticity (x 10^4 psi);
- t = concrete curing age, in days;
- H = relative humidity, in percent; and
- x = aggregate type identifier:
  - $x = 0$ for siliceous river gravel aggregate and
  - $x = 1$ for limestone aggregate.

This analysis showed that the effects of relative humidity and curing time were statistically significant at a 95 percent confidence level. The effect of coarse aggregate type was significant at a confidence level of 92 percent. Although curing temperature was a variable in this analysis, results indicated that its inclusion did not influence the model. Plots illustrating the modulus values for limestone and siliceous river gravel aggregate mixes are shown in Figs 3.2 and 3.3, respectively. For both mixes the moduli increased with age and higher relative humidity conditions. This is consistent with previous experience (Ref 10).

These laboratory results showed a higher modulus value for limestone concrete mixes. This is contrary to the expectations prior to laboratory testing. A comparison of elastic modulus values with those recommended in CTR Research Report 177-22F (Ref 11) is provided in Table 3.1.

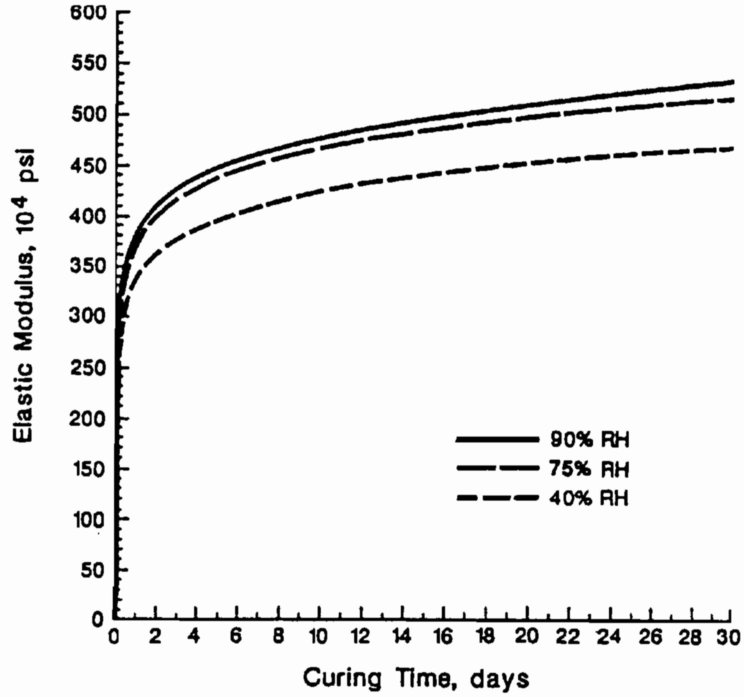


Fig 3.3. *Concrete elastic modulus relationship for siliceous river gravel aggregates.*

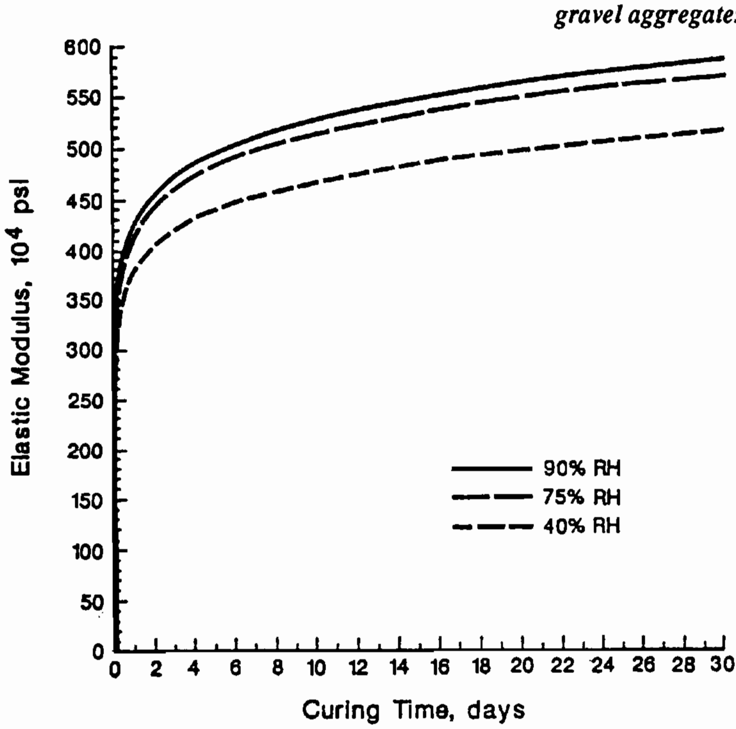


Fig 3.2. *Concrete elastic modulus relationship for limestone aggregate.*

These values indicate a higher modulus value for siliceous river gravel mixes. The causes of this difference are being investigated by the researchers and the results will be reported in the next project report.

TABLE 3.1. ***A COMPARISON OF CONCRETE ELASTIC MODULUS VALUES FOR THIS PROJECT WITH PREVIOUS RECOMMENDATIONS***

Aggregate Type	Previously Recommended Values (psi)	Project 422 Values (psi)
Limestone	4.5×10^6	5.7×10^6 *
Siliceous River Gravel	6.0×10^6	5.1×10^6 *

* Values determined for 28-day concrete strength at 75 percent relative humidity.

Tensile Strength

The tensile strength of concrete was determined by using a split cylinder test following ASTM C-496 test method (Ref 9). The predictive model developed for tensile strength input to the CRCP-4 computer program is provided in Eq 3.2:

$$f_t = (e^{4.74 + 0.0642x})(t^{0.0926})(T^{0.180})(H^{0.0301}) \tag{3.2}$$

where

f_t = tensile strength (psi);
t = concrete age in days;
T = curing temperature (°F);
H = humidity, in percent; and
x = aggregate identifier:
$x = 0$ for siliceous river gravel and
$x = 1$ for limestone.

The results of this analysis indicated that aggregate type, curing temperature, and curing age were significant at a 95 percent confidence level. Plots illustrating the effects of curing temperature for limestone and siliceous river gravel aggregate mixes are shown in Figs 3.4 and 3.5, respectively. As expected, an increase in concrete tensile strength is noted with increased curing temperature (Ref 10).

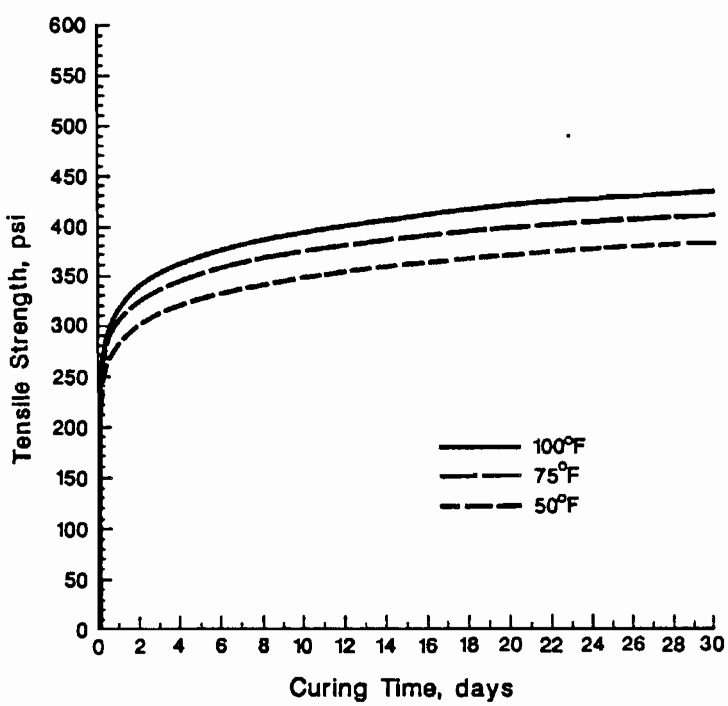


Fig 3.4. *Concrete tensile strength relationship for limestone aggregates (75°F curing temperature).*

Although relative humidity was included as a variable in this analysis, its influence on tensile strength is minimal. The effect of humidity on concrete tensile strength for limestone aggregate mixes is illustrated in Fig 3.6. A slight increase in tensile strength is noted with increasing relative humidity. This improvement in tensile strength with higher relative

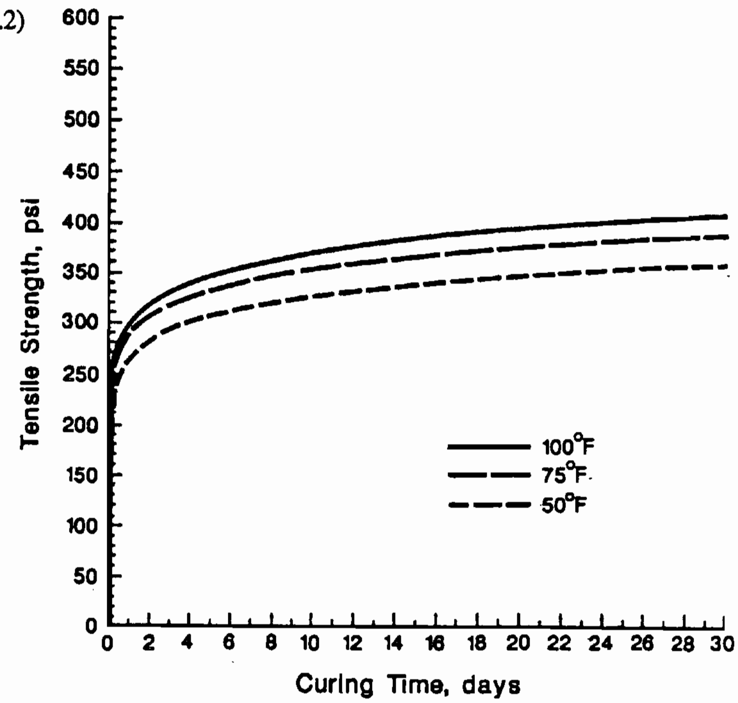


Fig 3.5. *Concrete tensile strength relationship for siliceous river gravel aggregates (75°F curing temperature).*

humidity is again in accordance with expectations (Ref 10). As in the case of elastic modulus, tensile strengths for concrete mixes comprising each aggregate showed an early strength gain. This aspect may be attributed to the fineness of the Texas cements. Ratio of hydration and the degree to which particles are hydrated improves with increasing fineness of cement (Ref 10).

