PERFORMANCE OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS IN INDIANA

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ABSTRACT

Continuously reinforced concrete pavements (CRCP) represent a relatively recent development in concrete pavement technology. Their widespread use in recent years has met with mixed success. This study reports the findings of a detailed field investigation of in-service CRC pavements in Indiana. One of the objectives of this evaluation was to recommend design and construction guidelines that would help to improve CRCP performance.

The study consisted of the following sequential steps:

1. Pavement condition surveys.
2. Detailed field testing and evaluation.
3. Laboratory testing program.

A statistical analysis of the condition survey data showed that subbase type, methods of steel placement and steel fabrication, concrete slump, and age of pavement since opened to traffic were significant contributors to pavement performance.

Based on these results, a field investigation of CRC pavements was conducted. In this study, 31 test sections, each 1000 ft. in length, were used. As far as possible, each test section represented a unique combination of material, construction, traffic, and temperature variables.

The results of the detailed field tests and laboratory studies showed that failures in CRCP are a function of a number of interacting variables. Generally, higher pavement deflection, decreased pavement stiffness, wider crack widths, and erratic non-uniform crack patterns were associated with failed pavement condition. The support conditions under CRCP were of particular significance relative to performance. Well-compacted granular subbases with high stability and good permeability (internal drainage) showed excellent performance. Failures on CRCP were found to be correlated with concrete having a relatively low bulk density and modulus of elasticity. It was observed that paving concrete with a relatively higher slump (minimum of 1.5 in.) was more suited to CRCP with respect to good performance. An analysis of CRCP design revealed that use of 0.6 percent steel was, at best, marginal. There were further indications that current CRCP thickness design may be inadequate to withstand the traffic loads imposed on Interstate Highways.

The paper concludes with a set of construction and design guidelines that should help to improve the performance of CRCP in Indiana.

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INTRODUCTION

The extensive use of continuous reinforcement in concrete pavement slabs is a relatively recent development in concrete pavement technology. It is of interest to note that the first experimental continuously reinforced concrete pavement (CRCP) was built in 1938 on Route US 40 near Stilesville, Indiana. This was a cooperative research project between Indiana and the Bureau of Public Roads in a study dealing with joint spacing and required steel for various slab lengths. The purpose of the research project was to obtain information on the possibilities of reducing the number of transverse joints in concrete pavements.

The percentages of steel used in the test sections varied from 0.07 to 1.82 percent. The higher percentages of steel were used for the longest slabs (1,310 feet in length). The results of the study indicated that the use of heavier amounts of steel and longer slab lengths was practicable in what is now known as a "continuously reinforced concrete pavement". The sections containing higher percentages of steel are still giving excellent service although sections with lower steel percentages have failed and have been resurfaced (3).

A substantial mileage of this pavement was constructed in Indiana between the years 1966-1971. As of 1966, there were 23 miles of equivalent two-lane CRC pavement in Indiana; by 1971, the CRCP mileage had increased to 696 miles, primarily on the Interstate System.

Most of the CRC pavements in Indiana are nine inches thick (primarily on the Interstate Highway System) although some have been constructed seven and eight inches in thickness. Generally, non-stabilized granular subbases have been used, although the trend in recent years has been towards the use of bituminous-stabilized subbases. The percentage of steel used has been 0.6 percent of the cross-sectional area, irrespective of other design factors.

Background of this Study

In Indiana, the use of CRCP has met with mixed success as regards service performance. Some pavement sections are in excellent condition though they have sustained over a million equivalent 18-kip, single-axle applications during a period of 3-4 years. On the other hand, some sections developed severe distress soon after they were opened to traffic.

Prior to the summer of 1972, the Indiana State Highway Commission investigated the performance of CRC pavements on Interstate Highway I-65. Though several reasons for poor performance were ascertained, the results were inconclusive. It was demonstrated that some distress had occurred because of poor consolidation of the concrete slab below the steel reinforcement. In some instances, it was shown that failures occurred at construction joints and at areas of steel overlap. There were, however, many other areas on I-65 that had shown distress, but the distress mechanism could not be determined.

In July 1972, engineers from the Indiana State Highway Commission asked the Joint Highway Research Project at Purdue University to evaluate the performance of CRC pavements with special reference to certain portions of I-65. In view of this request, a continuing study of the performance of CRC pavements was initiated during the summer of 1972 by the Joint Highway Research Project. The objective of this study was to evaluate and recommend design and construction techniques that would result in improved performance of continuously reinforced concrete pavements.

STUDY PLAN

The evaluation study consisted of the following sequential steps - pavement condition survey; detailed field testing; and laboratory testing. The data, resulting from the various sequential steps of the study, were analyzed within the framework of statistical experimental designs.

The condition survey was conducted on a statewide basis, wherein every CRCP construction contract in Indiana was included in the survey. A total of 89 pavement sections, each 5000 ft. in length, were used. These sections were obtained by random sampling, stratified over the following factors: construction contract, method of paving, method of steel placement, method of steel fabrication, subbase type and subgrade parent material.

