Material Property Requirements for Zero Maintenance of Continuously Reinforced Concrete Pavements

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This study involved the (a) selection of models suitable for predicting important distresses that previously have been observed on continuously reinforced concrete pavement (CRCP) and (b) determination of a set of material properties that will provide satisfactory performance for 20 years without maintenance and satisfactory performance for the next 10-20 years with normal maintenance. To accomplish this goal, the most important distresses occurring in CRCPs and the material properties that affect those distresses were identified. Mathematical models to predict those distresses by using the identified material properties were selected. A range of values for each material property was selected and resulting distresses were predicted. The distresses studied included fatigue cracking, punchouts, crack spalling, steel rupture, and low-temperature and shrinkage cracking. The mathematical models selected for the analysis were ELSYM5 for modeling fatigue cracking and CRCP-2 for modeling low-temperature and shrinkage cracking. Punchouts, crack spalling, and steel rupture were incorporated into the analysis of low-temperature and shrinkage cracking. Input values were selected for each model, and ranges of the material properties affecting distress were identified and used in the analyses. The results for each study are discussed, and practical criteria are cited to evaluate the level of material properties required to provide zero-maintenance performance. A set of material properties is identified, and the trade-offs on material properties are discussed.

For several years, the Federal Highway Administration (FHWA) has pursued multiple research studies aimed at producing premium pavement structures for heavily traveled highways. The objective of these efforts has been to develop pavement structures that will be maintenance-free for 20 years and will require only routine maintenance for 10-20 years thereafter.

The research reported here is drawn from a portion of the FHWA-sponsored research project Material Property Requirements for Zero-Maintenance Pavements. The overall goal of this project is the identification of material properties that will provide optimal performance in flexible, rigid, and composite zero-maintenance or premium pavements.

The research methodology used to study the influence of material properties on distress in premium highway pavements involved the following steps:

1. Identify significant distresses,
2. Review and select models to predict distress,
3. Determine the effects of varying material properties while holding other inputs constant, and
4. Identify ranges of material properties to minimize distress.

The results of the first two steps noted above were previously published in an interim report (1). This paper contains the findings of the study of continuously reinforced concrete pavement (CRCP) related to steps 3 and 4 above.

SIGNIFICANT DISTRESSES

CRCP distresses, whose frequency of occurrence and severity significantly influence serviceability, were derived from field studies reported by McCullough and others (2) and Darter and Barenberg (3). These distresses are crack spalling, fatigue cracking, low-temperature and shrinkage cracking, punchouts, and steel rupture.

SELECTED DISTRESS MODELS

Analytical models were available to study the effect of material properties on fatigue cracking, low-temperature cracking, and shrinkage cracking. The other important distresses noted above were incorporated into the low-temperature and shrinkage analyses.

Fatigue Cracking

Elastic-layer theory was used to study fatigue cracking because the individual layers can be characterized and their separate effects on pavement response can be studied. Stresses and strains predicted by elastic-layer theory are distributed with depth in a more realistic fashion than for plate models and are more economical to use. The primary limitations of the elastic-layer theory are its inability to define any horizontal boundaries and its inability to simulate directly the existence of stiffness variations such as cracks or voids. Despite these limitations, elastic-layer theory, as a pavement structural model, is very useful for a comprehensive study of the various layer materials.

Since a variety of elastic-layer models exist, the various available computer codes were compared (4) and the program most suited to project needs was selected. Based on that study, ELSYM5 was judged to be the best overall elastic-layer computer code for this study.

Low-Temperature and Shrinkage Cracking

The dimensional changes in a continuously reinforced concrete pavement, caused by drying shrinkage of the concrete and temperature variation after curing, were investigated by McCullough and others (2), and a design method that used the CRCP-1 program was developed and subsequently improved by Ma (5) to consider stresses imposed by wheel loads. This model was selected for the analysis of CRCP.

MODEL INPUTS

The use of these analytical models required the development of a consistent set of input values for each variable that occurs in the models.

Common Inputs

Several input factors are common to both models and are independent of the pavement material properties. These factors are the environment in which the pavement occurs, the traffic level to which the pavement is subjected, and the thickness of the pavement layers. Other factors, or inputs, depend on the individual models.

Environmental effects were incorporated by using environmental characteristics of four Interstate highway sections located in distinctly different environmental regions. The sections chosen for use in this study and the environmental regions represented were the following:

1. Wet freeze (WF)--Cold climate with generally high humidity and abundant precipitation, I-80 in Illinois (AASHO Road Test);
2. Dry freeze (DF)--Cold climate with low humidity and little rainfall, I-80 N in northern Utah (near Snowville);
3. Wet, no freeze (WNF)--Relatively warm climate with high humidity and abundant rainfall, I-20 in Florida (near Madison); and
4. Dry, no freeze (DNF)--Warm climate with low humidity and little rainfall, I-20 in Texas (between Midland and Odessa).