Concrete Shrinkage

Drying shrinkage of concrete was measured using a modified version of the ASTM C-157 test method (Ref 9). Measurements for this property were taken only at 40 percent relative humidity. It was assumed that no shrinkage would take place at 100 percent relative humidity. As a result of this limitation in the testing factorial, humidity could not be set as a variable for the drying shrinkage model. Equation 3.3 describes the model formulated for drying shrinkage:

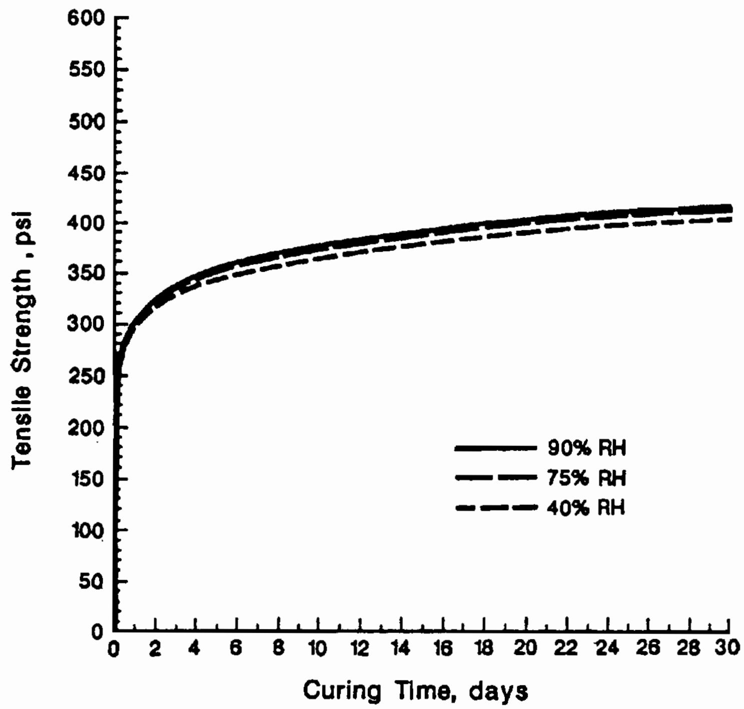


Fig 3.6. ***Effect of humidity on concrete tensile strength of limestone aggregate (75°F curing temperature).***

$$Zt = \left(e^{(0.0422 - (8.71/t) - 0.0919x)}\right)\left(T^{1.35}\right) \qquad (3.3)$$

where

- z_t = coefficient of drying shrinkage x 10^{-6} in./in.,
- t = concrete age in days,
- T = curing temperature (°F), and
- x = aggregate type identifier:
  - x = 0 for siliceous river gravel aggregate and
  - x = 1 for limestone aggregate.

Curing time and curing temperature were observed to be significant at a 95 percent confidence level. Relative to concrete strength models, coarse aggregate type was not as significant in this model. Plots relating the effects of curing temperature for limestone and siliceous river gravel concretes are shown in Figs 3.7 and 3.8, respectively. Increase in curing temperature significantly increases concrete shrinkage for both aggregates. Both aggregates show virtually the same amount of shrinkage under any set of given conditions. This relationship determines the maximum shrinkage for any condition for either aggregate at approximately 2.50 x 10^{-6} in./in./°F. This value is less than those recommended in the AASHTO Guide (1986) (Ref 7), where a value of 3.0 x 10^{-6} in./in./°F is assigned for siliceous river gravel and 5.0 x 10^{-6} in./in./°F is considered characteristic of limestone aggregate.

Thermal Coefficient of Concrete

Thermal coefficient of concrete was measured by placing two strain gages on each specimen. Measurements were recorded in 30°F intervals over a curing temperature range from 45°F to 135°F. Regression analysis of laboratory data for thermal coefficient indicated a constant value during the curing periods of one through 90 days. Values used for CRCP–2 analysis are indicated in Table 3.2 together with values recommended by the AASHTO Guide (1986) (Ref 7).

ANALYSIS OF RESULTS

Differences in concrete properties due to aggregate type had initially been identified in CTR Research Report 177-22F, "Summary and Recommendations for the Implementation of Rigid Pavement Design, Construction and Rehabilitation Techniques" (Ref 11). Results from this project indicate that the effect of aggregate type is quite significant in the determination of concrete properties. This is demonstrated by the signifi-

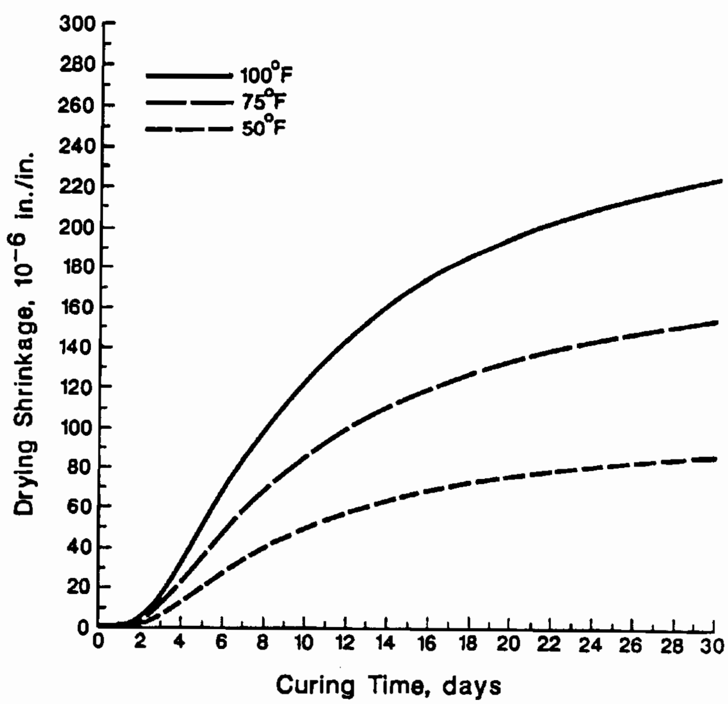


Fig 3.7. ***Relationships for concrete shrinkage for limestone aggregate.***

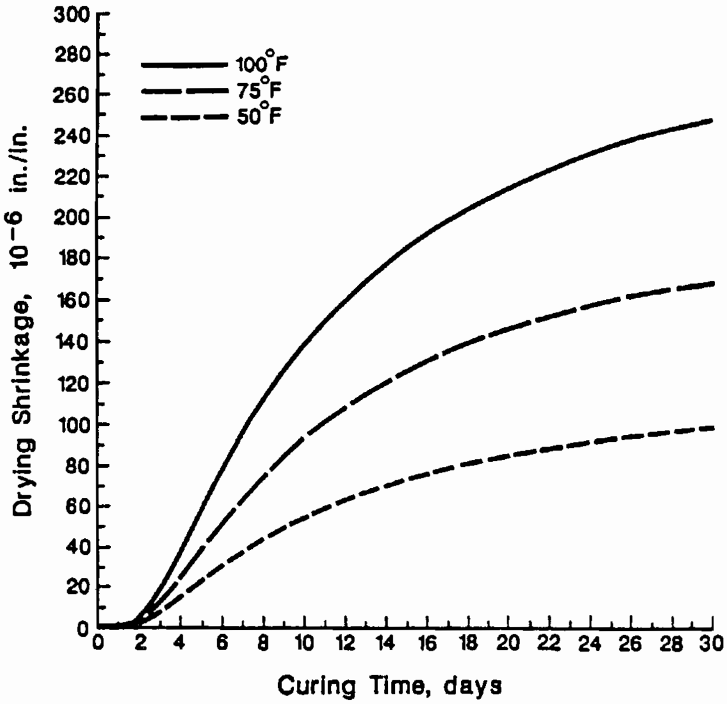


Fig 3.8. ***Relationships for concrete shrinkage for siliceous river gravel aggregate.***

TABLE 3.2. ***A COMPARISON OF CONCRETE THERMAL COEFFICIENT VALUES***

Aggregate Type	AASHTO Recommendations (in./in./°F)	Project 422 (in./in./°F)
Limestone	3.8×10^{-6}	6.0×10^{-6}
Siliceous River Gravel	6.0×10^{-6}	8.0×10^{-6}

cance of the aggregate identifier in the concrete property predictive models for elastic modulus, tensile strength, and thermal coefficient developed for this project. Several comparisons were made between the concrete properties determined for this project and the standard values recommended in the past. When such a comparison is made it is important to note that this study analyzing the effects of aggregate type on pavement concrete is the very first of its kind. While previous reports, such as CTR Research Report 177-22F, have recognized this aspect of concrete behavior, their recommendations for concrete properties were not based on laboratory testing done to the scale of this project. It is the objective of this project to develop recommendations for concrete composed of different aggregates.

Work is currently being pursued to determine the variability associated with the values measured in the laboratory. It is important to note that a significant amount of scatter was observed in the laboratory data for elastic modulus and shrinkage of concrete. While these concrete property predictive models have been developed and used for the formulation of design algorithms, their significance will be better defined as the results of the variability analysis become apparent.

CHAPTER 4. DESIGN ALGORITHMS

INTRODUCTION

A meaningful comparison of limestone and siliceous river gravel aggregates can be considered on the basis of performance based design algorithms. The development of concrete predictive models and other design inputs initiated the formulation of such design algorithms based on the performance predictions by the CRCP–4 program. These models are equations for the design of longitudinal steel reinforcement for CRC pavements.

Design of CRC pavements is based upon the premise that concrete volume changes are accounted for by the occurrence of transverse cracks in the pavement. Concrete volume changes primarily occur as a result of shrinkage and temperature variations. Restraint of the concrete slab due to subbase friction and steel reinforcement causes the concrete to fracture. A balance between the properties of concrete and steel reinforcement must be achieved for the pavement to behave in a satisfactory manner. It is notable that longitudinal reinforcement is provided not to prevent cracking from occurring but to provide control over crack width and crack spacing of the pavement.

This chapter describes the development of models for the design of steel reinforcement for pavements to be constructed with limestone and siliceous river gravel aggregate types. Significant differences have been observed in the properties of concrete mixes composed of the two aggregate types. It is expected that these variations in concrete properties, attributed to the use of either aggregate type, can be translated in terms of different steel reinforcement requirement for each aggregate type.

LIMITING CRITERIA

Prior to the introduction of the design equations it is important to provide some understanding of the limiting criteria which control the design of longitudinal reinforcement. Level of steel reinforcement for a CRC pavement is determined by acceptable limits of crack spacing, crack width, and steel stress. The limit of acceptance on these criteria is based upon minimizing the distress manifestations for continuous pavements (Ref 12).

Crack Spacing

Limits on crack spacing requirements are based upon considerations of spalling and punchouts. When the crack spacing has been allowed to exceed 8.0 feet, an increase in the probability of spalling has been noted. It is also recommended that crack spacing is greater than 3.5 feet. This is based on the consideration of extremely small slab lengths, which induce punchouts. Thus, the crack spacing criteria have a maximum limit of 8.0 feet and a minimum of 3.5 feet (Ref 12).

Crack Width

The magnitude of acceptable crack width is determined by concerns for water infiltration and spalling. Water infiltration is controlled by limits on the pavement crack width, which is related to the permanent deformation of the reinforcement steel. This control is provided by the design criteria for steel stress.

The other concern is spalling. In general, spalling is attributed to environmental and vehicular loading stresses. A correlation of crack width as a function of design temperature drop was introduced in CTR Report 177–22F (Ref 11). This relationship is illustrated in Fig 4.1. On the basis of an approximate design temperature of 75°F, a crack width of 0.047 inches was chosen for of this analysis.

Steel Stress

Permanent deformation and steel fracture are the primary concerns in this aspect of the limiting criteria. Previously, a value of 3/4 the ultimate steel tensile strength was defined as a design control. However, past experience has shown that CRC pavement performance is not significantly affected if the steel yield point is exceeded. Based on this consideration allowable stress for Grade 60 steel is recommended to be between 54 and 67 ksi based upon the indirect tensile strength of the concrete and the rebar size for the reinforcement (Ref 11). A constant value of 60 ksi was used for this analysis.