A statistical analysis of condition survey data showed that subbase type, methods of steel placement and steel fabrication, concrete slump, and age of pavement since opened to traffic were significant contributors to CRC pavement performance defined in terms of structural failures (breakups), patches, close transverse crack spacing, spalled cracks and edge pumping (10).

Based on these results, a field investigation of CRC pavements was conducted. In this study 31 test sections, each 1000 ft. in length were used. Each test section, as far as possible, represented a unique combination of material, construction, traffic and temperature variables. The test sections were located on Interstate Highways to obtain a homogeneous set of test sections in terms of slab thickness (9 in.) and traffic characteristics. Detailed field evaluation, including tests on all pavement layers and dynamic deflection measurements, was followed by a comprehensive laboratory testing program. A detailed description of the field and laboratory testing programs is given in Reference 8.

The results of the detailed field investigation showed that the failures in CRCP are a function of a number of interacting variables. Generally, higher pavement deflection, decreased pavement stiffness, wider width of cracks, and erratic non-uniform crack patterns were associated with poor pavement condition. Pavement failures were found to be correlated with unstable support conditions and concrete having relatively low bulk density and modulus of elasticity (11).

This paper presents the significant findings of the condition and evaluation study of CRCP in Indiana. In addition, minimum steel requirements for CRCP are discussed. In conclusion, recommendations covering the structural design, material specifications and construction aspects of CRCP are presented.

SIGNIFICANT FINDINGS OF THE PERFORMANCE STUDY

The CRCP evaluation study has provided substantial evidence, regarding the reasons for the relatively poor performance of some CRCP sections in Indiana as well as reasons for the good performance of other sections. Though significant explanations for poor performance have been found, it should be recognized that failures in CRCP are a function of a number of interacting variables relating to factors of material variability, design, construction, climate, and traffic.

Material Properties

The material properties discussed in this section are those of subgrade soil, subbase, and paving concrete.

Subgrade Properties

From a theoretical viewpoint, thickness design of concrete pavements is relatively insensitive to the properties of subgrade as long as pumping, frost action, etc., are accounted for in the design. Nevertheless, being the foundation of the pavement structure, the subgrade influences the behavior of the upper pavement layers. This appears to be of particular relevance to CRCP because of its continuity characteristics as compared to conventional jointed concrete pavement.

There is evidence that CRC pavements on coarse-grained and sandy subgrades experienced significantly less distress than pavements on fine-grained subgrades.

Subgrades with a relatively higher sand content assist in the internal drainage of the pavement. This result acquires added significance, considering that subgrades at most test locations were found to have been poorly compacted, irrespective of the pavement condition.

The percent compaction for subgrades, ranged between 65 to 120 percent of the standard AASHTO (T-99) values, with an average of 92 percent. A distribution of subgrade compaction values is illustrated in Figure 1. At 76 percent of the test locations, the degree of compaction for the subgrades was below the specified minimum.

It has been known for many years that the relative strength or load bearing capacity of the subgrade soil influences the degree of compaction that can be achieved for a graded aggregate base placed on the soil. This has recently been documented by Marek and Jones (13). In view of these considerations, the data show that relatively stiff subgrades are desirable and hence, it is possible that for very weak subgrades some form of stabilization other than mechanical compaction may be needed. It is worthwhile to note that one of the best performing CRCP sections in Indiana (Contract R-7634, I-65) was built on a shale rock subgrade with a crushed stone subbase.

Properties of Granular (Graded Aggregate) Subbases

Subbase, the major element of the support system for CRCP, was found to be a primary factor influencing CRCP performance. In all studies, ranging from visual condition surveys to detailed field and laboratory testing programs, subbase was shown to be a significant determinant of CRCP condition. This is not surprising as continuity requirements of CRCP presuppose the existence of a stable, firm, and non-erodible subbase.

In general, subbase-related distress in CRCP has been associated with gravel subbases, although there are pavement sections on gravel subbase that have performed admirably. In most cases, crushed stone and slag subbases have performed better than gravel subbases.

At test locations, where distress in CRCP could be attributed to graded aggregate subbases, the causes for the poor performance of subbase were found to be inadequate compaction, poor stability (as evaluated by penetrometer CBR), and low permeability.

Compaction: A distribution of percent compaction for gravel subbases is plotted in Figure 2. The average percent compaction was found to be 93.6 percent varying from a low of 75 to a high of 110 percent. These data clearly show that a sizeable amount of the gravel subbases under CRCP were placed in a relatively loose state, with obvious implications of densification under traffic. Dynamic deflection measurements, evaluation of subbase density data, and grain-size distribution considerations indicate that gravel subbases have a tendency to densify under traffic with the related loss in pavement support. Slag subbases in comparison were found to be extremely well compacted, while crushed stone subbases had the same degree of compaction as gravel subbases.