The traffic level was derived from past data on heavily trafficked rigid pavements. Traffic information reported in Darter and Barenberg (3) indicated that some rigid pavements are experiencing over 2 million 80-kN (18-kip)

Table 1. Levels of material property inputs varied in fatigue cracking analysis.

	Level		
Material Property	Low	Intermediate	High
E for PCC surface (psi 000 000s)	24	36	98
f_r for PCC corresponding to E (psi)	3860	5860	6890
E for subbase (psi 000s)	103	3450	6890
Poisson's ratio corresponding to E for subbase	0.40	0.30	0.20
Fatigue potential[a]: $N_{18k} = C_1(f_r/\sigma_t)^{C_2}$	C_1 = 18 000 C_2 = 3.9	–	C_1 = 100 000 C_2 = 3.9

Note: 1 psi = 6.89 kPa.

[a]N_{18k} = number of 18-kip (80-kN) ESAL applications to produce class 3 and class 4 cracking; f_r = PCC modulus of rupture; σ_t = tensile stress in bottom of PCC layer.

equivalent single-axle loads (ESALs) annually, with 1-1.25 million relatively common. Therefore, 2 million 80-kN (18-kip) ESALs per year was selected for the traffic level that a zero-maintenance rigid pavement should carry.

The AASHO design procedure was used as a preliminary design for rigid pavements. A small study that used two types of subbase, three subbase thicknesses, and their corresponding moduli was performed by using a load transfer value of 2.2. This study was followed by an analysis that used the serviceability index equation developed by Darter and Barenberg (6) for zero-maintenance rigid pavements. Since Darter's procedure is also based on data collected at the AASHO Road Test, but over a longer period of observation, it was used to select the following cross-section design: portland cement concrete (PCC) surface thickness = 279 mm (11 in) and subbase thickness = 203 mm (8 in).

Specific Inputs to Distress Models

The results of this study were greatly affected by the ranges of input values selected for individual material properties due to possible interactions of the material properties. The ranges selected are representative of the variability of conventional materials under field conditions.

The important material properties affecting fatigue cracking were identified as (a) modulus of elasticity of the PCC surface layer, (b) modulus of elasticity of the subbase, and (c) fatigue potential of the PCC layer (1). Three levels were selected for the modulus of elasticity of both the PCC surface layer and the subbase layer (Table 1). The flexural strength of the concrete corresponded to the modulus of elasticity.

The fatigue potential of the PCC layer is estimated from the fatigue equation that relates the allowable number of load applications to the ratio of induced stress to flexural strength. The fatigue relations were based on field performance of in-service pavements. The curve used in this study was developed from interior stresses predicted by elastic-layer theory and the number of repetitions required to produce class 3 and class 4 cracking at the AASHO Road Test. This curve was modified for edge stresses and the high-low range introduced through variation in the coefficients.

The value inputs held constant throughout this analysis are listed below. (The input values are presented in the form used during research; therefore, no conversion to SI units appears in this section.)

Input Variable	Value
Number of loads	2
Load magnitude (kips)	9
Distance between loads (ft)	6
Tire pressure (psi)	75
Modulus of elasticity of subgrade layer (10^3 psi)	20
Subgrade Poisson's ratio	0.45
PCC surface thickness (in)	11
PCC surface Poisson's ratio	0.15
Subbase thickness (in)	8
Stress adjustment factor	1.23

The interior stress adjustment factor, which accounts for transverse traffic distribution and fatigue damage, represents the increase in stress from the interior condition to a position approximately 406 mm (16 in) from the edge of the pavement that was found to accumulate greater fatigue damage as a result of increased load applications.

The effects of thermal coefficient, shrinkage, and tensile strength of the PCC surface layer were of primary interest in this study. Values for each material property combination are shown below:

	Level	
Material Property	Low	High
Ultimate PCC tensile strength (indirect) (psi)	400	800
PCC thermal coefficient (10^{-6}/°F)	4.5	9.0
Ultimate concrete shrinkage (10^{-6})	200	800

The constant input values for low-temperature and shrinkage cracking analysis are summarized here.

1. Steel properties have these values--Reinforcement type = deformed bars; bar diameter = 0.625 in (#5); yield stress = 60 ksi; elastic modulus = 29.0 x 10^6 psi; and thermal coefficient = 6.0 x 10^{-6} in/in/°F.
2. Concrete properties have these values--Slab thickness = 11 in; unit weight = 150 lb/ft³ (W); ratio of tensile and flexural strength$^{1.5}$ = 0.86 (STRNMUL); and the PCC modulus of elasticity = 33.0 $(W)^{1.5}$ x (tensile strength)/(STRNMUL x 7.5).
3. Slab-base friction characteristics used in low-temperature and shrinkage cracking analysis are as follows:

Concrete Movement (in)	Friction Stress (lb/in²)
0.0	0.0
0.01	0.21
0.10	0.63
0.15	0.80
0.20	0.94
0.30	0.97

4. Design low-temperature factors are (a) WF, 25°F; (b) DF, 29°F; (c) WNF, 56°F; and (d) DNF, 46°F.
5. Temperature data are as follows: curing temperature = 80°F, first 28-day temperature drop = 20°F, and the time until minimum temperature = 28 days.
6. Wheel-load considerations are (a) wheel load = 9000 lb, (b) effective tire radius = 6.18 in, (c) modulus of subgrade reaction = 600 lb/in³, and (d) time of load application = 28th day.
7. Iteration and tolerance control involve the maximum number of iterations, which is 60, and a relative closure tolerance of 5 percent.

The indicated relation between the concrete tensile strength and modulus of elasticity is internal in the program. The curing temperature is used to calculate the temperature drop for each environmental zone.

DISCUSSION OF DISTRESS STUDIES

Fatigue Cracking

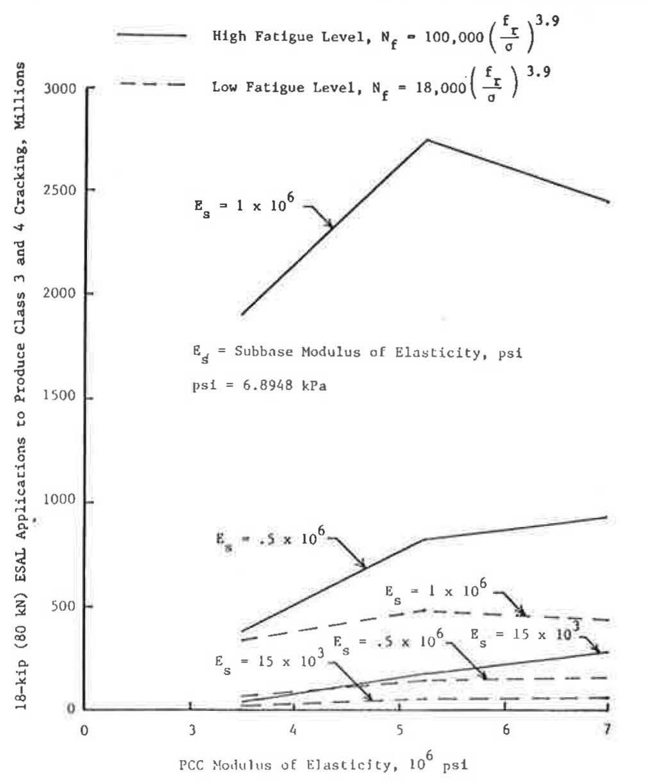
The results of the fatigue cracking study are summarized in Table 2. The influence of the modulus of elasticity of the

Table 2. Summary of fatigue cracking analysis.

PCC Modulus of Elasticity (10^6 psi)	Subbase Modulus of Elasticity (10^3 psi)	Predicted 18-kip ESAL Applications to Produce Class 3 and Class 4 Cracking (000 000s)	
		Low Fatigue Potential[a]	High Fatigue Potential[b]
3.5	15	8.25[c]	45.8
	500	68.4	380
	1000	342	1900
5.25	15	31.9[c]	177
	500	149	827
	1000	493	2740
7.0	15	49	272
	500	166	924
	1000	438	2440

Note: 1 psi = 6.89 kPa.
[a]Low fatigue, $N_f = 18\,000\,(f_r/\sigma_t)^{3.9}$.
[b]High fatigue, $N_f = 100\,000\,(f_r/\sigma_t)^{3.9}$.
[c]Does not satisfy zero-maintenance requirements of 40 million 18-kip (80-kN) ESAL applications.

Figure 1. Influence of concrete modulus of elasticity on CRCP fatigue cracking.

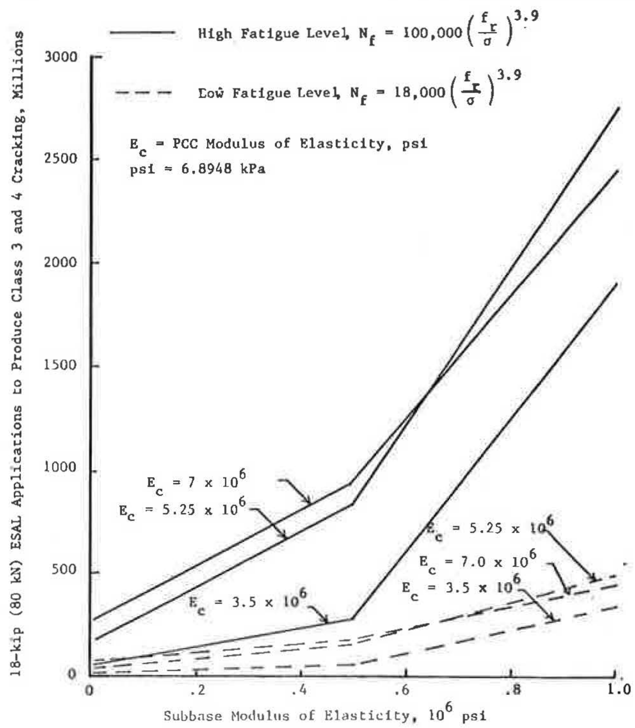


surface and subbase is illustrated in Figures 1 and 2.

The failure criterion used to determine the zero-maintenance potential was the ability of the pavement to carry 40 million 80-kN (18-kip) ESAL applications without developing class 3 or class 4 cracking. Treybig and others (7) found that the structural response of a pavement changed significantly when class 3 and class 4 cracking occurred; at that point, routine maintenance was required.

Increasing the subbase modulus, modulus of the concrete, and fatigue characteristics of the concrete increased the number of loads required to produce fatigue cracking. For the ranges investigated, increases of the subbase modulus produced larger improvements in fatigue performance than did increases in the modulus of elasticity of the concrete. For example, for the low fatigue level and subbase modulus, the traffic carried increased by about six times as the

Figure 2. Influence of subbase modulus of elasticity on CRCP fatigue cracking.



concrete modulus varied from 24 to 48×10^6 kPa (3.5 to 7.0×10^6 psi); however, for the low concrete modulus, the traffic carried increased by a factor of almost 40 over the range of subbase moduli. Changes in the traffic carried at other concrete moduli were not as large but did vary by a factor of 8 or more. Because the values of subbase moduli used in this analysis are quite common, efforts to improve the subbase moduli should be given prime consideration.

For the values used in this study, the effect of varying the fatigue relation on traffic was larger than the effect of varying the modulus of elasticity of the concrete. As shown in Figure 2, the fatigue life peaked at a concrete modulus of elasticity at 36.2×10^6 kPa (5.25×10^6 psi). This is probably not a significant observation because modulus and the fatigue characteristics are interrelated.

Conventional concrete offers excellent resistance to fatigue cracking, and it is quite capable of providing zero-maintenance service. All combinations of material properties except two carried traffic for 20 years without the class 3 and class 4 cracking. In fact, the traffic carried was substantially above that required before cracking. One missing element in the analysis was an evaluation of the effect of layer thickness that may be more significant than material properties. Zero-maintenance performance may not be produced at smaller thicknesses, even if the properties are significantly improved. Nevertheless, conventional materials exhibit material properties sufficient to meet the zero-maintenance fatigue criterion.

Punchouts, Crack Spalling, and Steel Rupture

The effects of material properties on the occurrence of punchouts, crack spalling, and steel rupture were studied by establishing limits on various pavement response parameters predicted by the CRCP-2 program, such as crack spacing and crack width, and by investigating the effects of material properties on these parameters. The limiting responses were established by theoretical consideration of the distress mechanisms and correlation with observed performance.

Punchouts

Transverse cracks in CRCP develop as a result of restrained volume change. As the transverse crack spacing becomes smaller and load transfer across the transverse crack deteriorates, the pavement slab begins to respond to load as a transverse beam instead of a longitudinal beam. This subsequently causes short longitudinal cracks to occur within the transverse beam sections. These, in turn, produce small blocks of pavement that eventually lead to punchouts. Thus, punchouts can be prevented by limiting the crack spacing to ensure that the transverse stresses are less than the longitudinal stresses.

An analysis of crack spacing and the concrete stresses in the transverse and longitudinal direction was performed by using the SLAB-49 computer program (8,9). To determine a reasonable limit on crack spacing to minimize the occurrence of punchouts, a minimum load transfer across the transverse cracks of 50 percent was assumed. This corresponds to an interpolated value of crack spacing of 1.34 m (4.4 ft). This limit is slightly less than the 1.52-m (5-ft) optimum crack spacing determined by Majidzadeh (10) and the 1.52-m minimum suggested by McCullough and others (2) based on field observations of in-service pavements. Therefore, theoretically, crack spacings greater than 1.34 m (4.4 ft) should minimize the occurrence of punchouts on CRCP. However, this fact assumes that the concrete is in full contact with the subbase.