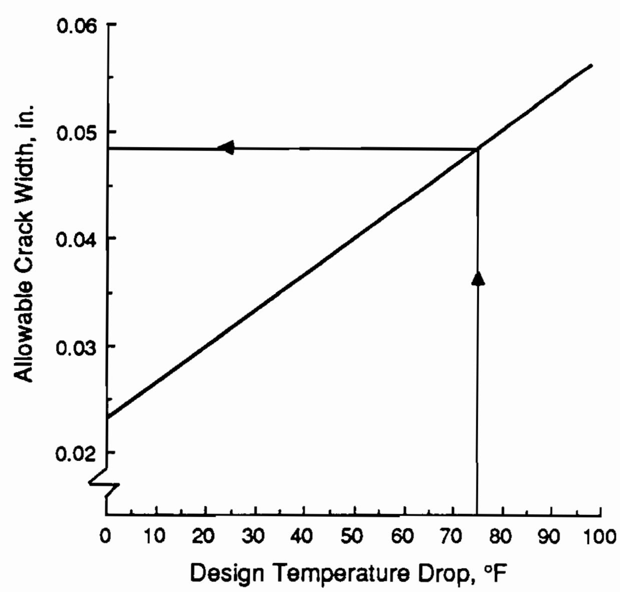


Fig 4.1. ***Limiting crack width for design temperature drop.***

DESIGN MODELS

Before introducing the design equations the prime objective of developing these models must be emphasized. These design models were formulated for comparing the use of limestone and siliceous river gravel aggregate based upon the requirement for steel reinforcement. The analysis factorial introduced in Chapter 2 encompassed a wide range of design variables. As a result of the CRCP-4 computer program runs, a massive amount of data was available relating the three limiting criteria to the following variables:

(1) pavement thickness,
(2) environmental conditions (geographical locations and placement season),
(3) steel reinforcement properties (reinforcement ratios and bar diameter), and
(4) aggregate type (dependent upon their concrete properties).

In order to develop any meaningful conclusion from these data, a relationship had to be formulated relating the limiting criteria for the design of steel reinforcement in terms of the variables mentioned above. A regression analysis was performed on the CRCP-4 program output in order to develop equations in the following format:

Limiting criteria = f {environmental conditions, aggregate type, pavement thickness, percentage steel}

Two different packages were used for regressing the performance prediction output from the CRCP-4. These were the MINITAB and SAS packages. The MINITAB program was used for preliminary analysis followed by a class regression utilizing the SAS package. In this analysis pavement thickness and steel percentage were continuous variables, while bar diameter, geographic locations, placement seasons, and aggregate type were considered as discrete levels. This classification procedure provided the model a slightly better fit on the CRCP-4 output data. Since both procedures produced similar results, only the class regression models are presented in the following sections.

Crack Spacing

The following log model was developed for the crack spacing criteria

$$\ln CS = -2.31 + S + A + B + 1.304 \ln D - 1.97 \ln PS \quad (R^2 = 0.963) \tag{4.1}$$

where

CS = crack spacing (feet),
S = coefficient for season (see Table 4.1),
A = coefficient for aggregate type (see Table 4.1),
B = coefficient for bar number (see Table 4.1),
D = slab thickness (inches), and
PS = percent steel reinforcement.

Crack Width

Equation 4.2 describes the model developed for the crack width criteria:

$$\ln CW = -2.59 + S + A + B + 1.23 \ln D + 1.94 \ln PS \tag{4.2}$$

$$(R^2 = 0.920)$$

where

CW = crack width x 10^{-2} inch.

Other variable notation is similar to Eq 4.1. Coefficient values for the equation may be obtained from Table 4.2.

Steel Stress

The model developed for steel stress is represented by the relationship

$$\ln SS = -0.688 + S + A + B + 0.731 \ln D - 1.12 \ln PS \tag{4.3}$$

$$(R^2 = 0.916)$$

where

SS = Steel Stress x 10^{-1} ksi.

Again, the remaining notation is identical to Eq 4.1. Coefficient values for this equation are listed in Table 4.3.

ANALYSIS OF DESIGN MODELS

The equations presented in the earlier section appear to predict reasonable values. However, an analysis must be made of the model in terms of the involved variables. Theoretical relationships were developed at CTR between the design parameters and the relevant input variables. The summarized form of these relationships, adopted from CTR Research Report 177-16 (Ref 2), is as follows:

$$CS \propto \frac{(f_t)^{a_1} (\phi)^{a_2} (\alpha_s)^{a_3}}{(P)^{a_4} (\sigma_w)^{a_5}}$$

$$CW \propto \frac{(f_t)^{b_1} (\phi)^{b_2}}{(P)^{b_3} (\sigma_w)^{b_4}}$$

where

CS = crack spacing (feet),
CW = crack width (inches),

TABLE 4.1. *COEFFICIENTS FOR USE IN EQUATION 4.1*

Season	Coefficient	Aggregate Type	Coefficient	Bar Number	Coefficient
Winter	0.195	Limestone	0.000	4	-0.779
Spring	0.153	Siliceous		5	-0.541
Summer	0.035	River Gravel	-0.385	6	-0.283
Fall	0.000			7	0.000

TABLE 4.2. *COEFFICIENTS FOR USE IN EQUATION 4.2*

Season	Coefficient	Aggregate Type	Coefficient	Bar Number	Coefficient
Winter	-0.079	Limestone	0.000	4	-0.774
Spring	0.131	Siliceous		5	-0.537
Summer	0.181	River Gravel	-0.137	6	-0.283
Fall	0.000			7	0.000

TABLE 4.3. *COEFFICIENTS FOR USE IN EQUATION 4.3*

Season	Coefficient	Aggregate Type	Coefficient	Bar Number	Coefficient
Winter	0.018	Limestone	0.000	4	-0.040
Spring	0.104	Siliceous		5	-0.031
Summer	0.102	River Gravel	-0.168	6	0.000
Fall	0.000			7	0.000

f_t = tensile strength (psi),
ϕ = bar diameter (inches),
α_s = thermal coefficient of steel (in./in./°F),
P = percent steel reinforcement,
σ_w = wheel load stress (psi), and

$a_1, a_2, a_3, a_4, a_5, b_1, b_2, b_3, b_4$ are positive constants.

The theoretical results were further confirmed in the model study reported in CFHR Report 177-16 (Ref 2). Results obtained from that analysis may be further simplified as follows:

(1) Crack spacing increases with increasing D, f_t, α_s/α_c, and ϕ. It decreases with increasing σ_w, ΔT_i, ΔT_f, F/y, Z and p;

(2) Crack width increases with increasing f_t, α_s/α_c and ϕ. It decreases with increasing σ_w, ΔT_i, ΔT_f, F/y, D and p;

(3) Steel stress increases with increasing ΔT_f, D, f_t, α_s/α_c and ϕ. It decreases with increasing σ_w, ΔT_i, F/y, Z and p

where

D = pavement thickness,
α_c = thermal coefficient of concrete,
ΔT_i = daily temperature change,
ΔT_f = final temperature change,
F/y = friction movement ratio, and
Z = shrinkage strain.

Although individual identity of the concrete properties is not maintained in the models developed for Project 422, an analysis of the results can still be made based upon the work presented in Chapter 3.

All three steel reinforcement design equations introduced in this chapter predict a higher steel requirement for limestone aggregate. Based on the concrete property models presented in Chapter 3, limestone and siliceous river gravel aggregates compare as shown in Table 4.4.

Table 4.4 shows a higher concrete tensile strength for limestone in comparison with siliceous river gravel. Furthermore, the values for concrete shrinkage and thermal coefficient are lower for limestone in comparison with siliceous river gravel. Based on these three concrete properties and considering all other parameters constant, pavement concrete comprised of limestone aggregate should develop a higher steel stress, crack spacing, and crack width than the siliceous river gravel concrete. A relative increase in these three parameters would result in a higher steel requirement for the limestone aggregate (see Fig 4.2). In this respect the design equations are accurate in predicting a higher steel requirement for concrete composed of limestone aggregate in comparison with siliceous river gravel.

Elastic modulus, which was the fourth concrete property used as an input to the computer program, does not appear directly in the theoretical relationship presented earlier. However, it is related to the wheel load stress, σ_w, by

TABLE 4.4. *COMPARISON OF CONCRETE PROPERTIES OF LIMESTONE AND SILICEOUS RIVER GRAVEL AGGREGATES*

Concrete Property	Aggregate Type: Limestone	Aggregate Type: Siliceous River Gravel
Tensile strength, $f_{t_{28}}$	411 psi	386 psi
Drying shrinkage, Z_{28}	149×10^{-6} in./in..	163×10^{-6} in./in.
Thermal coefficient, α_c	6×10^{-6} in./in./°F	8×10^{-6} in./in./°F
Elastic modulus, E_{28}	5.7×10^{-6} psi	5.1×10^{-6} psi

Westergaard's equation for pavement loading (Ref 13). An increase in pavement stiffness would result in higher wheel load stress. This, in terms of the theoretical relationship, would cause lower steel requirements. The slightly higher modulus for limestone concrete has reduced the difference in steel percentage requirement for the two aggregates.

There are four variables other than the concrete properties mentioned in the theoretical relationship. These are pavement thickness (D), bar diameter (ø), temperature drop (ΔT_i and ΔT_f), and friction moment ratio (F/y). For the purpose of this study, friction movement, or the slab–subbase friction relationship, was not considered as a variable. Hence, it does not form a part of the design algorithms developed for this project.

As is apparent from the models developed for this project, crack spacing, crack width, and steel stress increase with increasing pavement thickness. This is also illustrated by the design charts shown in Figs 4.2 and 4.3. A higher steel requirement indicates that crack spacing, crack width, and steel stress increase with thicker pavements. This is in accordance with the theoretical relationship.

In a similar manner, the design equations also show an increase in crack spacing, crack width, and steel stress with larger bar diameters. This is also illustrated in the design charts in Figs 4.2 and 4.3. Larger bar sizes result in a higher steel reinforcement requirement. The effect of higher temperature drop also influences the design models by lowering the steel requirement.

The exact magnitude of the effect each of the variables discussed above had on the design equations cannot be ascertained directly. Only a factorial study leading to a sensitivity analysis can determine the significance of the parameters involved in the model. However, it is certain from the preceding discussion that the parameters of which the model is composed affect it in accordance with the theoretical expectations. Furthermore, the two models developed for this project comprise the necessary components influencing the design of steel reinforcement for CRC pavements.

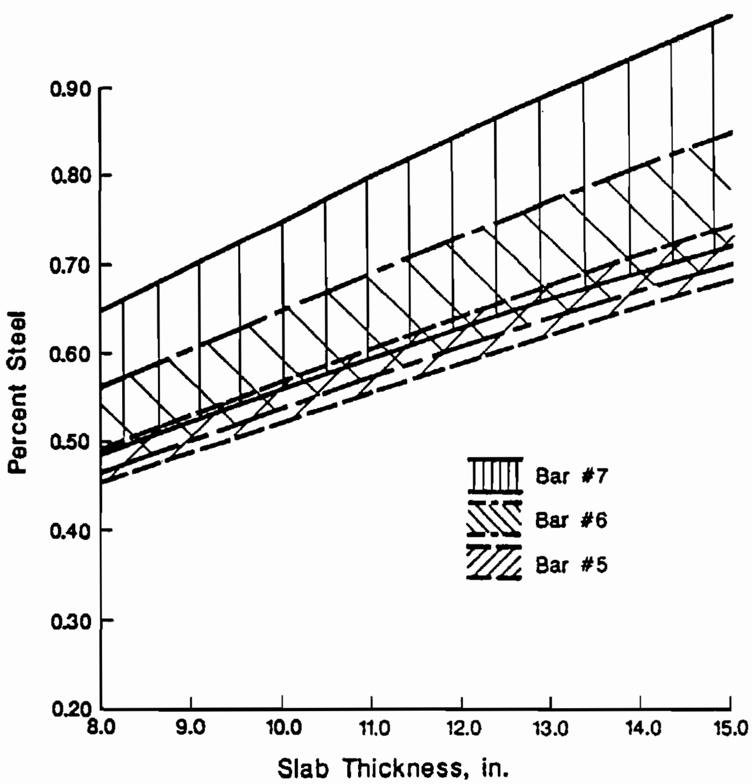


Fig 4.2. ***Steel design charts for limestone aggregate for all seasons and locations in Texas.***

DESIGN CHARTS

The primary objective of developing the design models was to provide design specifications identifying variations in concrete properties due to aggregate type. This study was done on aggregates found and used in Texas. Accordingly, a design which would be applicable for the whole state had to be formulated. Such design charts are illustrated in Figs 4.2 and 4.3 for limestone and siliceous river gravel aggregates, respectively. These design charts are applicable for all locations and placement seasons in the State of Texas. While geographic location was considered as a variable in terms of simulating different temperature drops, it appeared to be insignificant for the range of values considered for conditions in Texas, in the regression analysis. The placement season for concrete, however, was an important variable.

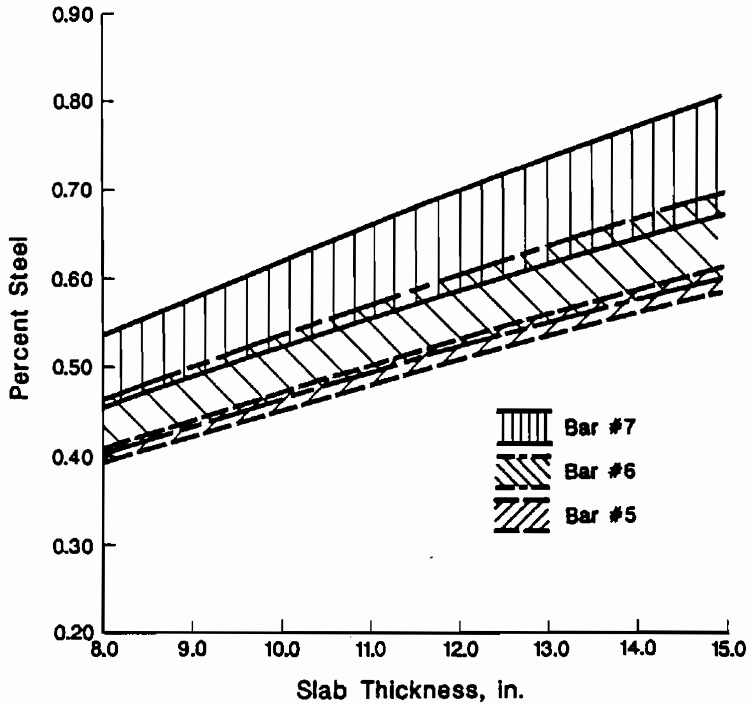


Fig 4.3. ***Steel design charts for siliceous river gravel aggregate for all seasons and locations in Texas.***

Several steps had to be taken in order to develop a design applicable for all seasons. The following sections explain this procedure.

Utilization of the Design Equations

The major accomplishment of the design models is the capability to design steel reinforcement controlling the limiting criteria of pavement design. In this respect, the design equations may be rewritten in the following format:

$$PS = [(e_B^{-2.312 + S + A + B}) (D^{1.304}) (CS^{-1})]^{0.508} \tag{4.4}$$

using coefficients from Table 4.1;

$$PS = [(e_B^{-2.587 + S + A + B}) (D^{1.230}) (0.01 \cdot CW^{-1})]^{0.516} \tag{4.5}$$

using coefficients from Table 4.2; and

$$PS = ([e^{-0.688 + S + A + B}) (D^{0.732}) (10 \cdot SS^{-1})]^{0.889} \tag{4.6}$$

using coefficients from Table 4.3.

With an appropriate input of the limiting criteria and design inputs a design value for the steel reinforcement may be calculated. The procedure to determine the design value, however, is slightly complicated and can be understood only in terms of some basic concepts. As mentioned earlier the design of steel reinforcement is controlled by the following limits:

Crack spacing	3.5 feet to 8.0 feet,
Crack width	less than 0.047 inch,
and	
Steel stress	less than 60 ksi.

The relationships between the amount of steel reinforcement and the three performance criteria are shown in Fig 4.4. All three factors decrease with increasing steel percentage. However, the crack spacing criterion requires a range of acceptable steel reinforcement ratios. A conceptual illustration of how the design bands for Figs 4.2 and 4.3 are formulated is provided in Fig 4.5. The largest steel design range is conceived by the crack spacing criterion, with 3.5 feet crack spacing requiring the maximum steel percentage and 8.0 feet requiring the least (Fig 4.5, Case 1). For these conditions, both the crack width and steel stress criteria require lesser steel ratios than crack spacing of 8.0 feet. In the event that either crack width (Fig 4.5, Case 3) or steel stress (Fig 4.5, Case 2) should require more steel, the maximum crack spacing criterion would control the lower limit of the acceptable design range. The last possibility is that crack width or steel stress could require more steel percentage than the crack spacing criterion of 3.5 feet. This situation would result in no solution (Fig 4.5, Case 4), i.e., there is no percentage of steel which can satisfy all four pavement performance criteria. Mathematically, this concept may be illustrated in a very simple manner, performing the following steps.

(1) Determine the percent steel required for the crack spacing of 3.5 feet. This is P_{max}.

(2) Calculate the three steel ratios corresponding to crack spacing of 8.0 feet, crack width of 0.047 inch, and steel stress of 60 ksi.

(3) Choose the highest value of the three ratios determined in Step 2. This value is P_{min}.

(4) Compare the values of P_{min} and P_{max}. If $P_{max} > P_{min}$ then any value chosen in the range of P_{min} to P_{max} is acceptable for all criteria.

(5) If $P_{min} > P_{max}$ then there is no solution for the pavement criteria set for design.

In the event of no solution, there are two methods to approach.

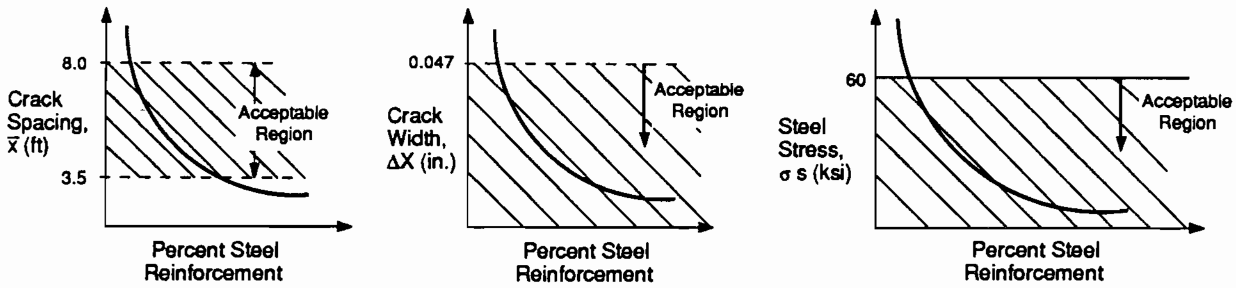


Fig 4.4. ***Conceptual illustration of the relationship of the three pavement performance criteria with steel reinforcement ratios.***

(1) Alter the limiting criteria for design.

(2) Change the variables in the controlling equation for P_{min}. For example steel stress may be reduced by choosing a larger bar size.

Requirements for the Design Charts

A primary requirement in the development of design charts applicable for the entire State was the elimination of the temperature variable. The involvement of the temperature variable was through geographic locations and placement seasons. Regression analysis performed on the CRCP-4 output had included both these variables. However, results had indicated that the affect of geographic locations (Brownsville, Port Arthur, and Amarillo) was insignificant, and it had therefore been dropped from the model. However, the importance of the placement season could not be ignored. The task at this stage was to recommend a design which could satisfy the simulated climatic conditions for all four seasons. Conceptually, this task may be explained in the following manner.

(1) Determine the lowest value of P_{max} for crack spacing of 3.5 feet by selecting the appropriate season.

(2) Evaluate the highest possible value of P_{min} for any of the three criteria for crack spacing of 8.0 feet, crack width of 0.047 inch, or steel stress of 60 ksi. This again would be done by choosing a season coefficient which would maximize the steel percentages.

(3) The process would have to be repeated for both aggregates and all pavement thicknesses.

This in effect would provide the narrowest band for all seasons and thus provide a steel design range applicable to all placement seasons. This is the concept used to develop the design charts shown in Figs 4.2 and 4.3.

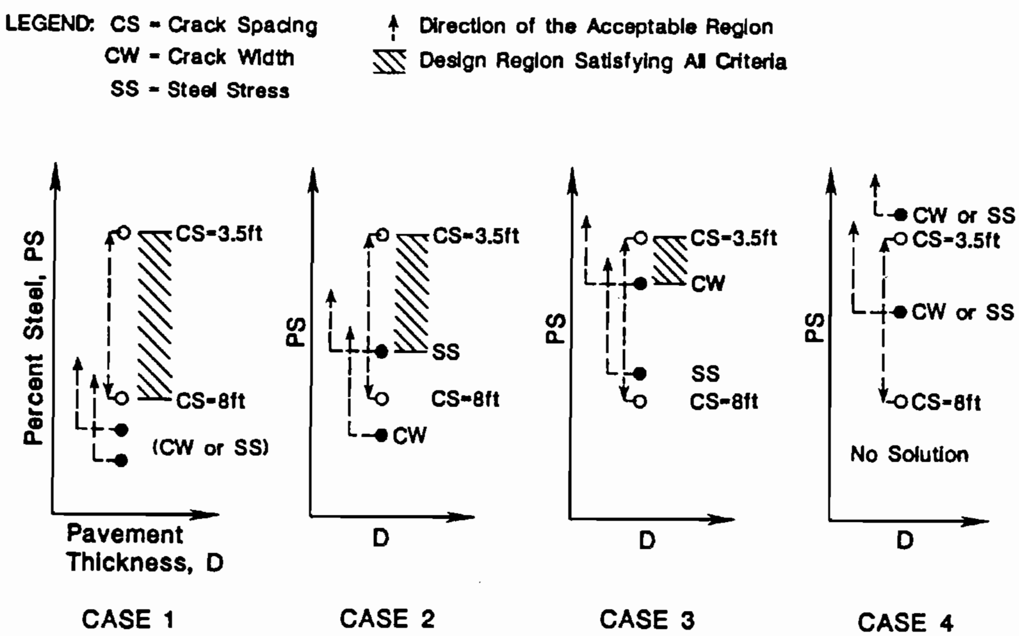


Fig 4.5. ***A conceptual illustration of various possibilities controlling the design range of steel reinforcement.***

CHAPTER 5. COMPARISON OF THE DESIGN MODELS WITH THE AASHTO EQUATIONS

INTRODUCTION

In order to evaluate the significance of any design algorithm it must be compared with other similar models in existence. The 1986 AASHTO Guide for Design of Pavement Structures (Ref 7) has for the first time provided the engineer with design equations for an approximate solution of steel reinforcement for CRC pavements. These equations were introduced in Center for Highway Research (CFHR) Research Report 177-16, "Nomographs for the Design of CRCP Steel Reinforcement" (Ref 2). Implementation of these equations into the complete design procedure for CRC pavements was recommended in Center for Transportation (CTR) Research Report 177–22F (Ref 11). It was in this report that for the first time different values were recommended for properties of concrete comprised of limestone and siliceous river gravel aggregates. The AASHTO equations are similar in form and input requirements to the models developed for Project 422. Accordingly, a comparison of the AASHTO equations with the equations developed in this study is provided in this chapter.

$$\overline{X} = \frac{1.32\left(1+\frac{f_t}{1000}\right)^{6.70}\cdot\left(1+\frac{\alpha_s}{2\alpha_c}\right)^{1.15}\cdot(1+\phi)^{2.19}}{\left(1+\frac{\sigma_w}{1000}\right)^{5.20}\cdot(1+P)^{4.60}\cdot(1+1000Z)^{1.79}} \tag{5.1}$$

$$\Delta X = \frac{0.00932\left(1+\frac{f_t}{1000}\right)^{6.53}\cdot(1+\phi)^{2.20}}{\left(1+\frac{\sigma_w}{1000}\right)^{4.91}\cdot(1+P)^{4.55}} \tag{5.2}$$

and

$$\sigma_s = \frac{47300\left(1+\frac{DT_D}{100}\right)^{0.425}\cdot\left(1+\frac{f_t}{1000}\right)^{4.09}}{\left(1+\frac{\sigma_w}{1000}\right)^{3.14}\cdot(1+1000Z)^{0.494}\cdot(1+P)^{2.74}} \tag{5.3}$$

THE AASHTO EQUATIONS

The CRCP-4 computer program developed at The University of Texas at Austin provides the most comprehensive procedure for the analysis and design of CRC pavements. This computer program has the capability to incorporate all the required design inputs for CRC pavements, including concrete properties, to provide pavement performance predictions. The program, however, is not available to every engineer interested in continuously reinforced concrete pavement design. It was for this purpose that an approximate solution in the form of equations for the design of steel reinforcement was developed in CFHR Research Report 177-16. These are regression equations for the prediction of the three design parameters-crack spacing, crack width, and steel stress. Formulation of these equations was made using multiple linear and nonlinear square fits to a fractional factorial of simulated observations which were outputs of the CRCP–4 computer program. Theoretical models developed in CFHR Report 177-17 (Ref 12) were the basis for selecting the form and variables to be considered for these equations. These theoretical models were discussed in the previous chapter for the purpose of analyzing the design models developed in this study. The design equations presented in the (1986) AASHTO Guide are as follows:

where

$\overline{X}$ = crack spacing (feet),
ΔX = crack width (inches),
σ_s = steel stress (psi),
f_t = concrete tensile strength (psi),
α_s = thermal coefficient of steel (in./in./°F),
α_c = thermal coefficient of concrete (in./in./°F),
ϕ = rebar diameter (inches),
σ_w = wheel load tensile stress (psi),
P = percent steel reinforcement,
Z = concrete shrinkage (in./in.), and
DT_D = design temperature drop (°F).

DESIGN CHARTS BASED ON AASHTO EQUATIONS

A computer program utilizing the AASHTO equations (5.1, 5.2, and 5.3) was developed for the purpose of preparing design charts similar to those produced by the models introduced in the previous chapter. Concrete properties determined by the models presented in Chapter 3 were used as inputs to the computer program. A curing temperature of 75°F and 75 percent relative humidity were used for all calculations. These were the conditions used for developing the design charts shown in Figs 4.2 and 4.3. The approach for modelling the climatic conditions, however, was different in this case. The AASHTO procedure requires a single input of a constant value for the design temperature drop for its steel stress equation. This value was chosen from the recommendations provided in CTR Research Report 177-22F. Considering the form of the AASHTO steel stress equation, the highest value of the temperature drop in Texas maximizes the steel reinforcement based on the steel stress criteria. Referring to the explanation provided in Chapter 4 for the derivation of the design charts, this condition produced the design chart applicable to all environmental conditions in the State of Texas. Accordingly, a value of 95°F was used as an input to the computer program. For the determination of the stress due to wheel load, the program utilized Westergaard's interior loading equation (Ref 13). The design charts developed from the AASHTO equations for limestone and siliceous river gravel aggregates are shown in Figs 5.1 and 5.2, respectively.

A comparison of the design charts determined by the AASHTO equations with those developed with the models from this report (Figs 4.2 and 4.3) indicates a similarity in the form of these charts. The design bands determined by the AASHTO equations have considerable overlap with those derived on the basis of the models introduced in this report. Similar to the results obtained in this study, the AASHTO equations also require a higher steel percentage for limestone aggregate. There are, however, two notable differences between these charts and those described in Chapter 4. First, although the results from both algorithms are quite close for lower pavement thicknesses, the AASHTO equations do not increase the steel requirement for thicker pavements as much as the models developed in this study. Secondly, the lower boundary (P_{min}) for the design bands for the AASHTO equations is identical for all bar diameters in the cases of both aggregates.

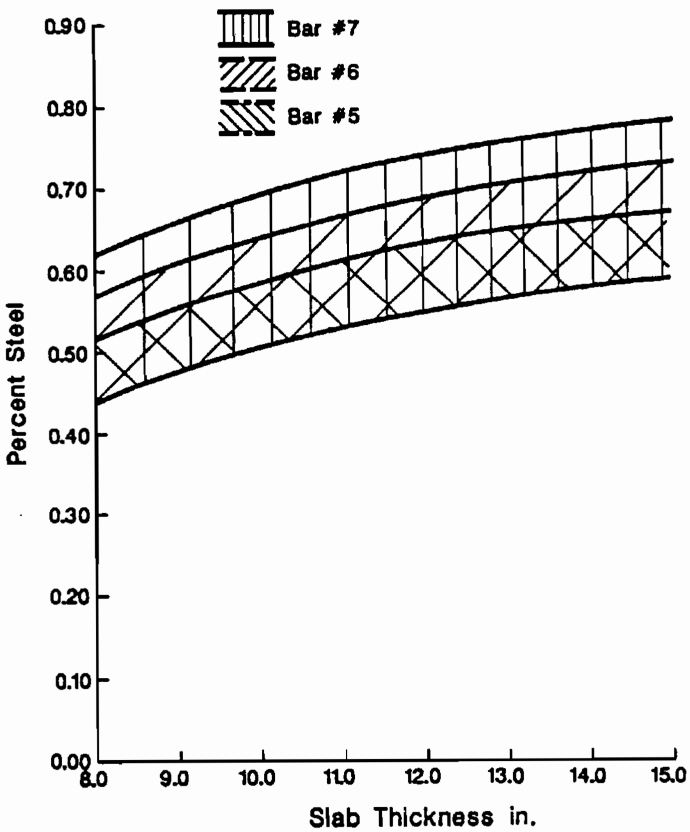


Fig 5.1. *Design chart for limestone aggregate using the AASHTO equation.*

DISCUSSION OF THE RESULTS

Considering the form of the AASHTO equations and the process of their derivation, a similarity between the results predicted by them and the models developed in this study is expected. The models developed in this study were compared to the theoretical models from CFHR Report 177-16 in the previous chapter. Models from this study confirmed the expectations based on those theoretical models. It is important to note that the parameters for the AASHTO equations were selected based on those theoretical models. In this respect the effects of various concrete properties on the AASHTO equations and the models introduced in this report are similar.

There are some differences, however, between the AASHTO equations and the models derived in this study. The AASHTO equations are severely limited in modelling climatic conditions. For the AASHTO design procedure, temperature drop appears as a variable only in the steel stress equation (Eq 5.3). The design models from this report indicate that temperature drop affects all three CRC pavement design criteria, which is in agreement with the theoretical models discussed in Chapter 4. A similar case is observed for the rebar diameter. Both the theoretical models and the results from this study indicate the influence of bar diameter on crack spacing, crack width, and steel stress. However, bar diameter is not a variable in the steel stress equation for the AASHTO procedure. For the design charts in Figs 5.1 and 5.2, steel stress was the controlling criterion for the lower boundary of the design bands. It was for this reason that the AASHTO equations indicated the same P_{min} value for all bar diameters.

The primary objectives of the AASHTO equations and the models developed in this study were different; therefore emphasis on the various involved parameters is different in both cases. Thus, slight differences in the final values are expected, due to the regression process involved in their formulation. The AASHTO equations were derived on the basis of simulated data to formulate an approximate procedure for the design of steel reinforcement. On the

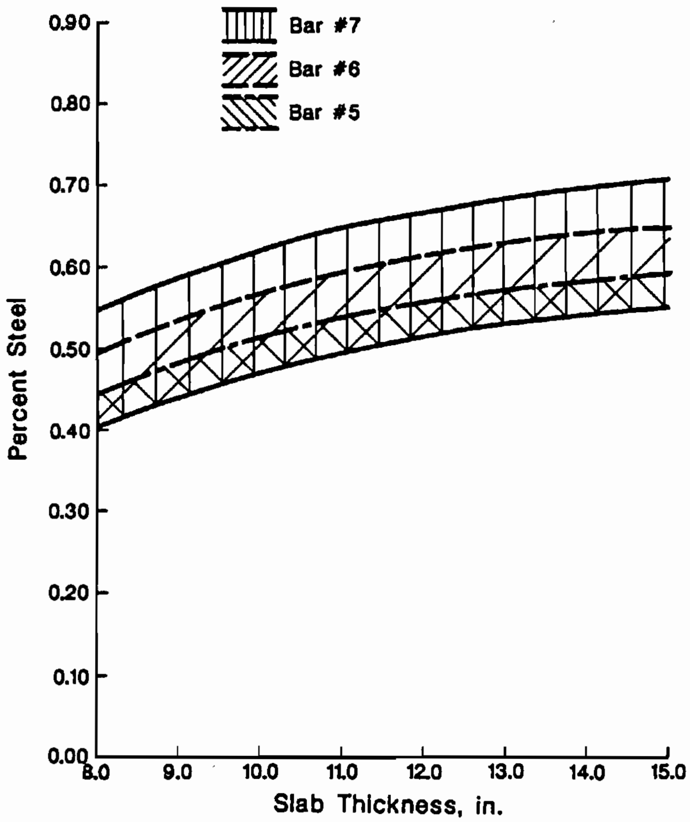


Fig 5.2. ***Design chart for siliceous river gravel aggregate using the AASHTO equations.***

other hand, the models for this project were specifically developed to compare the performance of mixes using the limestone or siliceous river gravel aggregates. These models were developed using the actual laboratory measurements of the properties of concrete comprising these two aggregates. Another constraint in developing other input parameters for this analysis was that the design process was to be formulated specifically for the state of Texas. In this respect the models presented in this report had a more definite but limited goal, while the AASHTO equations were developed to provide an approximate solution for more universal conditions.

Considering the fact that AASHTO equations were developed to provide a general solution, the results predicted by these equations are reasonably close to those determined by this analysis. An attractive feature of the AASHTO procedure is that it includes the concrete properties in the equations. Since, at present, laboratory data are available for only two aggregate types, the concrete properties for the models introduced in this report were lumped into a single variable, the aggregate type identifier. This imposes a limitation for the CRC design. In this respect the form of the AASHTO equations provides the engineer more flexibility in considering the specific conditions applicable to a particular case. Furthermore, the laboratory measurements of the concrete properties for this project have indicated that a significant amount of variability exists within the properties of concrete comprising a particular aggregate type. The provision using specific concrete properties would allow the possibility of including the measured properties in any given design. The laboratory data for limestone and siliceous river gravel aggregates are available at the present time. As more data on other aggregates become available, the current models can be improved to increase their applicability by including individual concrete properties. Additionally, both the AASHTO models and those from this report need to be calibrated on the basis of field observations.

CHAPTER 6. PROBABILISTIC APPROACH TO CRC PAVEMENT REINFORCEMENT DESIGN

INTRODUCTION

As observed in the previous two chapters, the solution to steel design of CRC pavements is often not unique. This aspect is illustrated by the acceptable solution "bands" presented in the design charts in the earlier chapters. Furthermore, the form of the design equations introduced in Chapter 4 allows the variation of several parameters in design solutions. In the absence of any other guideline, and considering only the cost factor, the engineer is limited to selecting the minimum amount of acceptable steel. This minimum amount of steel (P_{min}) corresponds to the lower boundaries of the bands in the design charts. These design charts were formulated on the basis of the limits on crack spacing, crack width, and steel stress. Keeping the nature of these limiting criteria in perspective, the choice of P_{min} for the design solution may not be appropriate. In fact, an excessive and a too small amount of steel can be equally bad for the pavement. This aspect may be explained in light of Fig 4.4, where the relationship of the three pavement design criteria with the ratio of steel in the pavement was explained. Increasing steel percentage above the minimum requirement is allowable for both the steel stress and the crack width criteria. This is because both criteria have an upper boundary for their limit and increasing steel in the pavement would only reduce both parameters, which is acceptable. The criteria for crack spacing, however, are more complex. This is because the limiting criterion for crack spacing has both an upper and a lower boundary. While the upper boundary (8.0–foot spacing) may be violated with too small an amount of steel, an excessive amount of steel reinforcement in the pavement creates problems with the high occurrence of very small crack spacings (<3.5 feet). Thus, the crack spacing requirement creates complexity in obtaining the solution. Harnessing the crack spacing criteria to develop an optimum solution is, therefore, the key to determining steel designs with increased reliability. This chapter introduces a probabilistic method for approaching the optimum solution.

PROBABLISTIC ESTIMATION OF TRANSVERSE CRACK SPACING

The design procedures introduced in this report as well as those in the AASHTO method (Chapter 5) provide a deterministic solution for steel design of CRC pavements. However, pavements built according to the specifications based on these design methods may not perform in exactly the manner predicted by these models. This is due to the variability that exists in the material properties, construction techniques, and, above all, field conditions. Thus, there is a need to incorporate this variability into the design procedure to develop a better assessment of the design. In this respect, this chapter introduces a concept for determining design solutions based on the variability observed in the field. The main idea is to minimize the probability of violating the limiting criteria and thus maximize the chances of satisfactory performance. In order to illustrate the concept let us assume that the transverse crack spacing (CS) is normally distributed with a mean equal to CS and a standard deviation of σ_{cs}. The probability that crack spacing is equal to or less than a specified value, A, can be estimated with the help of the standardized parameter, Z, as follows (Ref 14):

$$Z = \frac{A - \overline{CS}}{\sigma_{CS}} \tag{6.1}$$

where Z is normally distributed with a mean = 0, and standard deviation = 1 or [Z is N(0,1)]. A standard table of normal distribution can be used to calculate the probability that crack spacing is equal to or less than A as follows:

$$P[CS \le A] = F(Z)$$

where F (Z) is the cumulative distribution function of the standard normal random variable, Z, between $-\infty$ and Z.

Therefore, to determine the probability, P, for crack spacings between 3.5 and 8.0 feet, use

$$P = F(Z_{8.0}) - F(Z_{3.5}) \tag{6.2}$$

where

$$F(Z_{3.5}) = P(CS \le 3.5) \text{ and}$$

$$F(Z_{8.0}) = P(CS \le 8.0).$$

Using this concept, various design solutions can be compared in terms of their corresponding reliability. The optimum steel design solution would involve the selection from the acceptable design bands (Chapter 4) the steel reinforcement which maximizes P (Eq 6.2).

FIELD DATA

In the procedure described above, two variables are required for the estimation of Z. These are the mean crack spacing CS, and the standard deviation (SD) σ_{cs}. It is reasonable to assume that the crack spacing determined by Eq 4.1 represents the mean crack spacing. The σ_{cs} value, however, is the SD of the crack spacing observed in the field. Field data providing crack spacing measurements are, therefore, required for determining this value.

The future work plan for Project 422 involves the collection of field data on existing sections as well as the construction of special test sections to model the various design variables. Such field data should provide information valuable for improving and calibrating the design models. At present, however, the availability of some crack spacing data from the Center for Transportation Research Data Base allows the presentation of an illustrative example.

The data form a part of the condition survey performed in 1978. The sections on which these specific crack spacing data were collected are located in Texas, Districts 1, 2, 3, 13, 15, 18, and 20. All sections had an 8-inch thickness, were constructed with limestone aggregates in the mix, and had a cement-treated base. There were 35 test sections and the number of measurements were 1,676. An analysis of the crack spacing data indicated a log normal distribution with a coefficient of variance (CV) equal to 43 percent. Considering the fact that the data are log normally distributed, Eq 6.1 can be rewritten as follows:

$$Z_{CS} = \frac{LA - \overline{LCS}}{\sigma_{CS}} \tag{6.3}$$

where

$\overline{LCS}$ = the mean value of the log of crack spacings and

LA = log of any crack spacing being considered.

In order to calculate the standard deviation, the following relationship can be used:

$$CV = \frac{\text{standard deviation}}{\text{mean}} \times 100 \tag{6.4}$$

or

$$\sigma_{cs} = \overline{LCS} \times \frac{CV}{100} \tag{6.5}$$

As mentioned earlier, the $\overline{LCS}$ (ln CS) value is determined directly from Eq 4.1. The following section provides an illustrative example for developing the cumulative probability relationship for crack spacing.

ILLUSTRATIVE EXAMPLE

Equation 4.1 relates the crack spacing to percent steel as follows:

$$\ln CS = -2.31 + S + A + B + 1.304 \ln D - 1.97 \ln PS$$

where all terms used in this equation are explained in Chapter 4.

For this example, the following conditions are assumed:

- winter season placement
- #5 bar diameter
- limestone aggregate
- 8-inch pavement thickness

Considering 0.5 percent steel and utilizing the coefficients for other parameters as provided in Table 4.1, the mean crack spacing is determined as follows:

$$\ln CS = -2.31 + 0.195 + 0.000 - 0.541 + 2.711 + 1.365$$

$$= 1.42$$

or

$$\overline{LCS} = 1.42$$

Using Eq 6.5 and a coefficient of variance (CV) of 43 percent, the standard deviation (σ_{cs}) is determined as 0.61. For different crack spacings, the values for Z can be calculated with the help of Eq 6.3. The determination of Z allows the estimation of cumulative probability from any standard normal distribution tables. Results of the computation for 0.5 percent steel are presented in Table 6.1. To expand on this concept and observe the effect of the amount of steel reinforcement on the crack spacing, computations for 0.4 and 0.6 percent steel reinforcement are also recorded in Table 6.1. All factors were considered identical to those in the case of 0.5 percent steel. The computations provided 0.80 and 0.46 as the values for σ_{cs} corresponding to 0.4 and 0.6 percent steel, respectively. The comparison of cumulative probability for crack spacings corresponding to different steel percentages is illustrated in Fig 6.1.

TABLE 6.1 ***COMPUTATIONS FOR CUMULATIVE PROBABILITY CORRESPONDING TO DIFFERENT CRACK SPACINGS***

Crack Spacing (feet)		0.4 Percent Steel		0.5 Percent Steel		0.6 Percent Steel	
(A)	ln A	Z	Percent Probability	Z	Percent Probability	Z	Percent Probability
2	0.69	11.46	7.0	-1.19	11.7	-0.80	21.2
4	1.39	-0.59	27.8	-0.05	48.0	0.71	75.8
6	1.79	-0.09	46.4	0.61	72.0	1.59	94.4
8	2.08	0.27	60.6	1.08	86.0	2.21	98.6
10	2.30	0.55	71.0	1.45	92.6	2.70	99.6
12	2.49	0.78	78.2	1.75	95.9	3.09	99.8
14	2.64	0.98	83.5	2.00	97.7	3.43	-
16	2.78	1.14	87.3	2.22	98.6	3.72	-
18	2.89	1.29	88.9	2.41	99.2	2.98	-

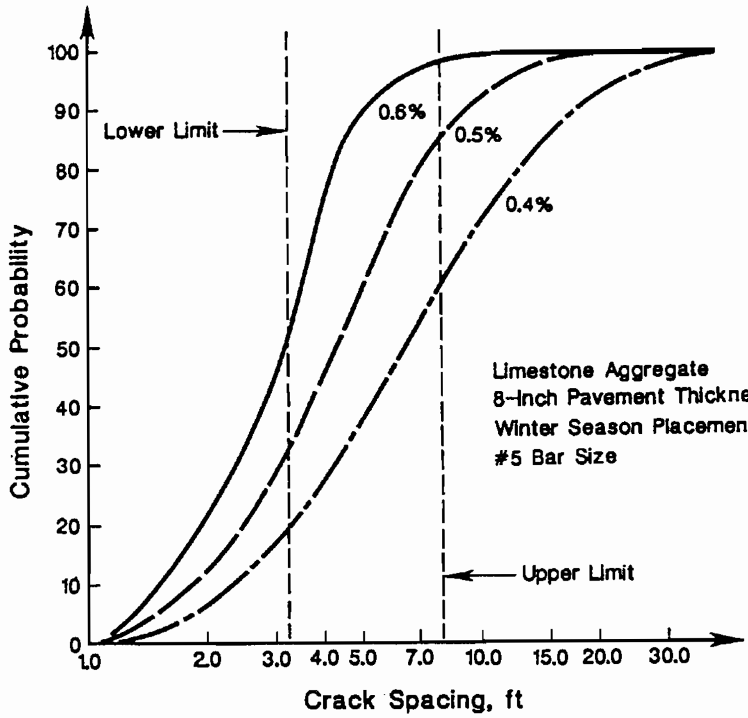


Fig. 6.1. ***Comparison of cumulative probability for crack spacings (effect of variation in steel percentage).***

An extension to the concept of varying steel ratios in the pavement is to note the effect of different bar sizes. All models including crack spacing introduced in Chapter 4 are influenced by the choice of bar diameter to be used in the design. To analyze the effect of bar size on crack spacing, Eq 4.1 was used in a manner similar to that in the previous case. The only exception was that the percent steel was kept constant at 0.5 percent and different bar sizes were used to determine the mean crack spacings and their corresponding σ_{cs} values. The results of the computations for bar numbers 5, 6, and 7 are summarized in Table 6.2. The comparison of cumulative probability for crack spacings corresponding to different bar sizes is illustrated in Fig 6.2.

ANALYSIS OF THE RESULTS

Two important conclusions can be observed in the comparisons plotted in Figs 6.1 and 6.2.

(1) While increasing the percent steel provides a higher probability of staying within the upper limit (8.0 feet) of crack spacing, it reduces the chances of satisfying the lower limit (3.5 feet) criteria for crack spacing.

(2) An effect similar to that in (1) is noted by reducing the bar diameter. Smaller bar sizes increase the possibility of staying within bounds of the higher (8.0–foot) crack spacing limit. However, the choice of a smaller bar diameter also raises the possibility of violating the lower limit (3.5 feet) of the crack spacing criteria.

The general conclusion is that there are obvious trade offs in both increasing or decreasing steel ratios and bar diameters. This procedure allows the engineer to investigate several steel reinforcement design options. The optimum solution is the one which provides the maximum probability of remaining within the upper and lower limits of the crack spacing criteria, or, explained another way, the method to obtain the optimum solution would involve the maximization of P (Eq 6.2.).

In order to complete the design procedure, the method introduced in this chapter should be used along with the design equations provided in Chapter 4. The design solutions must first be obtained in the form of the design charts (Figs 4.2 and 4.3). Several solutions should be obtained

TABLE 6.2 ***COMPUTATIONS FOR CUMULATIVE PROBABILITY CORRESPONDING TO DIFFERENT CRACK SPACINGS***

Crack Spacing (feet) (A)	ln A	# 5 Bar Z	# 5 Bar Percent Probability	# 6 Bar Z	# 6 Bar Percent Probability	# 7 Bar Z	# 7 Bar Percent Probability
2	0.69	-1.19	11.7	-1.38	8.4	-1.51	6.6
4	1.39	-0.05	48.0	-0.40	34.5	-0.68	24.8
6	1.79	0.61	72.0	0.15	56.0	-0.20	42.1
8	2.08	1.08	86.0	0.55	70.9	0.14	55.6
10	2.30	1.45	92.6	0.86	80.5	0.40	65.5
12	2.49	1.75	95.9	1.13	87.1	0.63	73.6
14	2.64	2.00	97.7	1.33	90.8	0.81	79.1
16	2.78	2.23	98.6	1.53	93.7	0.98	83.7
18	2.89	2.41	99.2	1.68	95.4	1.11	86.7

from within the design bands. This process insures that all solutions being considered have been screened to satisfy all the limiting criteria. The optimum design which maximizes P should then be selected from these solutions. Thus the method developed in this chapter should act as a screening method for approaching a maximum reliability design.

The computations provided in this section are valid only for the particular set of conditions set at the beginning of the illustrative example. As field data for various other design conditions are made available, this method can be expanded for any design situation. By including field variability as the basis of computations, the method introduced in this chapter allows the incorporation of this variability in the design procedure. Furthermore, this method serves as a calibration procedure for the design models. The accuracy of the predictions from the design models is adjusted in terms of the field variablity. In the event that the design models' predictions deviate from the actual conditions in the field, it would correspond in terms of a lower design reliability by this method. Thus this method also serves as a gage for assessing the reliability associated with different design recommendations.

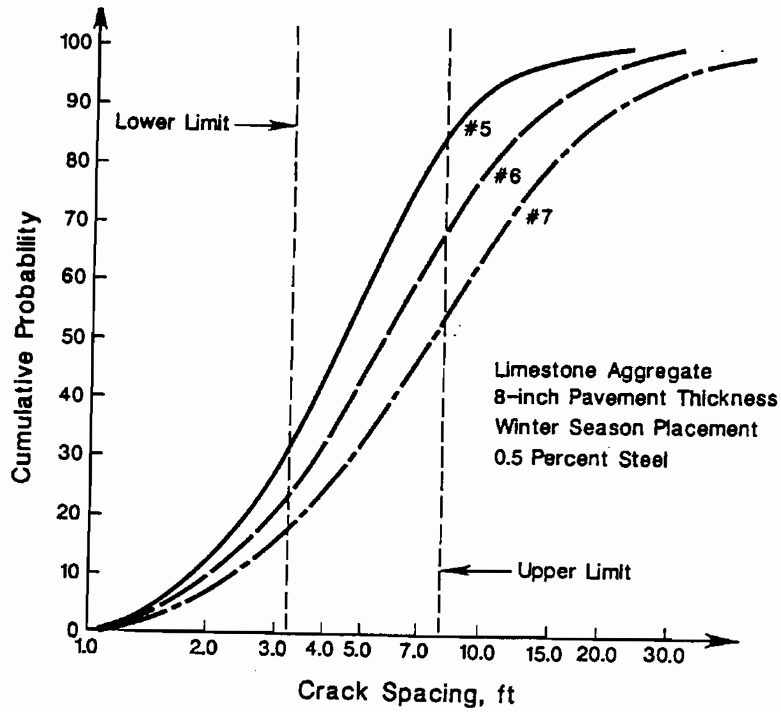


Fig. 6.2. ***Comparison of cumulative probability for crack spacings (effect of variation in bar size).***

CHAPTER 7. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

SUMMARY

Several objectives have been accomplished to date as a result of this study. Significant differences have been noted in the material properties of concrete mixes composed of limestone and siliceous river gravel aggregates. This is reflected in the differences that exist in the concrete property models for the two aggregate types. These models, introduced in Chapter 3 of this report, were developed on the basis of a statistical analysis performed on the laboratory measurements of the material properties of concrete mixes composed of limestone and siliceous river gravel aggregates.

Considering the CRC pavement design, the fact that concrete mixes composed of different aggregate types reflect variations in material properties is inconclusive. Accordingly, an assessment of the influence that the choice of either limestone or siliceous river gravel aggregate has on CRC pavement design was made by developing design models based on the concrete properties of these two aggregate types. These design models were presented in Chapter 4 of this report. The design models were developed utilizing the concrete property models and formulating an elaborate factorial of parameters influencing CRC pavement design. Parameters considered in this factorial included variablity in environmental conditions, pavement structure geometry, and steel reinforcement. This factorial was analyzed using the CRCP–4 computer program. A statistical analysis was performed on the outputs of the CRCP–4 program. It consisted of predictions for crack spacing, crack width, and steel stress for the variables considered in the analysis factorial. The limiting criteria for these three parameters control the design of steel reinforcement for CRC pavement.

The design models developed as a result of this analysis predict a significant difference in steel requirement between pavements built with limestone and those built with siliceous river gravel aggregates. Thus, the variation in concrete properties due to the use of these two aggregate types was translated into different steel requirements in terms of design. The models developed from this study were compared with the AASHTO steel design equations in Chapter 5 of this report. The AASHTO models were developed through a process similar to that used in formulating the design models for this report. The comparison resulted in a reasonable similarity between the predictions of the AASHTO equations and the models developed in this study. As a further refinement of the CRC design process a probabilistic approach to estimate the reliability of different designs was introduced in Chapter 6 of this report. This procedure when further developed and assimilated in the design process should serve as the criterion for comparing different variables affecting CRC pavement design.

CONCLUSIONS

The primary accomplishment of this report is that it has established a procedure for comparing the effects of the aggregate type upon the design requirements of CRC pavements. Results from the laboratory testing of concrete mixes for limestone and siliceous river gravel aggregates have indicated that significant differences exist in the material properties of concrete mixes made by using these two aggregate types. When these differences are considered in terms of CRC pavement design, the consideration of aggregate type cannot be ignored in the design. This aspect is emphasized by the distinctly different steel design requirements for limestone and siliceous river gravel aggregates predicted by the models developed in this report and confirmed by the AASHTO equations. Furthermore, the design solutions developed in this study indicate that a single steel reinforcement design which can provide satisfactory performance for pavements built with either aggregate type cannot be recommended. Models developed in this study, as well as the AASHTO equations, predict a higher steel requirement for limestone aggregate type. It is important to emphasize, however, that based on the concepts discussed in Chapter 6 of this report, the amount of steel reinforcement is not the criterion for the selection of a particular design or the parameter (e. g., aggregate type) upon which the design is based. The amount of steel reinforcement provides only a comparison of the first cost associated with any design. The actual cost is also dependent upon the expected performance of the design to be built in the field. In Chapter 6 of this report an example of estimating this expected performance or the reliability of a design was discussed. This concept can be further expanded after acquiring and analyzing sufficient field data. The criteria for selection of a design should be based upon both cost and the maximum reliability which can be achieved under the given constraints of a design situation. This concept leads to the comparison of life cycle costs for the selection of a design.

Based upon the work described in this report, limestone and siliceous river gravel aggregates influence the steel requirements of CRC pavements differently. Thus, it is important to distinguish which aggregate type is being used in the design. The design models from this report should be utilized to develop several alternate solutions which satisfy the limiting criteria for steel reinforcement. The optimum solution should then be selected, based upon a comparison of reliability and first cost. This approach should insure the selection of designs based on satisfying the limiting criteria for steel design and a screening process to determine the optimum solution based on maximum reliability. This report has established that CRC pavements built with limestone and siliceous river gravel aggregate types cannot be treated by a single design. Furthermore, a methodology for developing design recommendations for limestone and siliceous river gravel aggregates, as well as an example of estimating the associated reliability of these recommendations, has been developed in this report. The design models and the reliability concept need to be further developed as more field data are made available.

RECOMMENDATIONS

This report marks the completion of Phase I of Project 422. Two aggregate types, limestone and siliceous river gravel, have been tested and analyzed. Future plans for this project involve the testing of other aggregate types. Two important factors which have been observed in this part of the project need to be considerated for future work on the project.

(1) A significant amount of variability was observed in the measurements of material properties for each aggregate type. It is important to estimate the variability associated with the properties of each aggregate type in order to develop a better assessment of the design models formulated on the basis of these properties. Several material properties were measured for concrete mixes comprised of each aggregate type. It was noted that the maximum amount of scatter was present in the measurements for elastic modulus and concrete shrinkage. It is important to evaluate the testing procedures used for measuring these properties to investigate the possibility of reducing this scatter for the measurements to be made in the future. This in turn will help reduce the variability noted within the measurements of each aggregate type.

(2) In Chapter 6 of this report a concept for incorporating field variability into development of reliability estimates for the design models was introduced. The concept could not be expanded because of the lack of adequate data. Furthermore, the predictions of the models developed in this report cannot be confirmed until such data are available. It is recommended that field data be collected on existing pavement sections as well as sections built according to the new specifications issued by the SDHPT based upon the results of this study.

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APPENDIX

LABORATORY MEASUREMENTS OF THE CONCRETE PROPERTIES

APPENDIX

LABORATORY MEASUREMENTS OF THE CONCRETE PROPERTIES

TABLE A.1. *SRG MODULUS OF ELASTICITY (10^4 PSI)*

SILICEOUS RIVER GRAVEL							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	298.8	421.2	335.3	333.3	374.6	395.3
	2	248.4	199.3	322.4	234.2	230.0	766.3
	3	257.8	469.4	440.1	357.6	304.2	293.4
	AVG.	268.3	363.3	365.9	308.4	302.9	485.0
3 DAYS	1	336.3	233.2	412.6	495.1	255.0	460.8
	2	528.9	195.6	516.8	478.6	522.7	358.6
	3	526.4	325.2	443.5	420.3	664.7	480.7
	AVG.	463.9	251.3	457.6	464.7	388.9	433.4
7 DAYS	1	415.7	219.5	287.4	652.1	387.2	479.4
	2	612.2	328.4	355.4	583.7	602.1	853.4
	3	411.5	442.5	287.4	1309.9	1064.8	533.6
	AVG.	479.8	330.1	310.1	848.6	684.7	622.1
28 DAYS	1	362.5	452.8	378.3	457.1	539.0	769.0
	2	515.0	501.4	427.5	524.5	580.7	605.4
	3	410.9	575.0	354.6	375.8	662.7	599.0
	AVG.	429.5	509.7	368.8	452.5	534.1	657.8
90 DAYS	1	611.7	314.2	592.8	1440.7	238.7	1084.9
	2	506.6	602.1	215.0	467.5	668.6	1580.8
	3	602.1	524.5	650.4	787.7	452.8	1026.8
	AVG.	573.5	480.3	486.1	898.6	453.4	1230.8

TABLE A.2. ***LS MODULUS OF ELASTICITY (10^3 PSI)***

CRUSHED LIMESTONE							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	348.8	403.5	430.0	301.9	314.4	324.6
	2	120.9	628.4	333.3	320.0	1320.6	541.6
	3	366.9	465.6	549.5	298.0	960.4	415.7
	AVG.	278.9	499.2	437.6	306.6	865.1	427.3
3 DAYS	1	385.8	417.2	670.6	569.0	320.9	449.2
	2	479.4	392.5	489.4	560.5	563.3	461.8
	3	563.3	448.8	520.4	599.2	485.6	531.8
	AVG.	476.2	419.5	560.1	576.2	456.6	480.9
7 DAYS	1	516.8	291.2	387.2	701.4	500.7	549.3
	2	539.0	405.2	574.8	710.3	682.7	356.5
	3	428.3	414.2	373.0	663.8	426.7	450.6
	AVG.	494.7	370.2	445.0	691.8	536.7	452.1
28 DAYS	1	692.8	399.3	560.7	220.7	539.9	299.5
	2	577.7	487.3	547.6	129.2	351.7	583.8
	3	539.9	635.4	416.9	283.8	527.5	457.9
	AVG.	603.5	507.3	508.4	211.2	473.0	447.1
90 DAYS	1	638.9	1007.2	1185.6	694.0	849.4	526.9
	2	527.5	300.3	281.9	539.9	566.2	701.2
	3	577.7	—	184.9	631.9	293.3	1063.7
	AVG.	581.4	653.8	550.8	621.9	569.6	763.9

TABLE A.3. ***SRG SPLIT CYLINDER TENSILE STRENGTH (PSI)***

SILICEOUS RIVER GRAVEL							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	191.8	266.9	271.3	232.6	350.7	273.0
	2	183.2	254.3	303.3	203.6	313.9	274.3
	3	158.2	255.2	287.9	192.7	308.4	291.7
	AVG.	177.7	258.8	287.5	209.6	324.3	279.7
3 DAYS	1	249.6	345.5	318.3	334.6	380.6	377.4
	2	269.4	288.9	363.3	289.3	411.9	322.1
	3	254.1	329.8	308.1	370.3	373.0	305.9
	AVG.	257.7	321.4	329.9	331.4	388.5	335.1
7 DAYS	1	309.6	370.8	412.5	313.5	441.1	343.1
	2	325.3	397.3	383.9	333.8	457.1	344.6
	3	334.0	377.8	417.6	360.8	440.1	320.3
	AVG.	322.9	381.9	404.7	336.0	446.1	336.0
28 DAYS	1	329.0	429.2	355.3	346.8	528.2	435.3
	2	326.6	372.6	403.2	420.1	543.9	380.5
	3	340.6	374.6	337.8	399.2	492.8	390.7
	AVG.	332.1	392.2	365.5	388.7	521.6	402.2
90 DAYS	1	361.4	400.7	376.6	384.9	—	464.5
	2	399.5	422.3	428.5	387.6	—	409.9
	3	260.1	425.3	333.7	404.1	—	471.3
	AVG.	340.3	416.1	379.6	392.2	—	448.6

TABLE A.4. ***LS SPLIT CYLINDER TENSILE STRENGTH (PSI)***

CRUSHED LIMESTONE							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	203.2	242.0	291.0	239.6	249.6	302.0
	2	194.2	316.3	276.0	248.7	249.4	313.3
	3	269.4	269.2	288.9	228.7	237.9	350.6
	AVG.	222.3	275.9	285.3	239.0	245.6	322.0
3 DAYS	1	348.1	357.3	433.2	294.9	284.4	413.5
	2	315.8	395.5	353.0	329.1	340.2	316.4
	3	337.6	351.2	391.8	383.6	322.6	339.8
	AVG.	333.8	368.0	392.7	335.8	315.7	356.6
7 DAYS	1	352.3	400.8	323.6	426.8	284.4	449.2
	2	337.9	427.9	413.9	370.4	379.3	451.2
	3	335.7	407.8	428.8	367.5	371.1	404.8
	AVG.	342.0	412.2	388.8	388.2	344.9	435.1
28 DAYS	1	456.8	515.3	465.2	398.3	423.7	463.1
	2	404.8	494.1	320.5	363.9	432.3	357.1
	3	463.0	355.1	445.9	376.4	407.9	456.3
	AVG.	441.5	454.9	410.5	379.5	421.3	425.5
90 DAYS	1	411.2	476.8	277.0	372.5	437.0	443.2
	2	456.5	370.6	339.1	362.2	419.9	408.8
	3	—	393.6	384.8	488.2	451.6	391.8
	AVG.	433.9	413.7	333.6	409.6	436.2	414.6

TABLE A.5. *SRG FLEXURAL STRENGTH (PSI)*

SILICEOUS RIVER GRAVEL							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	255.9	316.7	418.3	225.6	365.0	409.9
	2	237.1	414.4	384.7	195.4	325.0	465.3
	3	234.9	390.9	367.6	225.6	300.0	409.9
	AVG.	242.6	374.0	390.2	215.5	330.0	428.4
3 DAYS	1	343.2	446.2	347.8	352.1	435.5	437.8
	2	414.6	485.0	357.2	432.7	425.0	482.9
	3	413.6	470.2	361.9	393.7	410.7	487.1
	AVG.	390.5	467.1	355.6	392.8	430.2	469.3
7 DAYS	1	409.4	420.0	313.2	493.2	475.1	553.7
	2	488.8	382.3	345.2	488.5	596.4	535.8
	3	375.4	446.8	366.6	501.5	529.0	533.4
	AVG.	424.5	416.3	341.7	494.4	533.5	541.0
28 DAYS	1	460.5	470.1	398.4	599.7	600.2	652.5
	2	465.3	417.9	406.5	605.9	493.2	700.4
	3	436.9	524.1	427.0	588.9	528.9	546.1
	AVG.	454.2	470.7	410.6	598.1	544.1	633.0
90 DAYS	1	514.3	540.6	489.9	741.6	648.3	732.1
	2	519.6	597.6	480.0	651.4	653.2	707.6
	3	519.2	533.2	528.3	677.9	643.3	781.9
	AVG.	517.7	557.1	499.4	690.3	648.3	740.5

TABLE A.6. ***LS FLEXURAL STRENGTH (PSI)***

CRUSHED LIMESTONE							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	352.5	401.6	465.2	308.7	437.2	400.7
	2	319.6	415.7	370.3	294.8	388.0	404.8
	3	386.2	426.7	446.7	323.8	411.7	418.7
	AVG.	352.8	414.7	427.4	309.1	412.3	408.1
3 DAYS	1	424.1	488.2	441.8	478.7	506.2	513.6
	2	521.1	483.9	456.0	460.6	523.8	523.2
	3	538.1	474.3	451.2	456.0	483.6	484.4
	AVG.	494.5	482.1	449.7	465.1	504.6	507.1
7 DAYS	1	551.8	383.1	479.2	544.3	550.9	442.3
	2	585.0	456.1	474.4	546.1	543.5	497.1
	3	542.7	416.0	370.3	563.1	519.9	469.7
	AVG.	559.8	418.4	441.3	551.2	538.1	469.7
28 DAYS	1	524.1	422.2	437.2	656.3	673.9	568.2
	2	583.0	475.2	484.8	676.5	612.2	547.2
	3	575.1	480.0	465.5	669.6	630.3	492.9
	AVG.	560.7	459.1	462.5	667.5	638.8	536.1
90 DAYS	1	602.5	620.6	612.3	721.9	618.9	593.7
	2	603.7	673.9	562.4	761.2	693.4	597.6
	3	585.3	—	566.4	773.9	700.5	617.4
	AVG.	597.2	647.3	580.4	752.3	670.9	602.9

TABLE A.7. *SRG THERMAL COEFFICIENT (10^{-6} IN./IN./°F)*

SILICEOUS RIVER GRAVEL							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	6.95	7.27	7.93	7.46	6.65	4.50
	2	7.88	7.96	7.55	7.75	5.70	7.27
	3	7.83	7.83	8.26	7.71	7.36	6.92
	AVG.	7.55	7.69	7.91	7.64	6.57	6.23
3 DAYS	1	8.88	8.02	6.49	6.84	8.16	7.04
	2	9.51	8.86	7.10	7.16	8.33	7.51
	3	8.45	8.42	7.32	6.89	–	7.27
	AVG.	8.95	8.43	6.97	6.96	8.24	7.27
7 DAYS	1	10.07	7.41	7.51	8.13	6.78	6.61
	2	8.96	7.21	7.92	8.06	10.50	6.87
	3	7.92	7.56	7.20	8.18	7.27	6.83
	AVG.	8.98	7.39	7.54	8.12	8.18	6.77
28 DAYS	1	8.48	8.17	8.82	8.58	7.72	8.50
	2	9.21	8.18	9.36	8.35	7.77	8.27
	3	8.91	8.21	9.36	9.48	7.58	7.90
	AVG.	8.87	8.18	9.18	8.80	7.69	8.22

TABLE A.8. ***LS THERMAL COEFFICIENT (10^{-6} IN./IN./°F)***

CRUSHED LIMESTONE							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
1 DAY	1	5.40	5.10	6.06	7.13	5.41	5.75
	2	4.38	4.89	6.66	7.42	5.53	5.83
	3	5.04	6.09	5.21	6.23	5.07	—
	AVG.	4.94	5.36	5.98	6.93	5.34	5.84
3 DAYS	1	5.32	5.86	5.70	4.62	4.58	5.33
	2	3.72	5.65	—	4.64	4.87	4.45
	3	5.59	6.77	—	2.82	5.19	5.59
	AVG.	4.88	6.02	5.70	4.03	4.88	5.12
7 DAYS	1	5.64	6.03	5.13	5.31	5.11	4.44
	2	6.03	6.12	5.20	5.49	5.16	4.78
	3	5.77	6.31	4.68	5.57	4.69	4.21
	AVG.	5.81	6.15	5.00	5.45	4.99	4.48
28 DAYS	1	6.38	5.86	6.30	5.26	5.68	7.85
	2	6.23	6.42	6.69	6.00	6.11	8.81
	3	6.47	6.60	6.71	6.15	6.12	7.67
	AVG.	6.36	6.29	6.57	5.80	5.97	8.11

TABLE A.9. ***MODULUS OF RUPTURE AT 7 DAYS (PSI)***

SILICEOUS RIVER GRAVEL							
MOISTURE CONDITION (% HUMIDITY)		40% REL. HUMIDITY			100% REL. HUMIDITY		
CURING TEMPERATURE (°F)		50°F	75°F	100°F	50°F	75°F	100°F
CURING TIME	TEST SAMPLE						
7 DAYS	1	498.2	534.3	400.7	552.7	717.5	649.9
	2	554.6	530.4	381.5	554.6	846.2	704.7
	3	496.5	483.9	413.6	646.7	591.5	693.2
	AVG.	516.4	516.2	398.6	584.7	718.4	682.6
CRUSHED LIMESTONE							
7 DAYS	1	589.6	531.1	507.4	618.4	628.1	575.3
	2	635.6	487.5	473.7	633.9	618.7	488.0
	3	580.5	588.3	548.1	601.8	635.6	576.9
	AVG.	601.9	535.6	509.8	618.0	627.5	546.7