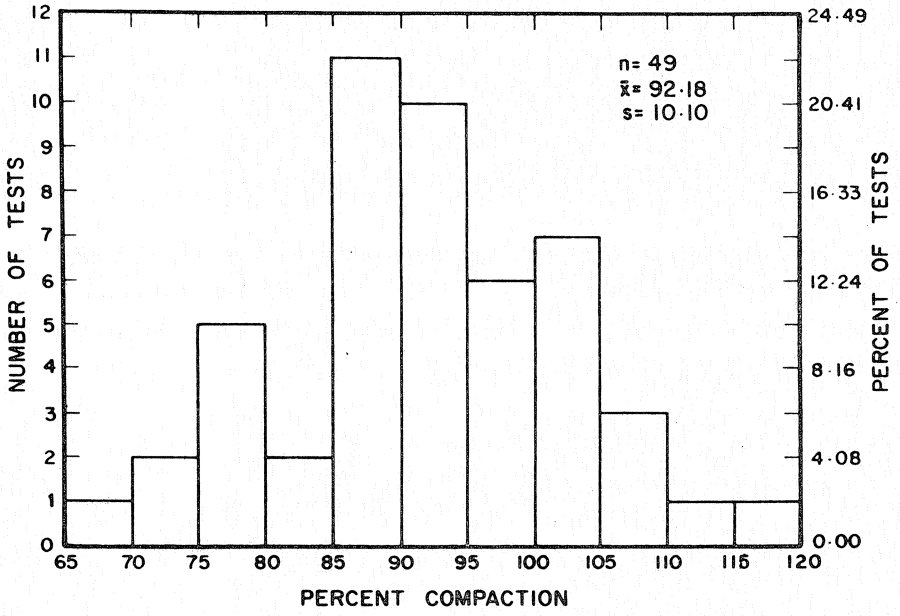


FIG. 1 VARIATION OF PERCENT COMPACTION FOR SUBGRADES

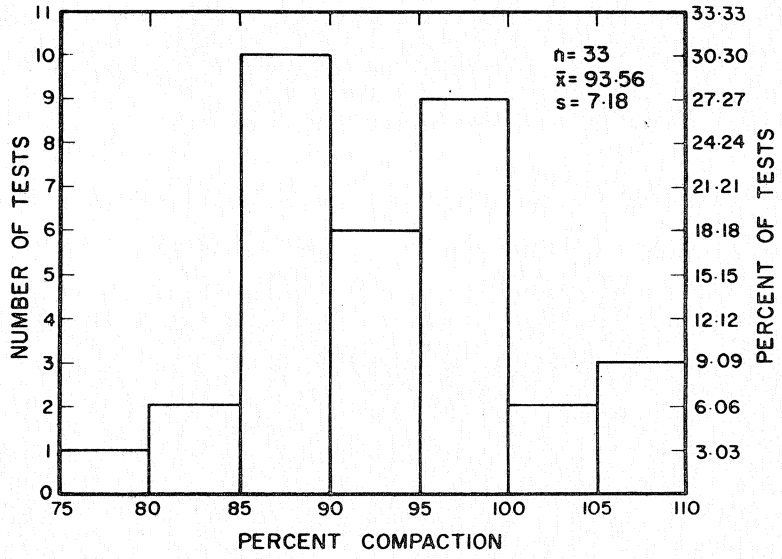


FIG. 2 VARIATION OF PERCENT COMPACTION FOR GRAVEL SUBBASES

Strength (CBR): The stability of gravel subbases, as indicated by a field penetrometer test, was found to be in the range of 20-60 percent CBR with the majority of the CBR values less than 40 percent. For slag and crushed stone subbases, at test sections that had no apparent distress, CBR values of 90-100 percent were obtained. One of the reasons for the better performance of CRCP sections on crushed stone and slag subbases was, no doubt, the better stability characteristics of these latter subbases.

Permeability: Another important element governing subbase performance is the permeability of the subbase. The permeability characteristics of graded aggregate mixtures depend upon grain-size distribution, the type of coarse aggregate, the quality and amount of binder (minus No. 200 sieve fraction), and density.

Permeability of subbases becomes an important consideration regarding pavement performance becaust it reflects the water-retaining ability of the subbase. It has been demonstrated by Cedergren that 80-90 percent of severe and premature damage to pavements is caused primarily by excess water. There are further indications that free water in granular base courses can easily reduce their strength by 25 percent or more under dynamic loads (6).

An appraisal of the permeability characteristics of graded aggregate subbases under CRCP in Indiana leaves much to be desired in view of the above considerations. Figure 3 presents permeability functions for gravel, crushed stone, and slag subbase material sampled during the detailed field study. These functions were obtained from permeability equations, given in Table 1. It is quite clear from these data that crushed stone subbases had markedly better permeability than gravel or slag subbases. In the density range, normally encountered for well-compacted gravel and crushed stone subbases (130-140 lb/ft^3), the permeability of the crushed stone subbases was estimated to be 50 to 100 times higher than that for the gravel subbases.

The permeability curves illustrated in Figure 3 also show the effect of subbase densification on permeability. An increase in the density of gravel subbase from 120 lb/ft^3 to 130 lb/ft^3 resulted in a ten-fold decrease in permeability from 250 ft/day to 25 ft/day. The effect of densification on permeability was less significant for crushed stone and slag (good condition) subbases, as evidenced by the flatter slopes of the permeability functions for these subbase types.

There are possibilities for improving the permeability characteristics of the existing Indiana subbase aggregates by effecting modifications in the subbase specifications. Higher permeabilities may be obtained by:

1. Permitting higher dust ratios (reducing the amount retained between No. 30 and No. 200 sieves).
2. Lowering the material passing the No. 4 sieve (obtaining a more open-graded aggregate).

The net effect of these approaches is to increase the water draining ability of granular subbases. It is pertinent to note that most aggregate specifications limit the dust ratio (the ratio of material passing the No. 200 sieve to the material passing the No. 40 sieve) to about 0.66. It has been proven in a laboratory study by Faiz (9) that for the extreme case, where the dust ratio was approximately 1.0 for a gravel subbase (i.e., skip-graded between No. 30 and No. 200 sieves), there was a marked increase in both stability and permeability over an equivalent densely-graded soil-gravel aggregate.

From a viewpoint of what can be achieved practically, without substantial changes in the existing specifications, the results of this study show that well compacted stable subbase materials must be required to have a minimum permeability of 1000 ft/day. It was observed in the detailed study that subbases with permeability above this limit showed good performance.

TABLE 1 PERMEABILITY RELATIONSHIPS

SUBASE TYPE	MODEL
GRAVEL	$LOG_{10}K = 16.05572 - 0.09884\,\gamma_d - 0.03435(P.4) + 1.10754(DR)$
CRUSHED STONE	$LOG_{10}K = 12.56327 - 0.07464\,\gamma_d$
SLAG *	$LOG_{10}K = 8.68635 - 0.05093\,\gamma_d - 0.11301(P.200)$
SLAG **	$LOG_{10}K = 10.07609 - 0.08756\,\gamma_d$

K = COEFFICIENT OF PERMEABILITY AT 25° C (FT./DAY)
γ_d = DRY DENSITY (LB./FT^3)
P.4 = PERCENT PASSING NO. 4 SIEVE
P.200 = PERCENT PASSING NO. 200 SIEVE
DR = DUST RATIO = $\frac{\text{PERCENT PASSING NO. 200 SIEVE}}{\text{PERCENT PASSING NO. 4 SIEVE}}$
* = SECTIONS WITH NO FAILURE
** = SECTION WITH FAILURE

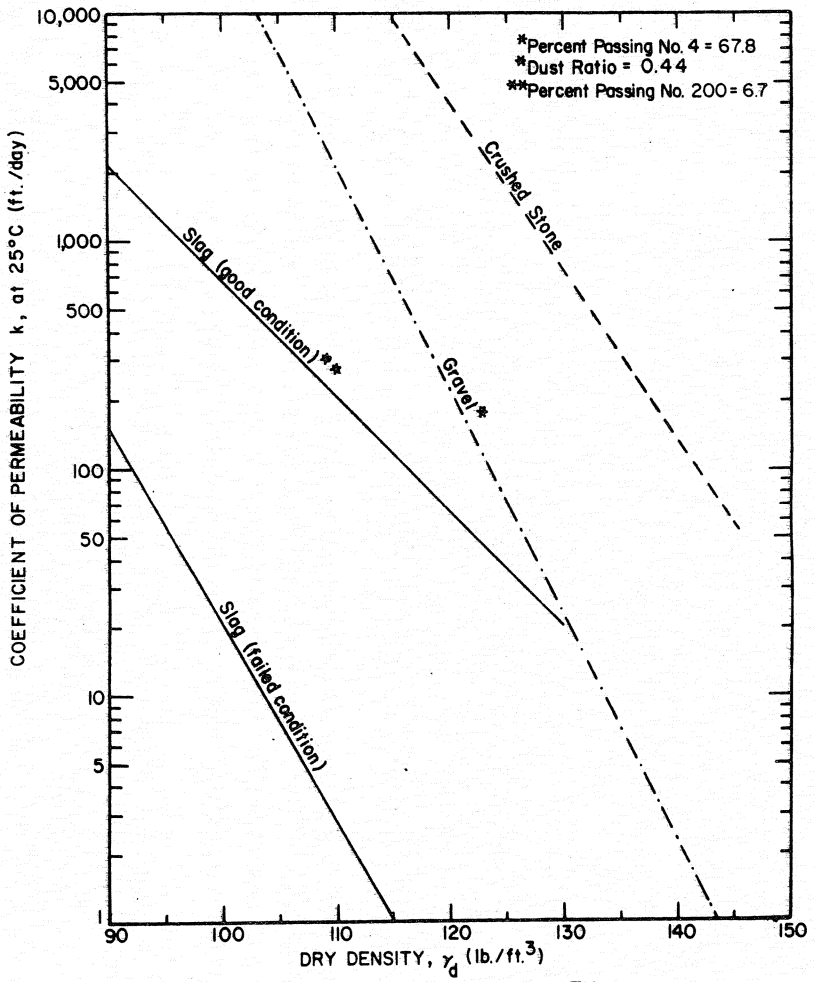


FIG. 3 EFFECT OF DENSIFICATION ON PERMEABILITY

The time required for 50 percent drainage of the subbase may be calculated from the following relationship (4).

$$t_{50} = \frac{n_e D^2}{2k H_o} \qquad (1)$$

where

t_{50} = time for 50 percent drainage of a sloping base course with an edge drain along its lower edge (indicates the rate of fall of saturation of the base course).
n_e = effective porosity of the base.
D^e = sloping width of the base (ft.).
k = coefficient of permeability (ft/day).
H_o = H + SD.
H^o = thickness of the base (ft.).
S = cross slope of the base.

The effect of improved permeability on drainage of the subbase is demonstrated by applying the above equation to the following cases:

a. Subbase with drains along both edges located 1.0 ft. from the pavement edge:

Considering a typical subbase, 6 in. (1/2 ft.) thick with a 13 ft. sloping width, 1.0 percent cross-slope (S = 0.01), and 30 percent effective porosity, it is seen that for:

k = 10 ft/day, t_{50} = 4.021 days
k = 100 ft/day, t_{50} = 0.402 day (9.7 hr.)
k = 1000 ft/day, t_{50} = 0.040 day (0.97 hr.)

An increase in permeability from 10 ft/day to 1000 ft/day reduces the 50 percent drainage time from 4 days to about one hour. For a permeability of 100 ft/day, the corresponding 50 percent drainage time would be about 10 hours.

b. Subbase with drainage through the shoulder:

In this case, the drainage outlet at the shoulder edge is considered as the equivalent of an edge drain. Repeating the above calculations for a 6 in. thick subbase, with a 28 ft. effective sloping width (from the center line of the pavement to the drainage outlet), 1.0 percent cross slope, and 30 percent effective porosity, it is found that for:

k = 10 ft/day, t_{50} = 15.08 days
k = 100 ft/day, t_{50} = 1.51 days (36 hr.)
k = 1000 ft/day, t_{50} = 0.15 day (3.6 hr.)

With drainage across the shoulder, an increase in permeability from 10 ft/day to 1000 ft/day approximately reduces the 50 percent drainage time from 15 days to about 4 hours. Similarly an increase in permeability from 100 ft/day to 1000 ft/day effects a reduction of 32 hours (from 36 hrs. to 4 hrs.) in the 50 percent drainage time.

It should be recognized that the 50 percent drainage time reflects the time for fall of saturation after stop of inflows and indirectly measures the ability of the subbase to retain water.

This presentation shows that the use of a minimum subbase permeability of 1000 ft/day is a reasonable step in the right direction.

Properties of Bituminous Stabilized Subbase

The type of bituminous-stabilized subbase evaluated in this study was a graded gravel aggregate cold-mixed with an asphalt emulsion. One of the test sections with this subbase showed extensive failures. In general, field tests on this subbase indicated relatively low stability similar to that obtained for the gravel subbases, the measure of stability being the penetrometer CBR. The asphalt content and minus No. 200 sieve fraction of the subbase at the failed test section were in excess of the amounts specified for Indiana Type 1 Subbase (12). It is of interest to note that the section without failures had a 6 in. stabilized subbase while the section with failures had a 4 in. subbase. In both cases, the asphalt-stabilized subbase was underlain by a 3 in. layer of gravel.

Comparison of Subbase Types

Table 2 describes the variation of subbase CBR, permeability, and degree of compaction with subbase type. Subbase permeability was estimated from the relationships given in Table 1. Though no clear differences in the properties of the gravel subbases were evident between sections with failures and sections showing no apparent distress, the gravel subbases were not sufficiently compacted and had relatively low permeability and strength.

The results bring to light important differences among the properties of the different subbase types. Crushed stone subbases were the most permeable while slag subbases had the lowest permeability. The relatively poor water transmission characteristics of the slag subbase was more than balanced by its high strength.

The stability of the bituminous-stabilized subbases, as measured by the High Load Penetrometer, was low. The equivalent CBR values were in the same range as those obtained for gravel subbases.

Concrete Properties

An analysis of test properties of concrete cores obtained during the field study showed:

1. Dynamic modulus of elasticity of concrete was of prime importance relative to pavement condition, higher moduli values being associated with good condition.
2. Splitting tensile strength had a minimal influence on pavement condition, within the range of values observed in the field study.
3. On sections that had failures, a relatively lower bulk density of concrete was obtained at the failed locations than at locations with no apparent distress.
4. No significant difference in concrete properties was evidenced above and below the steel reinforcement.
5. Higher concrete slump was associated with better pavement condition, sections with failures had a mean concrete slump of 1.7 in. while sections without failures had an average value of 2.1 in.

A summary of concrete properties relative to pavement condition and position of steel reinforcement is presented in Table 3.

The dynamic modulus of elasticity as evaluated from pulse velocity determinations is a measure of concrete quality, lower values reflecting deterioration of the concrete.

Although modulus of elasticity is not directly proportional to strength, concretes of higher strength usually have higher moduli. An effective way of increasing the modulus of elasticity is by decreasing the water-cement ratio. This also helps in increasing the bulk density, for a constant amount of entrained air (2). However, the increase in modulus of elasticity must not be achieved at the expense of workability, as measured by

Table 2. Effect of Subbase Type on Pavement Condition

Type of Subbase	Condition of Test Sections	No. of Test Sections	Avg. Subbase CBR*		Average Permeability, k	Average Compaction
			Shoulder	Core		
Gravel	With Failures	13	33.5%	38.0%	978.9 ft/day**	91.5%**
	Without Failures	10	35.9	45.2	702.6	93.2
Slag	With Failures	1	95.0	100.0	30.0	117.2
	Without Failures	3	81.0	96.6	142.9	100.6
Crushed Stone	With Failures	1	32.0	41.0	1369.7	89.9
	Without Failures	1	90.0	59.0	2497.1	85.3
Bituminous Stabilized	With Failures	1	34***	----	------	----
	Without Failures	1	38	45	------	----

* CBR values derived from correlations of High Load Penetrometer test values with CBR.

** Average of values from nine test sections.

***Measured at failed test location; all other data from structurally sound test locations.

Table 3. Variation of Concrete Properties with Pavement Condition and Position Relative to the Steel Reinforcement

Concrete Properties	Condition of Test Sections		Position	
	With Failures (15)	Without Failures (15)	Above the Steel Reinforcement (30)	Below the Steel Reinforcement (30)
Average Dynamic Modulus of Elasticity, E_c (psi)	4.97	6.15	5.41	5.71
Average Splitting Tensile Strength, f_{ts} (psi)	518.3	510.7	495.5	533.5

Note: 1. Data from structurally sound test locations.

2. Figures in parentheses denote the number of test sections included in the average.

concrete slump. It has been shown that concrete slump must be greater than 1.5 in. to ensure good CRCP performance.

The modulus of elasticity of concrete is also influenced by the type of aggregate, hard flinty type aggregates having high values. Soft limestone on the other hand has relatively low values. As long as the mix is workable, modulus of elasticity increases with fineness modulus. In general, longer mixing times and curing periods tend to increase the modulus of elasticity (16).

Design Considerations

The two basic considerations in the design of continuously reinforced concrete pavements are the amount of longitudinal reinforcement and the thickness of the pavement slab.

Percent Reinforcement: A detailed analysis of longitudinal reinforcement requirements, presented in a later section of this paper, showed that the use of 0.6 percent steel for Indiana conditions was marginal, even where no allowance for safety was applied. The results of this analysis suggest that the minimum amount of longitudinal reinforcement should be 0.7 percent of the cross-sectional area of the pavement slab.

Thickness Design: The Continuously Reinforced Concrete Pavement Group (7) recommended in 1968 that the thickness of CRCP could be reduced about 20-30 percent over conventional jointed pavement. As a result, it has been commonly accepted that pavement slab thickness may be reduced if the pavement is continuously reinforced. The major support for this contention comes from a deflection study wherein the deflection characteristics of CRCP were compared with those of jointed concrete pavements (14). In this study, it was demonstrated that a 10 in. jointed concrete pavement deflects 1.6 times more than an 8 in. CRCP. This result was based on a comparison of the actually measured deflections on jointed pavement with estimated deflections on CRCP computed from regression equations that had coefficients of determination (R^2) in the range of 0.307 to 0.486. This means that the estimation equations for CRCP deflection explained just 31-49 percent of the variability in the measured CRCP deflection data. In addition, the estimated deflections for CRCP were based on an 8-ft. average crack spacing and a 0.014 in. crack width. In-service CRC pavements are seldom found to have an average crack spacing of 8-ft. and only pavements in excellent condition have the indicated crack width. In view of these limitations and other anamolies in the reported data, any equivalence between the thicknesses of jointed pavement and continuously reinforced concrete pavements, based on deflection measurements, is subject to debate. An even more fundamental question is whether the allowable deflections for jointed pavements are the same as those for CRCP. If, as some researchers contend, CRCP is inherently more flexible than jointed pavement, then thickness equivalencies between the two pavement types based on deflections are highly questionable.

Most failures in both the statewide condition survey and the detailed field study were noted in the traffic lane. The data from this study show that distress in CRCP began at a relatively early stage in the pavement life - approximately after 30,000 equivalent 18 kip, single-axle applications in the critical traffic lane. Furthermore, it has been found that CRCP is subject to a critical edge condition relative to stresses and deflections. These considerations question the validity of current thickness design for CRCP (9 in. pavement slabs on Interstate Highways in Indiana). There is a need to re-evaluate the present thickness design criteria for CRCP from a more fundamental and theoretical standpoint. It is possible that current CRCP thickness design procedures may be entirely inadequate in terms of providing structurally adequate pavements that can withstand the heavy load repetitions on highly trafficked highways.

MINIMUM STEEL REQUIREMENTS FOR CRCP

Longitudinal steel equal to 0.6 percent of the cross sectional area of the pavement slab, has been used as the standard reinforcement design for CRCP in Indiana. Considering the temperature drops in Indiana, from extreme highs during the construction season in mid-summer to the sub-zero winter temperatures, and variability in concrete properties, there were indications that the designed reinforcement was not adequate. To evaluate this factor in more detail, the ACI reinforcement design equations (1) were used to calculate the minimum longitudinal steel requirements for CRCP in Indiana. First, an analysis of percent steel requirements as a function of temperature and material variables was made. Then using the values of maximum temperature drop and concrete properties determined in the course of this study (8), the minimum reinforcement requirements were estimated.

The ACI reinforcement design equations are as follows:

$$\text{Equation No. 1:} \quad p_s = \frac{f_t'}{f_y}(100)(FS) \tag{2}$$

$$\text{Equation No. 2:} \quad p_s = \frac{f_t'}{f_y + \delta E_s - n f_t'}(100)(FS) \tag{3}$$

$$\text{Equation No. 3:} \quad p_s = \frac{f_t'}{f_y - n f_t'}(100)(FS) \tag{4}$$

$$\text{Equation No. 4:} \quad p_s = \frac{f_t'}{2(f_y - \Delta t\ \varepsilon\ E_s}(100)(FS) \tag{5}$$

where

p_s = percentage of steel

f_t' = tensile strength of concrete (psi)

f_y = yield strength of steel = 60,000 psi

δ = 0.0006 for concrete with modulus of rupture of 650 psi (assumed constant for all concrete strengths)

E_s = modulus of elasticity of steel = 29,000,000 psi

ΔT = total temperature drop from temperature at time of construction to some later date (°F)

ε = thermal coefficient of expansions of steel = 0.0000065 in/in/°F

n = modular ratio = $\frac{E_s}{E_c}$

E_c = modulus of elasticity of concrete (psi)

FS = factor of safety (FS=1.3; recommended by ACI)

Figure 4 shows the results of a sensitivity analysis of percent steel requirements determined by the use of these equations. It was noted that percent steel requirements were relatively insensitive to modular ratio (n) values in the range of 5-10, over the parameters used in the analysis. Data from the detailed field study are presented below:

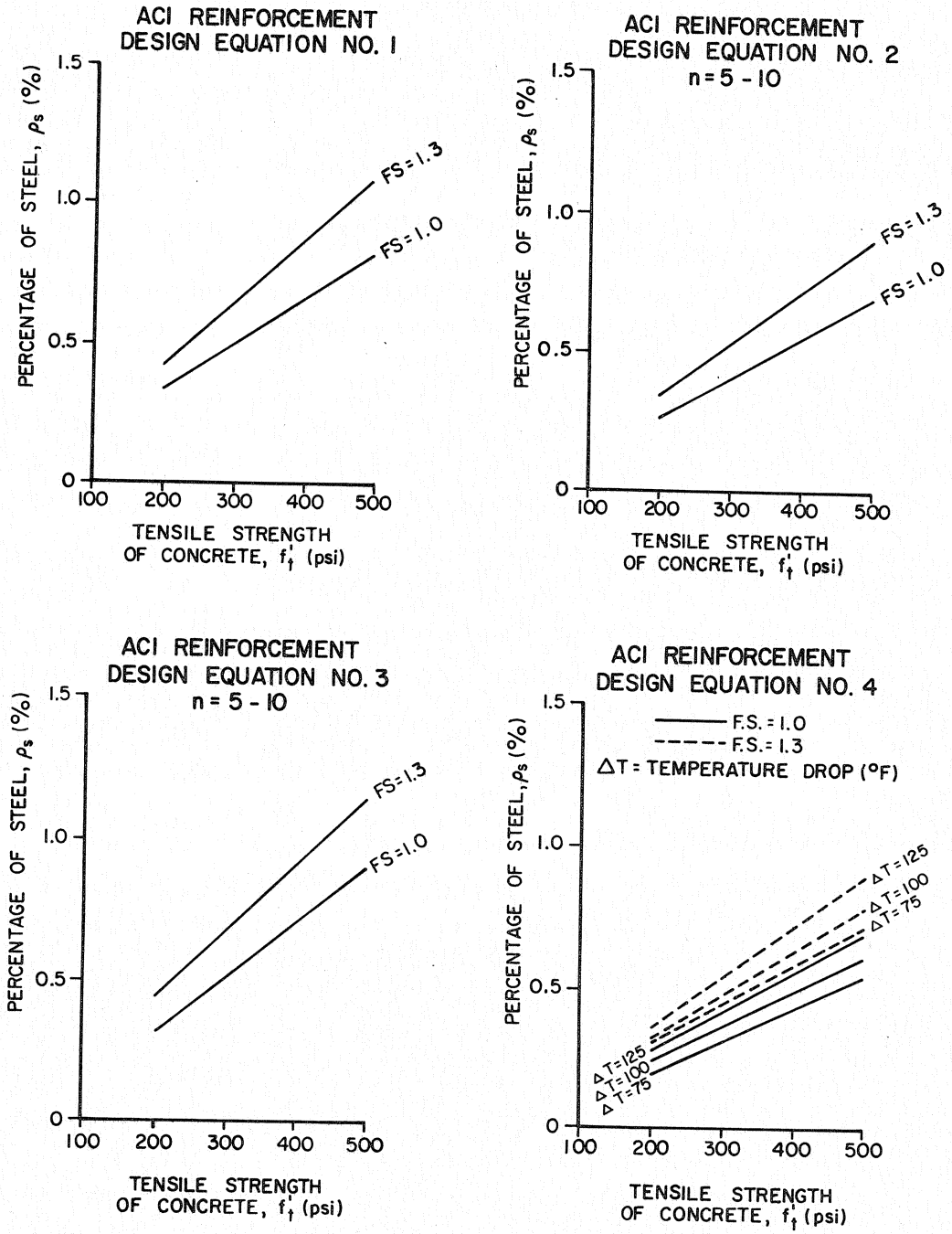


FIGURE 4 MINIMUM CRCP STEEL REQUIREMENTS DERIVED FROM ACI REINFORCEMENT DESIGN EQUATIONS

1. Average splitting strength data for:

Sections with failures		
a. Failed locations	506 psi	(359 psi)*
b. Good locations	523 psi	(371 psi)*
Sections without failures	511 psi	(362 psi)*

*Estimated design tensile strength (f_t')

2. Average dynamic modulus of elasticity data for:

Sections with failures		
a. Failed locations	4.76×10^6 psi	n = 6.1
b. Good locations	5.03×10^6 psi	n = 5.8
Sections without failures	6.15×10^6 psi	n = 4.7

Note: Static modulus of elasticity is about 0.55-0.75 times the dynamic modulus of elasticity, depending upon concrete strength (see Neville, Ref. 15). Use of the static modulus of elasticity would result in larger n values than shown above and consequently higher percent steel.

3. Average maximum temperature drop from time of paving to the time of detailed field study (8):

	Air Temperature (°F)	Estimated Slab Temperature (°F)
a. Sections with failures	99.6	102.8
b. Sections without failures	96.0	98.4

Design tensile strength of concrete was estimated, from the recommendations given in the ACI report, as follows:

$$\text{Splitting tensile strength, } f_{ts} = (0.564)(MR) \quad (6)$$

$$\text{Design tensile strength, } f_t' = (0.40)(MR) \quad (7)$$

Combining Equations 6 and 7,

$$\text{Design tensile strength, } f_t' = \left(\frac{0.40}{0.564}\right) f_{ts} = 0.70\, f_{ts} \quad (8)$$

The modular ratio (n) values are based on the dynamic modulus of elasticity of concrete. It is to be noted that dynamic modulus of elasticity reflects only the purely elastic characteristics of concrete and is not affected by creep. The dynamic modulus is approximately equal to the initial tangent modulus but is appreciably larger than the secant (static) modulus. The ratio of static and dynamic moduli varies from approximately 0.5 to 0.8 depending upon the strength of the concrete (15). Use of n values corresponding to the dynamic modulus of elasticity results in a slightly lower percentage of steel than would be obtained if the static modulus of elasticity values were used.

It is seen from Figure 4, that the critical steel requirements are those given by the ACI Equations No. 1 and 3, to control restrained volume changes due to shrinkage and temperature, respectively.

To control restrained volume changes due to shrinkage (ACI, Equation 1), the minimum steel requirement, using estimated design tensile strength (359-371 psi), is about 0.6 percent steel without any allowance for safety. Using a safety factor of 1.3, as recommended in the ACI report, the minimum steel requirement increases to about 0.78 percent.

For the control of restrained volume change due to temperature (ACI Equation 3), the minimum requirement is approximately 0.63 percent steel with no allowance for safety. This requirement increases to about 0.82 percent if a safety factor of 1.3 is used.

This analysis shows that, at best, the longitudinal reinforcement design for CRCP in Indiana (0.6 percent) was marginal. Without any allowance for safety, it was barely sufficient to control shrinkage stresses and was a little less than the minimum needed to counter restrained volume change due to temperature. If a factor of safety (1.3) is considered, as recommended by the ACI design procedure, then one cannot escape the conclusion that the designed steel was less than the minimum needed (0.78 to 0.82 percent). The estimates of minimum steel requirements are on the low side being based on:

1. An average temperature drop of 100°F.
2. Dynamic modulus of elasticity of concrete (rather than the static modulus).
3. A lower tensile strength than indicated by the splitting tensile strength test.

RECOMMENDATIONS

One of the main objectives of this investigation was to develop design and construction guidelines that would result in better performance of CRCP in Indiana.

As a fulfillment of this objective, recommendations covering the design, construction, and service performance of CRCP, are presented in the following sections.

CRCP Structural Design

1. Percent Steel: In view of the material variability and range of temperatures encountered in Indiana, it is recommended that the amount of minimum longitudinal reinforcement be increased to 0.7 percent.
2. Pavement Slab Thickness: The currently used "80-percent rule" regarding CRCP thickness (CRCP thickness = .80 x equivalent jointed pavement) should be re-evaluated. There are strong indications that the pavements obtained by the current designs are not adequate to withstand the traffic loadings on Interstate Highways. It is suggested that CRCP thickness be the same as that needed for an equivalent jointed pavement.
3. Subbase Thickness: Minimum thickness of bituminous stabilized subbases should be increased to 6-in. from the current standard practice of a 4 in. subbase. On weak subgrades, increase in thickness of both the graded aggregate and bituminous stabilized subbases may be warranted.

Material Specifications

1. