PAP-745

Review of Deck and Slab Design for A.R. Bowman Dam

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Dr. B. Frank McCullough
Professor of Civil Engineering
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PREFACE

The reader of this report should recognize it reflects a parametric study for guidance in using CRCP for the crest roadway and the overtopping protective slab for the A.R. Bowman Dam. The report is not intended to serve as a design document or should it be considered with that in mind. In many instances, the information for quantifying the variables was not available so other sources were used or assumptions made. Thus, the report should only be used for guidance in making design decisions.

Originally, the report was intended to document the author's visit to Denver and a review of the design. In the "exit interview" meeting in Denver, Colorado on March 1, 1991, the USBR staff along with the author agreed to develop an expanded report to provide USBR with a more informative insight into the design and performance of CRCP. The review comments provided by the USBR in their letter of April 25, 1991 indicated the need for a formal report to be used as a reference document.

The author expresses thanks to the USBR staff assisting in the various phases of the report.

B. Frank McCullough

Table of Contents

Chapter 1
INTRODUCTION

The U.S. Bureau of Reclamation (USBR) is currently considering the use of a continuously reinforced concrete slab(CRCS) as the downstream face for the A.R. Bowman Dam near Bend, Oregon. This report pertains to a preliminary study of using CRCS on this facility. This introduction provides background information, the basic design hypothesis as well as the objective and scope of the study.

Background

Since the late 1940's, continuously reinforced concrete pavement (CRCP) has been used on highways, and since the 1960's, on airports (Refs. 1,2). The design and construction techniques for CRCP permits expansion and contraction joints to be eliminated, thus removing the major performance deficiency of jointed pavements since joints experience blowups, faulting, pumping, spalling, and consequently surface roughness.

With CRCP, the concept is to permit random cracking to occur, and through the design process control the frequency of cracking, the steel stress and crack width magnitudes. In contrast, with jointed pavements the concept is to control the location of the cracks through the use of contraction and expansion joints (Refs. 1,3). Thus, with CRCP the premise is to reduce the crack width, thereby, minimizing the flow of water through the crack and the slab to respond in a similar structural manner as an uncracked slab.

In the late 1960's, the Corps of Engineers also used CRCP as canal linings on many of their projects. Excellent performance was achieved for these conditions of flowing water.

Concrete facings are commonly used on the upstream face of rockfill dams and serve as the dam's primary seepage barrier. Recently, some upstream slab facings continuously reinforced in both directions have been used. Previously most upstream facings, however, were only continuously reinforced from toe to crest with the reinforcement in the transverse direction being discontinuous with contraction joints at about 50-foot spacings. Experience indicates concrete facings on rockfill dams, generally, have an excellent service record.

The overtopping protective CRCS proposed on the downstream face of A.R. Bowman Dam is similar in design and construction to upstream concrete facings. It is intended to use continuous reinforcement in both directions throughout the slab, thereby eliminating expansion and/or contraction joints. In addition, the perimeter of the slab will be fully restrained thus insuring the slab will act as a unit. The left edge and the base of the slab will be anchored into bedrock. The right edge will tie into the top of a gravity retaining wall. The top of the slab will tie into a CRC road along the entire dam crest length. The road is also restrained at each end in the same manner as the slab. In so far as known, this will be the first application of a CRC slab as embankment overtopping protection, therefore, case history information for this application is almost non-existent.

Design Hypothesis

A CRC slab has been selected to provide overtopping protection because of the hydraulic and structural advantages of continuous reinforcement (Ref. 4). The hydraulic benefits are controlling crack width and faulting, i.e., differential elevations on adjacent crack edges. Excessive seepage through the slab during overtopping can lead to accumulation of detrimental uplift pressures beneath the slab or saturate the downstream dam shell causing the embankment to become unstable. Therefore, the continuous reinforcement needs to control crack widths so that seepage through cracks is eliminated or minimized to the fullest extent possible.

Upstream concrete facings typically exhibit little or no seepage. However, upstream slabs usually are underlain by a rather impervious bedding material which can possibly camouflage the watertight effectiveness of the slab. Also, seepage through an upstream face slab does not present potential stability problems due to uplift as is the case for overtopping protection. The water tightness of the CRC overtopping protective slab is the primary line of defense to prevent seepage through the slab. To provide redundancy, the slab will be underlain by a compacted crushed rock drainage layer that drains to a line of aspirating weep holes toward the base of the slab. The gradation of the drainage blanket depends on estimating the anticipated seepage flow rates through the slab.

Controlling faulting at cracks is another significant advantage of continuous reinforcement since offsets that project into the flow can precipitate slab failure. Contraction and expansion joints are completely eliminated throughout the CRC slab to avoid introducing the seepage and offset potential associated with such joints.

Monolithic behavior is another advantage offered by a CRC slab and is considered an additional redundancy. External loads on the slab, other than the water pressure from the flowing water, are not anticipated. However, to a limited extent, the monolithic behavior will preserve the hydraulic integrity of the slab should localized uplift pressures or other flow related loads develop during overtopping. Mechanical splices are being considered, as opposed to lap splices, wherever possible to provide more reliable monolithic behavior and to facilitate construction.

Objectives

The A.R. Bowman Dam is a 245-foot-high embankment dam with a probable maximum flood (PMF) causing overtopping flows that are 20-feet deep at the dam crest. Therefore, overtopping protection will be provided by paving the entire downstream dam face with a continuously reinforced concrete slab (CRCS).

This study was conducted to achieve the following objectives (Ref. 4):

1. Provide guidance on slab thickness and reinforcement for the CRCP on the crest slab and the CRCS on the downstream face of the

overtopping protective slab to prevent various distress types that may occur and minimize water seepage through the cracks.

2. Determine the restraint force generated by the environmental factors of temperature and moisture that will be transferred to the retaining walls on the perimeter of the CRCS and the CRCP terminals.

3. Provide guidance concerning the subgrade and overtopping protective slab material beneath the CRCS and CRCP on the crest.

4. Provide guidance concerning the concrete mix design for the various slabs.

5. Provide comments and guidance on construction considerations that will ensure quality and successful performance.

Scope

This report provides the key recommendations outlined in the objectives and the remainder of the report is organized in four chapters as described in the following paragraphs.

Chapter 2 presents the study approach in terms of transferring highway and airport experience to the design of a CRCS for an overtopping protective slab to prevent or minimize the various distress types anticipated. In addition, the design concepts for CRCS are discussed, and the design input parameters are characterized.

Chapter 3 presents the concepts, input, and limiting criteria used in the design analysis. Whereas the previous chapter was a more global discussion, Chapter 3 quantifies the discussion for the A.R. Bowman Dam.

Chapter 4 presents the design recommendation for the concrete, steel, terminal conditions and other special conditions for both the overtopping protective slab and the crest slab. Basically, this chapter reports the results developed for the first four objectives of the study.

Chapter 5 presents the construction recommendations for steel placement, sequence of placement, concrete temperature control and vibration in accordance with the fifth objective.

In addition, several appendices are provided to present the various information needed to support the recommendations. Appendix A is specifically a summary description for the development of the design input.

Chapter 2
STUDY APPROACH

This chapter outlines the performance and design concepts that have evolved from approximately sixty years of experience with CRCP. The intent of this chapter is to provide the reader with the essential background for interpretation of the various recommendations presented later in Chapters 4 and 5.

The first section characterizes behaviour through providing a brief transfer of highway and airport experience associated with the performance of CRCP. This leads to a discussion of cracking followed by a brief section on modeling. The next section is a brief discussion of the distress types observed on highways and airports with a postulation of those that may be associated with the downstream face of a dam. Next, the design concepts for the thickness and reinforcement are followed by characterization of the key design parameters used as input into the design analysis.

Characterizing CRCP Behaviour

Extensive research has indicated that the cracking pattern is primarily a function of environmental stresses experienced by the pavement due to moisture and temperature changes during the life of the facility (Ref. 1). Although the external load stresses, e.g., wheel load, water pressure, etc, may have a slight influence on the cracking pattern, it is minor relative to the environmental factors especially if large temperature changes are anticipated.

Figure 2.1, consisting of four parts, may be used to describe the basic mechanisms associated with cracking of CRCP or CRCS. The top part of the figure is a plan view of a typical CRCP showing the crack spacing. Then moving from top to the bottom of the figure, the concrete movement, concrete stress and steel stress as a function of distance along the pavement for two temperature and shrinkage conditions. The dashed lines represent a more severe environmental condition.

Figure 2.1(a) is a plan view of a typical segment of a CRCP with two existing cracks and a potential third crack. The cracks are uncontrolled and generally form transverse to the direction of concrete placement, i.e., longitudinal direction, hence they are referred to as "transverse cracks". The reinforcing steel in the slab is, thus, designated "longitudinal" and "transverse." Longitudinal cracks are controlled by keeping the placement widths within specified limits or sawing longitudinal joints (controlled cracks) (Ref. 5). The longitudinal steel must be designed as a continuous unit, i.e., continuously reinforced, whereas the transverse steel is considered in discrete units, i.e., width of slab and designed as a jointed reinforced concrete pavement (JRCP). For large pavement widths, the transverse steel must be considered as a continuous unit (Ref. 6).

If we assume that a change in temperature or shrinkage occurs, then the concrete and steel components of the pavement will experience stress, strain and movement. Note in Figure 2.1(b) which conceptually portrays the slab

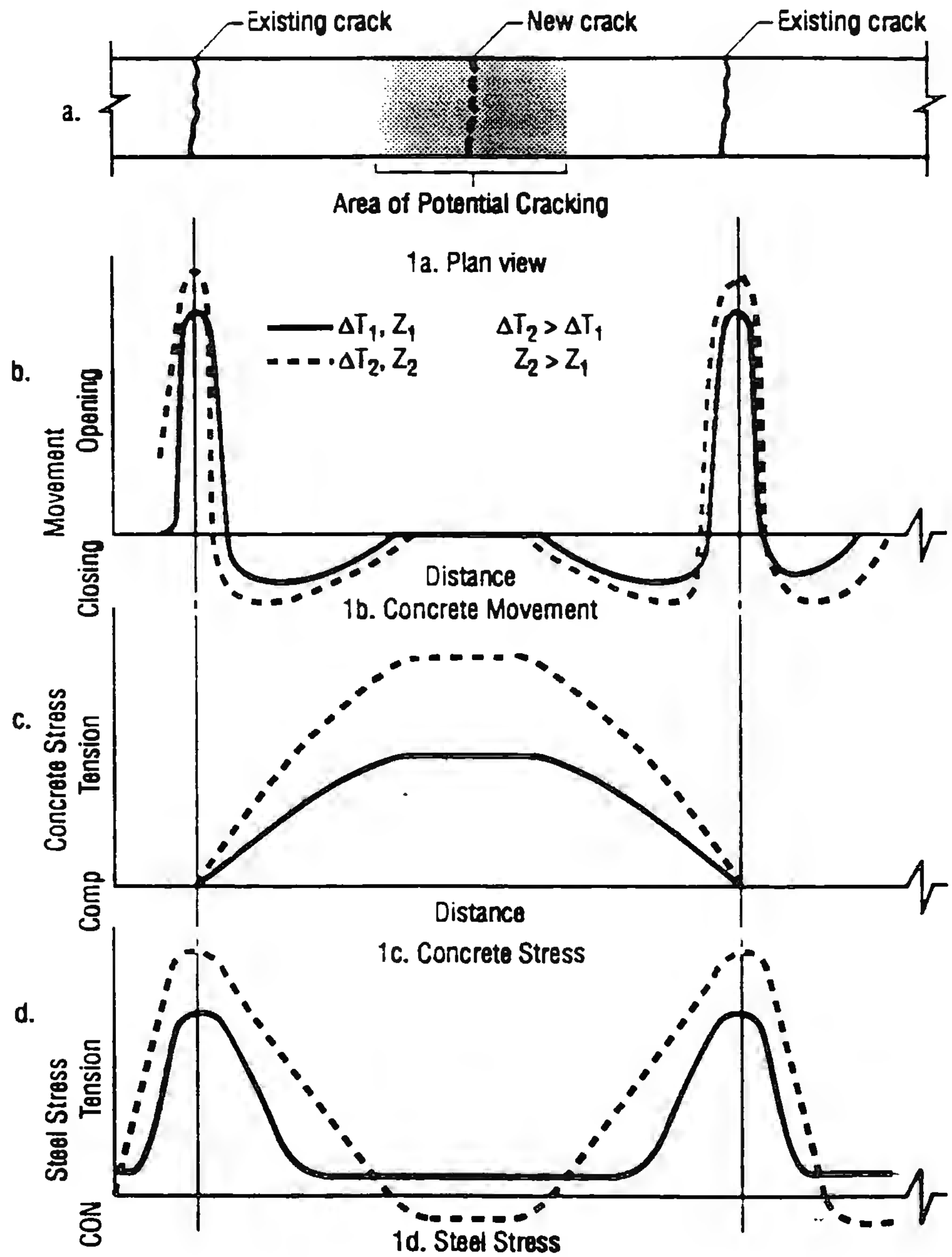


Figure. 2.1 Presents the distribution of stresses and movement in the concrete and steel longitudinally along a CRCP or CRSC. The effects of temperature changes are also illustrated as well as the area of potential cracking.

movement along the slab. Note there is a maximum movement at the crack with a decrease in relative movement toward the center of the slab. In the mid area of the slab, there may be zero movement.

Associated with the movement pattern is a corresponding relationship with concrete stress as illustrated in Figure 2.1(c). Note there is zero stress at the crack, as would be expected, and a maximum stress in the center of the slab where there is zero movement. With zero slab movement, then the concrete stress is the maximum value generated by temperature and moisture changes.

The steel stresses shown in Figure 2.1(d) have an opposite pattern of those illustrated for the concrete. The maximum steel stress occurs at the crack, while the stress in the center of the slab is smaller and may be tensile or compressive depending upon the conditions.

Crack Formation

If we change the temperature conditions or the concrete moisture(shrinkage), then the entire pattern presented will change. For example if a lower temperature is experienced, then the crack movement, concrete stress in the interior and the steel stress at the crack will increase as illustrated by the dashed lines in Figure 2.1. If the temperature decrease is large enough, then the stress may increase to a value that is greater than the concrete tensile strength as illustrated by the dashed lines in Figure 2.1(c). If this is the case, then cracking will be experienced in the approximate center of the slab as represented by the dashed line in Figure 2.1(a).

Figure 2.2 may be used to illustrate the overall formation of a crack pattern. Note the concrete strength is increasing with time as indicated by the solid line. Due to various temperature and moisture changes each day, the maximum concrete tensile stresses will vary. If the conditions are such that stress is greater than the strength, then additional cracks will form as marked with the notations on the figure. Thus, as illustrated on the Figure, four additional sets of cracks will occur in the original panel after the first day. This process will continue and cracking will occur until an equilibrium condition is reached, i.e., the concrete stress between the cracks does not exceed the strength, and additional cracking does not occur.

A review of Figures 2.1 and 2.2 indicates the critical features are the (1) steel stress at the crack (prevent steel rupture), (2) the movement at the crack (prevent water from seeping through the slab), (3) concrete strength (controls the cracking pattern) and (4) the spacing between cracks (establishes the balanced stress condition).

Modeling CRCP Behaviour

The modeling of these phenomenon is complex since the concrete properties such as strength and stiffness vary in time as well as the environmental conditions. The problem compounds since a substantial amount of interaction occurs among parameters. The CRCP-1-7 computer programs developed by the University of Texas for the SDHPT and NCHRP provides a complex computer model that simulates this cracking phenomena and can predict the information shown in Figures 2.1 and 2.2 for any set of environmental conditions, concrete properties, etc, (Refs. 1,7,8 and 9).

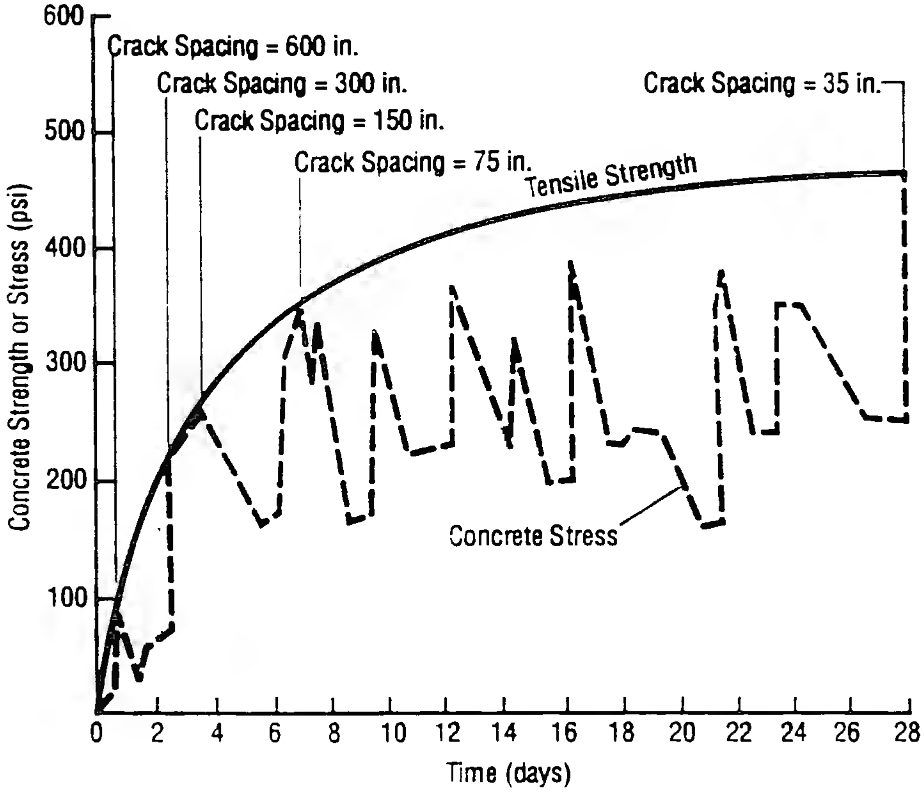


Figure. 2.2 A historical interaction of concrete tensile strength and stress resulting in the formation of cracks in a CRCP or CRCS over a 28 day period.

Distress Types

Once the behaviour pattern is understood, then an understanding of the distress manifestations follows. The inherent or stated objective(s) of any design procedure is to prevent or minimize the occurrence of distress. Thus, it is essential for the user of CRCP or CRCS to have an understanding of the CRCP distress types observed on highway and airport pavements as enumerated in Table 2.1 along with the methods used in minimizing or eliminating their occurrence.

Table 2.1 Distress Types Experienced With CRCP and Methods Used to Prevent Through Design, Specifications, and Construction Quality Control.

Distress Type	Method of Control *
1. Radial Cracking	Specs., Q.C.
2. Transverse Cracking	Specs., Q.C.
3. Pumping	Design-(Non Erosive Subbase)
4. Spalling	Specs, Q.C., Design (Max. Crack Width & Spacing)
5. Blowups	Q.C., Design (Crack Width)
6. Punchouts	Design (Max. Crack Width and Min. Spacing
7. Steel Rupture	Design (Max. Steel Stress)

*Q.C.- Construction Quality Control Specs-Specifications

The first two, radial and transverse cracking, were found to be related to initial construction problems. Radial cracking generally originates from a given point of a pavement similar to spokes from an axle. Generally, excavation at these points has found inadequately compacted concrete,i.e., honeycombing in the area beneath the steel. The transverse cracking generally occurs at an early age and has a narrow spacing in the range of one to two feet. These are normally associated with very weak concrete that may result from factors such as poorly vibrated concrete, high water cement ratio, etc. Both of these distress types show up with early age and if corrected do not reappear (Refs. 10,11,12). Furthermore, with good construction practices these types of distress can be eliminated.

The third item, pumping of the subbase or subgrade material, may be related to inadequate pavement thickness and/or material that goes into suspension in the presence of water and repeated loads. With a proper subbase and pavement design, this type of failure can be eliminated.

Crack spalling is a surface condition where a secondary crack starts at the surface and proceeds at an angle until it hits the primary crack and a small block of concrete comes out with the application of traffic loads. These blocks may be from 1/4 to 2 inches on side. Spalling may be related to inadequate concrete tensile strength or excessive deflections. With proper thickness design, specifications and construction control, spalling may be eliminated or minimized.

Blowups result from excessive compressive stresses, and the slab buckles as a column would, but in this case the slab moves upward. With

CRCP, this has not been a severe problem and has only been found with conditions where a crack opens too wide (greater than 0.1 in.) and incompressibles penetrated into the crack, and thus with expansion a blowup occurs. This condition generally occurs at construction joints where inadequate vibration is experienced on the next day's concrete placement (Ref. 10). Thus, the slabs have effective concrete thicknesses that are essentially different and eccentric loadings are experienced, hence a blowup may occur. Again with proper construction techniques these may be eliminated.

Punchouts involve a mechanism of closely spaced transverse cracks connected by parallel longitudinal cracks resulting from repeated load applications (Ref. 10). This results in a small block of concrete that does not have continuity with the rest of the pavement. The blocks become loose and eventually "whip-out" under repeated traffic load applications. This type of distress is prevented by maintaining a minimum crack spacing and providing adequate pavement thickness therecy, minimizing the conditions that result in punchouts (Ref. 13).

Steel rupture may occur when the steel is axially overstressed due to temperature and moisture changes similar to the process associated with the laboratory tensile test on steel. The mechanism may be supplemented by steel corrosion that occurs if a crack is too wide and water penetrates to the level of steel. As a result, rusting will occur thus, reducing the steel's effective cross-sectional area (Ref. 14).

<u>Summary</u>

The preceding sections have provided a basis for characterizing and modeling the behaviour of CRCP on highways and airports, i.e., the first step in design. Certain behaviour patterns lead to distress that the designer must minimize or eliminate. Obviously, these concepts are applicable to the crest slab, but now a decision must be made as to those applicable to the concrete overtopping protective slab (CRCS).

Chapter 3
DESIGN ANALYSIS

With the concepts defined in the previous chapter, a rational design analysis can be made for the A.R. Bowman Dam slabs. Thus, this chapter deals specifically with the design aspects of the slab thickness and reinforcement. First, the specific distress types of concern are defined, then the design procedure is discussed. The last section discusses the characterization of the design inputs.

Applicable Distress Types

For the crest roadway, all of the previously described distress types are applicable, but with the CRCS the key distress types are blowups and steel ruptures. The buckling or blowup potential may be compounded at the interface between slab segments of different orientation, i.e., the slabs intersect at an angle such as at the retaining walls along the spillway.

The excessive crack width is also detrimental since water penetrating through the crack into the drainage layer below may result in pressures which in turn result in lifting the overtopping slab off of its support thereby endangering the dam. Thus crack width is also a critical criterion. Another unique feature that may be associated with the overtopping slab is the reflection cracking in the slab that may be due to volumetric changes of adjacent and more massive concrete structures with small reinforcement percentages, e.g., the gravity retaining walls in the crest structure and the thickened area in the slab. Since the steel percentages in the more massive structures may be smaller, they may experience wider crack widths and place additional stresses on the adjacent slab.

Design Concepts

Essentially, the designer must prevent the distress types previously described from occurring as a result of stresses developed by load, temperature and moisture changes. Basically, the load variable is taken into account by thickness design, and the temperature and moisture changes by the reinforcement design. The CRCP programs previously described permit an interaction of these stress producing factors, and thus the stress history is predicted. By using proper limiting criteria for the critical condition as shown in Figure 2.1, i.e., steel stress, crack width and crack spacing, a slab may be properly designed (Ref. 13).

Since the CRCP computer program only simulates the pavement responses for a given set of conditions, the predictions are only applicable for these specific set of design inputs. These are then measured against the design criteria to establish an acceptable level. Figure 3.1 conceptually presents the interaction required to arrive at an optimum level, for example, the critical values during the pavement life of steel stress, crack spacing and crack width are plotted in terms of the cross-sectional area of the reinforcement. The limiting criteria for each of these are then applied and a satisfactory design is achieved. It is apparent from this graph that there is no unique solution, but a

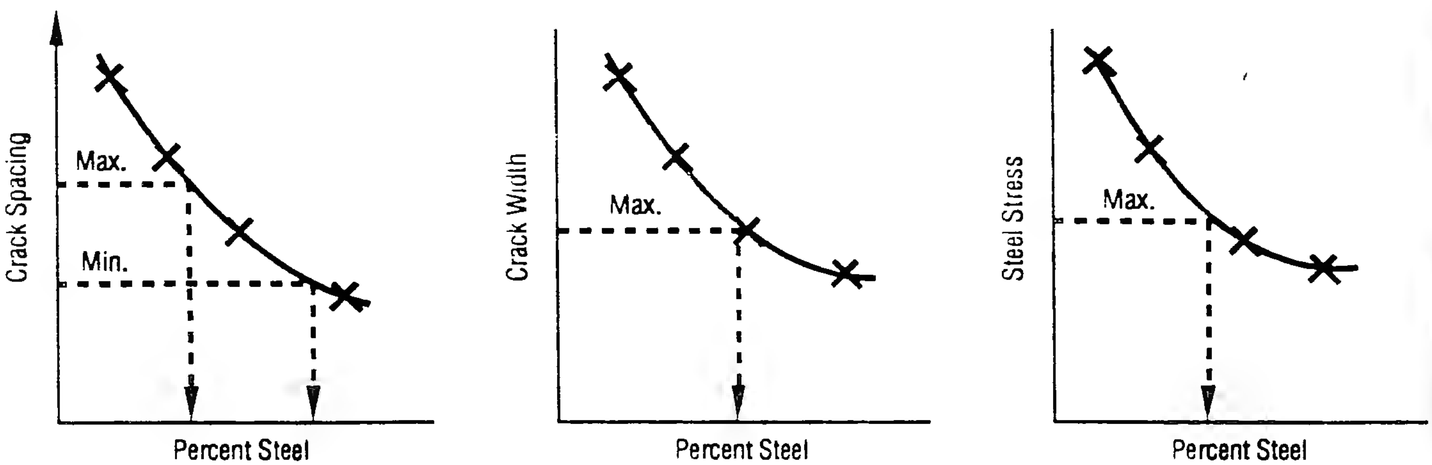


Figure 3.1.Interactions of limiting design criteria for CRCP as used in a design analysis.

number of acceptable solutions. By applying other criteria, an optimum solution may be developed for a specific set of conditions. It should be recognized that when the bar size, thickness or any one of the input parameters is changed, then the entire graph is changed as well. Thus, to explore the range of conditions for this problem over 40 solutions were made.

Characterization of the Design Parameters

Table 3.1 provides a summary of the design input for the crest and overtopping protective slab. For a more detailed description of how each design input parameter was derived, the reader is referred to Appendix A which provides a detailed description of the sources of data or value judgements used in developing the input (Refs. 15-20). Note in Table 3.1 that the input data are very similar for the overtopping protective slab and crest slab but differ for certain parameters such as slab thickness and the method the load is applied.

The design parameters may be characterized in terms of concrete properties, steel properties, temperature conditions, external load and pavement geometry.

TABLE 3.1 Design Input Summary Presenting Magnitude of Each Variable for the CRCP and CRCS Slabs (Refs. 15-20).

	Variable	Overtopping Slab	Crest Slab
(1)	Conc. Tens. Strength, psi	420.	420.
(2)	Drying Shrinkage	.00045	.00045
(3)	Bar Number	6,7,8	6,7,8
(4)	Slab Modulus, psi	3,880,000.	3,880,000.
(5)	Subbase Modulus, psi	60000.	60000.
(6)	Roadbed Modulus, psi	15000.	15000.
(7)	Single Axle Load	20 psi	18,000 lb
(8)	Age Load Applied,Days	120	28
(9)	Number of Load Applications	N/A	20,000,000
(10)	Coarse Aggregate Type	Basalt	Basalt
(11)	Daily Temp.Diff.@ 28 days,^{o}F	35.	35.
(12)	Temperature of Concrete Set,^{o}F	77	77
(13)	Annual Temp.Diff,^{o}F	110.	110.
(14)	Slab Thickness,in	12.0	8.0
(15)	Subbase Thickness,in	12.0	6.0
(16)	Subbase Type	untreated	untreated

Limiting Criteria

Table 3.2 defines the limiting value of crack spacing, crack width and steel stress. These values are used as conceptually shown in Figure 3.1 to develop the acceptable range of reinforcement steel. In the following paragraph, each of these factors are discussed in more detail.

The value of eight feet for maximum crack spacing has been set to prevent spalling at cracks (Ref. 13) and is primarily applicable to the crest slab. The minimum value has been set at three feet to prevent punchout type failures, and also provide adequate bond development length between the cracks (Ref.

9). The maximum crack width of 0.040 in. represents the value at the lowest anticipated temperature (-33^{o}F) during the life of the facility. This corresponds to a crack width of 0.023 in. at 32^{o}F which will prevent flow of the water thru the slab (Ref. 1). This is the slab temperature and represents the controlling value since any water in the crack would freeze, thus acting as a sealant. The maximum steel stress is set at 60,000 psi, the yield point of Grade 60 Steel.

TABLE 3.2 Limiting Criteria for the Crest and Spillway Slabs

Design Constraint	Magnitude
Maximum Crack Spacing,ft.	8.0
Minimum Crack Spacing,ft.	3.0
Maximum Crack Width @ -33^{o}F,in.	0.040
Maximum Crack Width @ 32^{o}F	0.023
Maximum Steel Stress,psi	60000.

Chapter 4
DESIGN RECOMMENDATIONS

This section summarizes the design recommendations developed using the concepts, parameters and limiting criteria discussed in Chapter 3 in connection with the CRCP computer program. The design recommendations are made in terms of the portland cement concrete slab, steel reinforcement, terminal conditions and other special conditions as outlined by the USBR.

Concrete Slab

This section provides guidance to the selection of the thicknesses for the slab and subbase as well as recommendations for the mix design. This information is presented in Table 4.1 for both the overtopping slab and the crest roadway slab.

Table 4.1
Summary of Pavement and Subbase Design and Construction Recommendations

FACTORS	OVERTOPPING SLAB	CREST SLAB
Slab Thickness, in.	12	8
Subbase Thickness, in.	12	6
Gradation	Open	Open
Cement Factor, sacks/CY	5	5 1/2 - 6
Compressive Strength, psi	4000	4000
Max. Size Coarse Agg., in.	2	1 1/2
Type II Cement/Fly Ash	80/20	80/20
Air Entrainment, %	4-5	4-5

The slab thickness for the crest roadway was designed based on the inputs of traffic and other variables while the thickness for the overtopping slab was based on recommendations provided by the USBR. The overtopping slab will not be subjected to loadings other than the water flow which only produces minimal loading in the slab as shown in Appendix A. Thus, the slab thickness will be controlled by other factors such as constructibility. The subbase thickness was based on design concepts for the crest roadway, whereas a value of 12 inches was used for the overtopping slab. A change in this dimension will not impact on the other properties thus, its thickness should be controlled by other factors such as drainage. It is recommended that an open graded subbase be used that will permit the flow of water through the subbase. The Corps of Engineers criteria for gradation in moving from the roadbed material to the subbase should be used to prevent the intrusion of fines from other layers into the coarser layer and clogging the drainage capabilities (Refs. 21,22).

As to the concrete mix design the cement factor for the crest roadway is based on providing adequate durability and a similar value was recommended for the overtopping slab. The cement factor along with the minimum compressive strength of 4000psi at 28 days should provide adequate durability for the slab. In addition, the air entrainment values will also provide durability especially if salt is used for deicing in connection with the crest roadway.

The maximum size of the coarse aggregate is adequate especially considering the bar spacing sizes that are recommended in the next section. A slightly larger maximum coarse aggregate is permitted for the overtopping slab since a larger spacing between bars will be present.

The use of the fly ash with the cement will reduce the heat of hydration and, thus, preventing some problems especially early random cracking from occurring.

Steel Configuration

Table 4.2 provides several combinations of bar size and spacing that meet the limiting criteria for crack spacing, crack width and steel stress (see Appendix B for graphical plots). It was pointed out previously there is no unique solution and every combination presented in Table 4.2 meets the criteria therefore, other factors may be used in selecting the optimum design. In general, it would probably be better from a long term construction standpoint to use the combination with the larger bar sizes, since there would be considerably more space between bars, hence reducing the probability of honeycombing that can lead to failures and blowups. The disadvantage of this recommendation is the larger the bar size the more difficult for the crew placing the steel unless mechanized methods are used. The predicted value for crack spacing, crack width and steel stress are also shown in the Table and as previously noted meet the criteria outlined in Chapter 3.

Table 4.2 Reinforcement Combinations Meeting Limiting Criteria at the Minimum Temperature for the Overtopping Protective Slab.

Bar Size (Number)	Percent Steel %	Bar Spacing (in.)	No. of Bars Per 24 ft.	Crack Spacing (ft.)	Crack Width (in.)	Steel Stress (psi)
#6	.473	8.00	37	3.31	.0392	55823
	.504	7.50	39	3.01	.0357	53102
#7	.545	9.50	31	3.29	.0389	50006
	.574	9.00	33	3.02	.0358	47779
#8	.592	11.50	26	3.50	.0413	46743
	.618	11.00	27	3.25	.0384	44951
	.646	10.50	28	3.00	.0355	43098

Table 4.3 provides several combinations of steel design for the crest roadway. These values are slightly different from those for the overtopping slab since a different slab thickness is used; furthermore, with crest roadway, loading will be experienced at an early age which in turn impacts on the crack spacing. In this case, the more optimum design is probably with the number six bars.

Table 4.3 Reinforcement Combinations Meeting Limiting Criteria at Minimum Temperature for the Crest Slab.

Bar Size (Number)	Percent Steel %	Bar Spacing (in.)	No. of Bars Per 24 ft.	Crack Spacing (ft.)	Crack Width (in.)	Steel Stress (psi)
#6	.499	11.50	26	3.21	.0390	54693
	.521	11.00	27	3.01	.0365	52809
#7	.563	14.00	21	3.28	.0398	49724
	.583	13.50	22	3.10	.0376	48223
	.604	13.00	23	2.92	.0354	46677
#8	.629	16.50	18	3.31	.0401	45205
	.667	15.50	19	2.98	.0361	42723

It should be recognized in both Tables 4.2 and 4.3 that values are derived based on the input as outlinea in Chapter 3. It should also be recognized that the most sensitive parameters are the material properties especially the concrete strength, and these have been assumed on the basis of experience. Perhaps a more detailed parameter study should be run with different aggregate types and concrete properties to fully bracket the range of conditions that may be expenenced at the A.R. Bowman Dam site.

Terminal Conditions

Table 4.4 provides an estimate of the restraint forces generated by the environmental factors of temperature and moisture,i.e., concrete shrinkage experienced at the end of the crest roadway and around the periphery of the overtopping slab. The table provides the restraint force in terms of kips per linear feet of joint. The values of force were computed using the average of the stress values presented in Tables 4.2 and 4.3. Since the slab is in a balanced stress condition, the full effects of friction, steel restraint, etc. are reflected. Values are expressed for botn the expansion and contraction conaitions since a greater temperature differential wiil be experienced for a drop in temperature than an increase in temperature. All the values shown in the table assume full restraint of the slab. Any movement will reduce the value of the force so these may be considered as maximum vaiues in designing the retaining walls, etc.

Table 4.4 Force Magnitudes Required at Edges or Terminal to Retain CRCS Continuity.

Force Mechanism	Force (a) Overtopping Slab	Force (a) Crest Slab
Expansion	24.2	16.1
Contraction	38.1	25.3

(a) Force in kips / linear foot

For the contraction condition one should recognize this will be for a frozen soil condition where the temperature is approximately 30 degrees below zero, hence the frozen soil will provide considerable restraint. Therefore, the most critical design condition may be the expansion conditions.

Special Conditions

One major special condition of particular concern is the possibility of reflective cracking in slabs placed adjacent to one of the massive concrete structures such as the retaining walls. Since some of these massive concrete structures may have lower reinforcement percentages, there is concern that the crack width will be greater thereby inducing a high level of stress in the adjacent slab. Although consideration should be given to this, there are two factors that probably negate the possibility of detrimental reflective cracking from adjacent concrete placements. Since the major part of the stress in the concrete is due to volume change resulting from temperature and moisture changes, the relative values for the two placement types must be considered.

First, considering the concrete shrinkage, Figure 4.1 shows the impact of the volume to surface area ratio on concrete shrinkage (Refs. 19, 21). Thus, for the mass concrete placement this value will be relatively high when compared to the overtopping or crest roadway slabs. Thus, the shrinkage in the adjacent mass placement may be much lower as indicated in the Figure, and consequently the reinforcement is adequate. The same observation is also true for temperature changes since the greater the mass of the concrete, the smaller the overall the average temperature differential experienced by the mass of concrete at a daily cycle or an annual temperature cycle. Hence, the stresses would be much lower and again the reinforcement is probably an adequate range. Therefore, if the cracks are small, then reflective cracks into an adjacent slab will not be detrimental.

For the slab connections at longitudinal construction joints, the transverse steel should extend through the joint and into the adjacent slab (Refs. 1,10). At least one third of the bars should extend twenty four times the bar diameter, another one third of the bars should extend twenty four diameters plus one foot and the other one third should extend twenty four bar diameters plus two feet. On highway construction, two foot intervals are used, but since there will be less stress at the longitudinal construction joint for the reasons

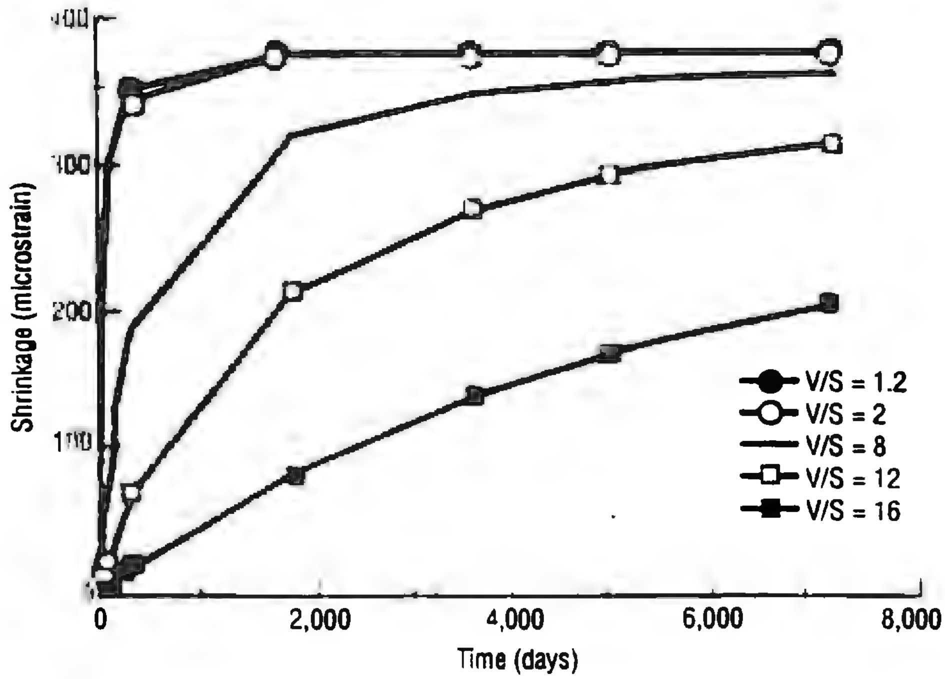


Figure 4.1 Hansen and Mattock's equation for CAT = Dolomite illustrating the effect of the volume to surface area ration on concrete shrinkage.

explained in the construction recommendations, a smaller value may be used than with airport or highway pavements.

For sharp angles of slab connection in the vertical, a more detrimental condition may exist since an uplift force may be experienced. As recommended as a minimum that these construction joints have the same stagger spacing as noted previously. In addition, where the intersection angle is greater than five degrees consideration should be given to adding additional bars at the longitudinal construction joint to prevent a blowup from occurring, i.e., conventional practice is to double the steel. In addition, vertical tie downs capable of sustaining the upward component of the force values shown in Table 4.4 should be considered.

<u>Sensitivity Study</u>

Changing any of the input value in Table 3.1 will alter the output reflected in Tables 4.2 and 4.3 with the sensitivity of some variables being greater than others. One interest area for the USBR is the effect of reducing the crack width criteria to 0.025 in. and 0.003 in. at -33°F and 32°F, respectively. Sample runs of the computer program found a satisfactory solution could not be obtained unless the minimum crack spacing criteria was reduced to two feet. These results for the overtopping protective slab are presented in the following Table 4.5. It is pointed out to the reader that the equilibrium forces shown in Table 4.4 do not change with reinforcement criteria, but will change with thickness.

Table 4.5 Reinforcement Combinations Meeting A Reduced Crack Width and Minimum Crack Spacing Criteria

Bar Size (Number)	Percent Steel (%)	Bar Spacing (in.)	No. of Bars Per 24 Ft.	Crack Spacing (ft.)	Crack Width (in.)	Steel Stress (psi)
#6	.626	6.00	49	2.11	.0239	43902.
#7	.686	7.50	39	2.21	.0250	40481.
#8	.750	9.00	33	2.28	.0257	37154.

Another area of interest was the impact of slab thickness, and Table 4.6 presents the results of increasing the slab thickness from 12 in. shown in Table 3.1 to 14 in.

Table 4.6 Reinforcement Combinations Meeting A Change in Slab Thickness Input

Bar Size (number)	Percent Steel (%)	Bar Spacing (in.)	No. of Bars per 24 ft.	Crack Spacing (ft.)	Crack Width (in.)	Steel Stress (psi)
#6	.496	6.50	45	3.55	.0398	57252.
	.537	6.00	49	3.15	.0353	53666.
#7	.552	8.00	37	3.72	.0416	52714.
	.588	7.50	39	3.35	.0375	49878.
#8	.643	9.00	33	3.50	.0391	46147.
	.679	8.50	34	3.16	.0354	43742.

Chapter 5
CONSTRUCTION RECOMMENDATIONS

This chapter covers various recommendations for construction activities that have been derived from over the years of CRCP placement. Recommendations were generally arrived at from successful experience or correcting poorly performing slabs, thus in some cases have no basic design reason other than improving the performance.

<u>Steel Placement</u>

1. It is recommended that the longitudinal steel going from top to bottom of the dam be placed at the mid-depth. In general, since the large spacings are being used, it is not necessary to resort to two level placement in order to provide adequate vibration. Improved performance has been noted with the steel at the center and also prevents an eccentric loading condition from occurring in the slab due to volumetric stresses.

2. All laps of the steel should be a value of 24 diameters. For the direction from crest to toe of the dam, the lap should be staggered so that for any given placement only one-third of the laps may be found in two running feet of slab. Thus, for all the bars to be lapped, at least six feet will be required along the pavement. This staggered lap placing has been found to eliminate the occurrence of wide cracking that may be associated with lap failures when all the laps are placed in a line, i.e., a point of weakness.

3. For the steel in the horizontal direction, the lap pattern as described for the longitudinal construction joint in Chapter 4 may be used. A slightly smaller distance is accepted here, since the overall stresses in the horizontal direction will be less due to the sequence of slab placement.

4. The bar placement may be by a preset method, tube or depressing to the proper depth although studies have shown that all three methods results in the steel being near the center of the slab. It is recommended that the preset method be given serious consideration since concrete is being placed on a much steeper slope than normally experienced with highway or airport pavements.

5. For any transverse (horizontal) joints that must be inserted in a placement from the toe to the crest, the lap stagger pattern outlined above should be fully utilized with the minimum distance of the first one third of the bars being 24 diameters from the construction joint.

6. All laps should be tied. For the longitudinal construction joints, consideration may be given to mechanical screw devices for laps. These will reduce the problems experienced during construction.

<u>Sequence of Slab Placement</u>

Since the slabs will be constructed from the toe to the crest in 50 foot widths generally, an advantage is gained for the steel in the horizontal direction. In the horizontal direction, the concrete shrinkage will not be initially restrained hence, considerable amount of its impact will be dissipated before the adjacent slab is tied to it. Since the stress is due to both the concrete shrinkage and temperature then a substantial part of the strain stresses is dissipated. Considering this factor, the following recommendations are made concerning the sequence of the slab placement:

1. The slabs be placed in alternate units, i.e., a unit placed then a void space then another unit. Thus, the stresses will be reduced substantially in the horizontal direction.

2. Slabs adjacent to retaining walls, etc., should be placed first, thus a good tie at a critical longitudinal joint is made without the slab being subjected to the full restraint stresses. By the time it is all tied together, the concrete would have cured out and the weakest point of design has achieved its maximum strength.

Concrete Temperature Control

Performance studies of CRCP especially during the early life has revealed that the maximum heat of hydration occurs simultaneously with the maximum daily air temperature thus the slab's temperatures are raised to a maximum level. For these conditions, the first day experiences a substantial temperature differential, hence a random cracking occurs in the slab since an adequate bond has not developed (Ref. 24). Cracks that occur in these conditions generally produce problems at a later date whereas those that occur uniformly seldom ever experience any detrimental problems (Ref. 25). Hence, control should be exercised to ensure the concrete temperature at placement and during concrete hydration does not exceed 110^{o}F.

Concrete Vibration

Good concrete consolidation or vibration will eliminate many potential problems from occurring, hence the inspection in this area should be maximized. At all construction joints, vibration should be done by hand vibrators to eliminate the possibility of honeycombing from occurring at these locations (Ref. 10). If honeycombing, i.e., weakest point, occurs at the construction joint, then a condition exists where the maximum stress occurs at the location of minimum strength; thereby increasing the probability of a failure condition.

An adequate head of concrete (4-6 inches) over the internal vibrators will enhance the effectiveness. Vibration should be controlled by the forward movement of the spreader so that vibration automatically ceases when the forward movement is stopped. In addition, special care must be taken to attain consolidation of the concrete under and around bar laps to avoid segregation and honeycombing.

To ensure adequate vibration is being achieved perhaps a test slab should be made at the angle of placement to check out the vibration techniques. The slab could then cored extensively to ascertain the effectiveness of the vibration.

Appendix A

Developing Inputs for Spillway and Crest Slabs

DEVELOPING CRCP-2 INPUT DATA

Load

With 20ft. of water over the crest of the dam, various levels of pressure (p) are reported along the face. A document was presented in terms of head in ft (h).

$$p = h\ (\text{ft}) \cdot 62.4\ \#\,/\,\text{ft}^3\,/\,144\ \text{in}^2\,/\,\text{ft}^2$$

$$= 0.433 \cdot h\ \text{psi}$$

%	h	h *	p(psi)
(1) 50	35	28.7	12.8
(2) 95	59.3	53.0	23
(3) 99	72.3	66.0	28.6

* reflects correction of 6.3 ft for reference

Concrete Properties

Age (days)	Elastic Modulus [a] psi x 10^6	Tensile [b] strength psi	Shrinkage [c]
2	1.83		
7	2.67	240	
28	3.88	420	
90	4.44	520	
365	5.21	600	0.00045

(a) furnished by USBR
(b) ft = 0.1 fc - fc from F23 of USBR
(c) Not available

Pavement Stiffness

E_{pvt} = 5.2×10^6 psi   USBR
$E_{Subbase}$ = 60,000 psi – Estimated for good crushed material
$E_{Roadbed}$ = 30,000 psi – Estimated for backfill on Dam
$D_{subbase}$ = 10 in.

Subbase – Crushed Stone

ΔT_{D-1}	–	Avg. difference of daily max. & min. temp. - vary with seasons
ΔT_{A-1}	–	Max. annual temp. differential
ΔT_{A-2}	–	Annual temp. diff. of avg. max. temp. & min. observed
ΔT_{A-3}	–	Avg. annual temp. difference

The above descriptions provides several methods for computing the annual temperature differential. The worst case scenario is proposed for use.

Daily

	Summer				Spring		
	T_{max}	T_{min}	ΔT		T_{max}	T_{min}	ΔT
June				March			
	75	42	33		56	25	31
	78	40	38		55	27	28
	79	42	37		57	27	30
July				April			
	83	43	40		60	27	33
	85	43	42		62	29	33
	88	44	44		63	31	32
	86	43	43		65	33	32
	86	42	44		66	34	32
August				May			
	85	40	45		70	35	35
	82	39	43		73	37	36
	81	38	43		72	37	35
	79	35	44		74	40	34
Mean Value		41.3° F			64.4	31.8	32.8
Std. Dev.		3.63			6.73	4.88	2.69
85 Percentile		44° F					35° F

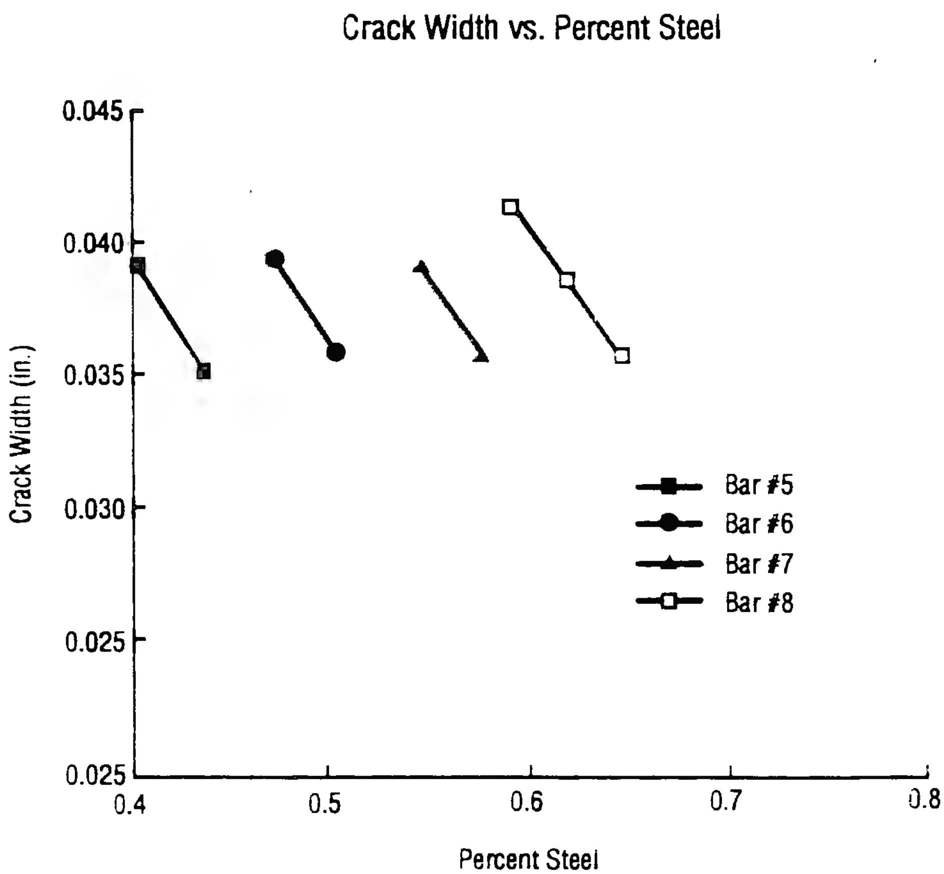
ΔT_{A-1} = 104 – (-33) = 137° F

ΔT_{A-2} = 88 - (-33) = 121° F

ΔT_{A-3} = 88 - 18 = 70° F

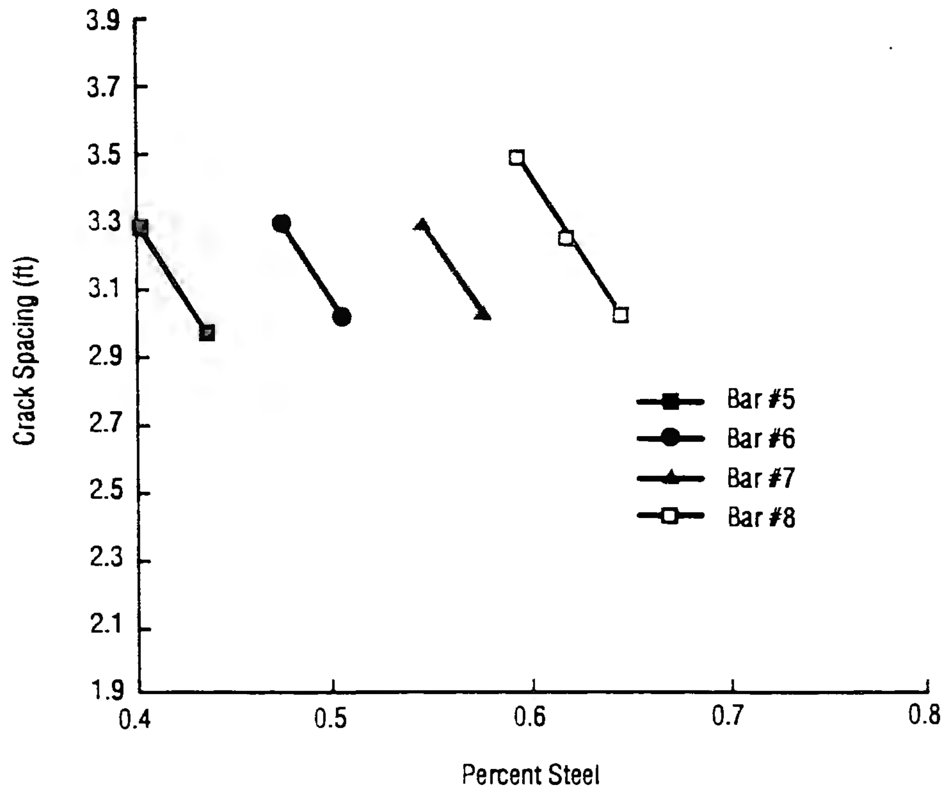
Appendix B

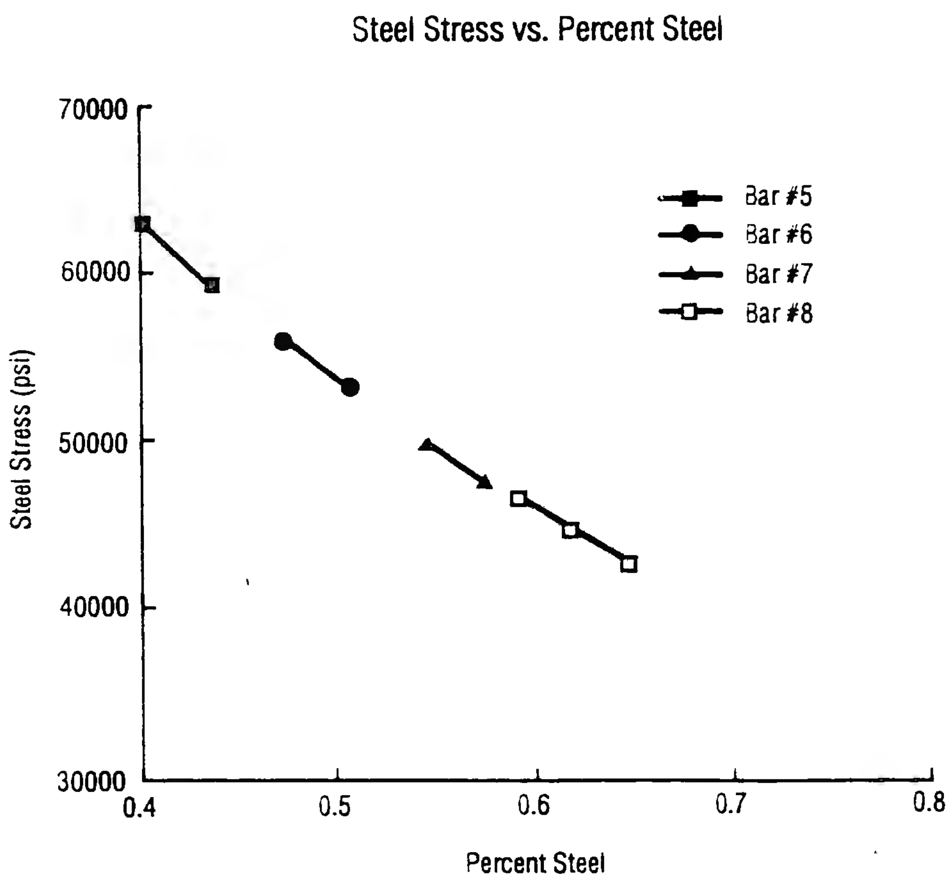
Plots of Limiting Criteria As a Function of the Percent Steel for Overtopping Protective Slab



Crack Width vs. Percent Steel
0.045
0.040
0.035
0.030
0.025
0.025
Crack Width (in.)
0.4
0.5
0.6
0.7
0.8
Percent Steel
Bar #5
Bar #6
Bar #7
Bar #8

Crack Spacing vs. Percent Steel





Steel Stress vs. Percent Steel
70000
60000
50000
40000
30000
Steel Stress (psi)
0.4
0.5
0.6
0.7
0.8
Percent Steel
Bar #5
Bar #6
Bar #7
Bar #8

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