Crack Spalling

The primary causes of spalling on CRCP are believed to be (a) entrapment of road debris in cracks that causes stress concentration when the cracks close as temperature increases and (b) combined shear and tensile stress at joints or cracks due to a combination of horizontal temperature loading and vertical traffic loading. However, based on a laboratory study, McCullough and others (2) concluded that CRCP spalling caused by road-debris entrapment was relatively insignificant but that combined horizontal and vertical forces were the major causes of spalling. Because crack width and degree of spalling are both functions of horizontal stress, crack width and spalling are related. In a diagnostic study based on condition-survey data from Texas (2), crack widths and the occurrence of spalling were measured in the field. Results of these studies indicate that spalling was more prevalent at larger crack widths. A mean crack width of 0.54 mm (0.021 in) was reported for the spalled sections and 0.45 mm (0.0176 in) for the nonspalled sections. Cracks with widths of less than 0.51 mm (0.02 in) exhibited no spalling.

To control spalling, Ma and others (11) established a maximum allowable crack width of 0.61 mm (0.024 in). Since the crack widths were measured during the summer at relatively high temperatures, it was necessary to calculate the corresponding width at a lower temperature. The expected crack widths corresponding to the above maximum allowable crack width at the low temperatures for each environmental zone used as a criterion to limit spalling are DF, 1.02 mm (0.040 in); WF, 1.04 mm (0.041 in); DNF, 0.86 mm (0.034 in); and WNF, 0.76 mm (0.030 in).

Steel Rupture

Steel rupture may be prevented by limiting the calculated steel stress from the low-temperature and shrinkage analysis. Because the CRCP-2 program is based on a linearly elastic steel model, limiting the steel stress to the yield strength of the steel served as a conservative limit against steel rupture.

Low-Temperature and Shrinkage Cracking

Combinations of material property levels investigated in low-temperature and shrinkage cracking analysis and the constant input values, both described earlier in this paper, formed the basis of this analysis.

Initial Study

This analysis was performed in a sequential order beginning with the high and low values of tensile strength and 0.6 percent steel. The properties varied were the concrete thermal coefficient, ultimate concrete shrinkage, and tensile strength. The calculated crack spacing and crack widths for the WF zone are shown in Figure 3. Relations for the other environmental zones were similar.

The maximum desirable crack spacing for a CRCP, determined from field observations, has been set at 2.44 mm (8 ft) (2,10). Crack spacings greater than 2.44 mm (8 ft) produce detrimentally wide cracks. The relation between steel stress and concrete strength for each environmental zone is shown in Figure 4. Shaded areas of Figure 4 represent stress that meets zero-maintenance requirements. For high-strength concrete, the yield strength in the steel was exceeded.

Concrete strength had the greatest effect on the predicted CRCP response parameters. At the low-strength level, shrinkage exhibited greater effect than the thermal coefficient. At the high-strength level, thermal coefficient produced a greater change in the predicted response than did shrinkage. In general, crack spacing increased as the concrete shrinkage and thermal coefficient decreased and as the concrete strength increased.

Crack width increased as the concrete shrinkage and thermal coefficients decreased and as the concrete strength increased. This result may seem contrary to the reasoning that increasing concrete shrinkage and thermal coefficient cause the concrete to contract more, thus creating a wider crack. This phenomenon occurs because of the relation between crack width and crack spacing.

To demonstrate the effect of these variables on crack width alone, without the effect of crack spacing, the crack widths were divided by the corresponding crack spacings and plotted against the ultimate concrete strength. The resulting relation for the DF zone (Figure 5), which is typical of the other environmental zones, indicates that the crack width per foot of slab increased as the concrete shrinkage and thermal coefficients increased. Figure 5 also shows the insensitivity of predicted crack width to concrete strength.

One other effect on crack spacing was detected in the analysis of steel stresses. As shown in Figure 4, the predicted steel stress at the crack was directly proportional to concrete strength. For a constant crack spacing, steel stress was expected to (a) decrease as the concrete shrinkage and thermal coefficients decreased and (b) increase as the concrete strength increased. This result occurred as concrete shrinkage decreased for the high concrete thermal coefficient but not for the low value. These results occur because of the interaction of crack width and crack spacing as exhibited in Figure 3. In reviewing the results, the interactive effects of crack spacing, crack width, and steel stress cannot be optimized separately to achieve a zero-maintenance pavement. This analysis indicated that crack spacing was a very influential response parameter for a CRCP. The effects of material properties were not separable from their effects on crack width and steel stress.

In general, the material properties at the high level produced values of response parameters outside the recommended range developed from field studies to ensure zero-maintenance service. The combination of high concrete strength, low shrinkage, and low thermal coefficient produced crack spacings greater than 6.1 m (20 ft). The combinations of high concrete strength, high thermal coefficient, and both low and high levels of concrete shrinkage resulted in equal crack spacings in all zones. However, the crack widths and steel stresses for these combinations with high shrinkage greatly exceeded the recommended limits in all zones. The crack widths and steel stresses for the combination with low concrete

Figure 3. Effects of concrete strength and shrinkage characteristics on crack spacing and crack width in the wet-freeze zone.

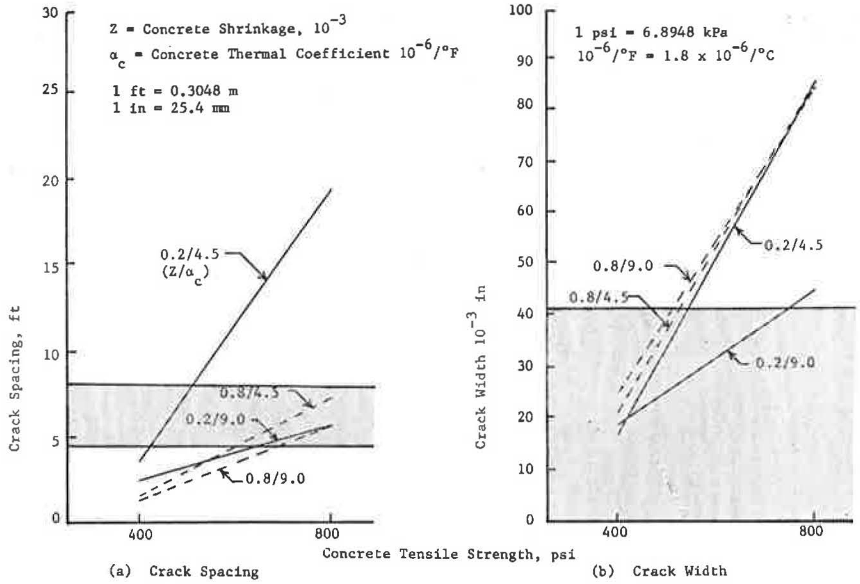


Figure 4. Effects on concrete strength, shrinkage, and thermal coefficients on steel stress at the crack for CRCP.

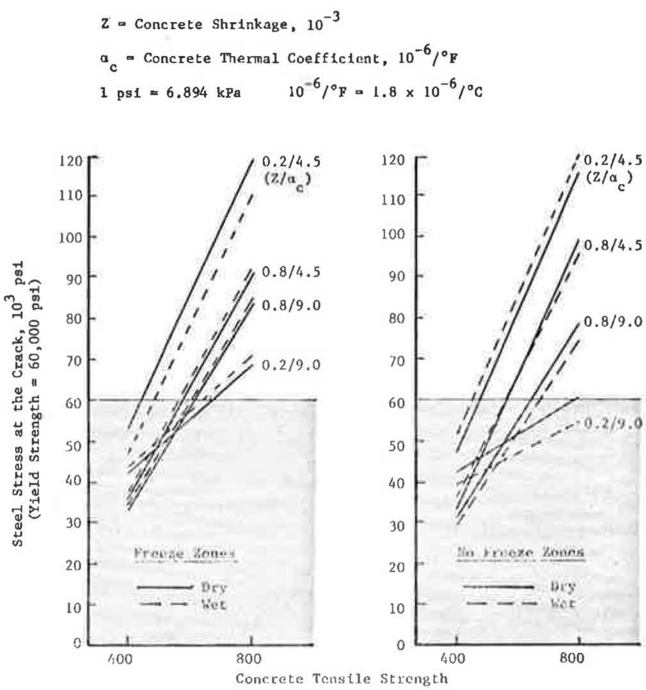
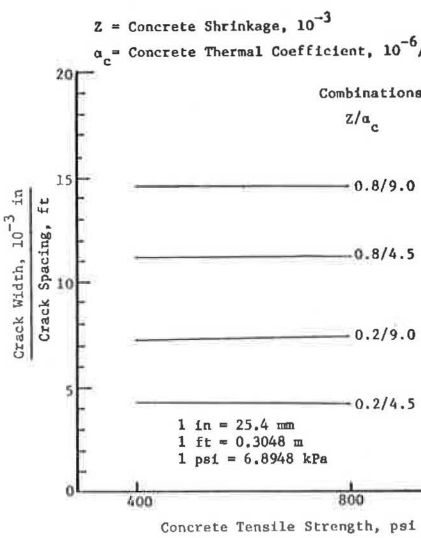


Figure 5. Effects of material property variables on crack width by crack spacing for the dry-freeze zone.



shrinkage fell within the recommended limits in all but the WF zone.

For the combination with all material properties at the low level, the response criteria were satisfied in all but the WF zone. For this zone, the crack spacing was closer than the recommended limit to prevent punchouts.

Second-Stage Study

A second set of material property combinations included steel reinforcement at 0.75 and 0.9 percent, ultimate concrete tensile strengths between 3.1 and 4.1 MPa (450 and 600 psi), and a steel thermal coefficient between 9.0 and 10.8×10^{-6}/°C (5.0 and 6.0×10^{-6}/°F).

The crack spacing, crack width, and steel stress decreased as the percentage of steel increased. Inspection of the results for higher percentages of steel showed that the calculated crack spacings were smaller than the punchout-limiting criterion of 1.54 m (4.4 ft). The concrete strength was high enough to resist the higher stresses produced by the increased restraint of the additional steel.

The results from this two-stage analysis that came closest to meeting the established zero-maintenance distress criteria are plotted in Figures 6 and 7. These results were all for a constant 0.6 percent steel. No material property combination investigated produced acceptable results for all four zones. Combination 1, which included low strength, low thermal coefficient, and low shrinkage satisfied the required criteria in all but the WF zone. In that zone, the crack spacing dropped below the required level. Combination 2 (Figure 6) met the crack-spacing and crack-width criteria for only the two freeze zones. The steel stresses for combination 2 exceeded yield strength in all zones. Combination 3 met all three criteria only in the no-freeze zones, while both crack width and steel stress exceeded the required limits in the freeze zones.

Figure 6. Summary of crack spacing and crack width with regions of acceptability for zero-maintenance pavements.

Material Properties	POINT NUMBERS		
	1	2	3
Tensile Strength, psi	400	550	800
Thermal Coefficient/°F	4.5×10^{-6}	5.0×10^{-6}	9.0×10^{-6}
Concrete Shrinkage	.0002	.0004	.0002

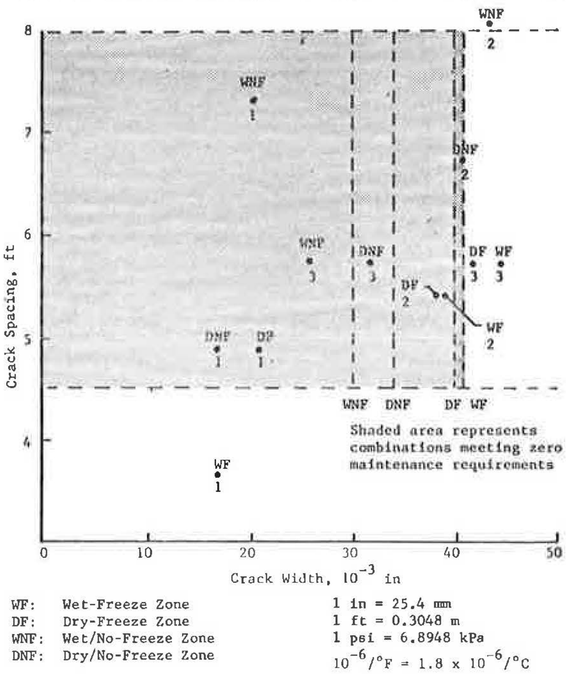
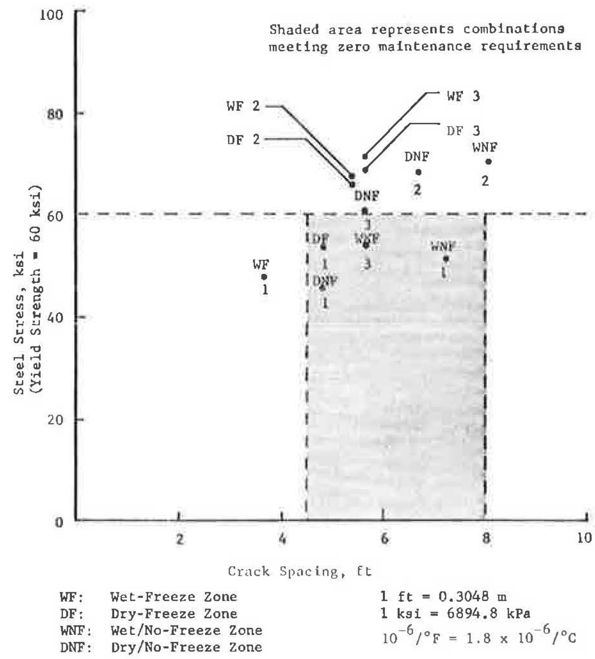


Figure 7. Summary of crack spacing and steel stresses with regions of acceptability for zero-maintenance pavements.

Material Properties	POINT NUMBERS		
	1	2	3
Tensile Strength, psi	400	550	800
Thermal Coefficient/°F	4.5×10^{-6}	5.0×10^{-6}	9.0×10^{-6}
Concrete Shrinkage	.0002	.0004	.0002



Several observations were made from these results. Combinations of material properties with a shrinkage greater than about 0.0004 yielded results outside the acceptable range. At low concrete shrinkage, the response parameters for the freeze regions approached the acceptable range when concrete strength was between 2.76 and 3.45 MPa (400 and 500 psi). The high-strength concrete was better suited for application in the no-freeze zones than in the freeze zones.

A subsequent analysis involved considering the effect on the response variables produced by changes in a material property. The response parameters used in this analysis were crack spacing and crack width. The effects on these two response parameters produced by the changes in the various material properties and percentage of steel were compared. The change in concrete strength produced the greatest change in both crack spacing and crack width. The second most influential material property was the thermal coefficient of concrete. Changes in both crack spacing and crack width were inversely related to changes in this coefficient. The concrete shrinkage was inversely related to crack spacing and directly related to crack width. Changes in steel percentage produced greater changes in both crack width and crack spacing than did changes in either the thermal coefficient or shrinkage. Crack spacing and crack width were inversely related to steel percentage.

The order of influence of material properties investigated, starting with the most influential, was concrete strength, thermal coefficient, and shrinkage. A CRCP with 0.6 percent steel combined with a concrete of sufficient thickness and a tensile strength of 3.45-3.79 MPa (500-550 psi), a thermal coefficient of $8.1\text{-}9.0 \times 10^{-6}$/°C ($4.5\text{-}5.0 \times 10^{-6}$/°F), and a concrete shrinkage equal to or less than 0.0004 will provide performance compatible with zero-maintenance requirements.

In this study, concrete tensile strengths greater than 3.79 MPa (550 psi) produced steel stresses in excess of yield.

CONCLUSION

The CRCP structure investigated in this study exhibited excellent resistance to fatigue cracking. Only the combinations with (a) low subbase modulus of elasticity and low fatigue and (b) both low and intermediate concrete modulus of elasticity did not meet the zero-maintenance requirements. Other combinations carried traffic for 20 years without the occurrence of class 3 and class 4 cracking. This analysis showed that the subbase modulus of elasticity had a more significant effect on increased fatigue life than did improvements in PCC.

The low-temperature and shrinkage cracking analysis indicated that concrete tensile strength was the most influential material property in this distress. The thermal coefficient and shrinkage of the concrete were less influential than the amount of steel reinforcement, which is a design factor rather than a material property.

The set of material properties derived from the various distress-model studies that were identified as minimizing distress are noted below:

1. PCC modulus of elasticity = 34 million kPa (5.0 million psi) < E_c < 41 million kPa (6 million psi);
2. PCC indirect tensile strength = 3445 kPa (500 psi) < f_t < 3790 kPa (550 psi);
3. PCC thermal coefficient = 8.1×10^{-6}/°C < α_c < 9.0×10^{-6}/°C (4.5×10^{-6}/°F) < α_c < 5.0×10^{-6}/°F);
4. Ultimate concrete shrinkage = Z < 0.004; and
5. Subbase modulus of elasticity = E_s > 6.9 million kPa (1.0 million psi).

The relationship among these properties forms a set of properties consistent with conventional PCC. The level of the tensile strength that corresponds to the modulus of elasticity of PCC determined from the fatigue analysis falls within the bounds determined from the low-temperature and shrinkage analysis. The lower bound on the tensile strength selected from the low-temperature and shrinkage analysis is greater than the minimum tensile strength indicated by the examination of spalling information. Thus, the set of material properties listed above may be considered an optimal combination of material properties for a premium CRCP designed with the pavement components that are described in the section of this paper that deals with specific inputs to distress models.

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Limiting Criteria for the Design of Continuously Reinforced Concrete Pavements

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The primary factors to consider in the thickness and reinforcement design for continuously reinforced concrete pavements (CRCPs) are the structural response variables—crack spacing, crack width, and maximum steel stress. They perform an important role in the outcome of the pavement's performance and can be related to the major distresses common to CRCPs. This paper describes the design-limiting criteria for these structural responses. Previous investigations of the design criteria are reviewed, and the most recently developed analytical models are studied. The basic procedures used to establish design criteria include an examination of the major distresses, such as punchout, spalling, and steel rupture, and a study of correlations between these distresses and the corresponding structural responses at appropriate levels. The procedure for use of the limiting criteria in CRCP design is outlined.

The design concept for continuously reinforced concrete pavement (CRCP) is to force cracks to form at relatively close intervals, thus controlling the tightness of the crack to provide good load transfer and prevent excessive water percolation. The frequency of cracks and the final crack width depend on a complex interaction of environmental variables, material properties, and magnitudes of applied loads. Initial cracks in the CRCP are primarily caused by critical stresses induced by the initial temperature drop and drying shrinkage of the concrete. Additional cracks may develop during application of an external load when the combined stresses of the internal and external forces exceed the concrete tensile strength. Close to 90 percent of the transverse cracks occur within one month after construction. The crack pattern will eventually reach a stabilized condition when the pavement has experienced the minimum temperature during the cold season and when most of the drying shrinkage in the concrete has occurred.

The CRCP-2 computer model (1) was designed to fully simulate the mechanistic behavior of the CRCP with respect to time and load. The model predicts the structural responses of the CRCP to environmental load and static external load from the time that initial cracks form to the time when the volumetric changes of the CRCP have stabilized. The final crack spacing, crack width, and steel stress appear to strongly influence the performance of the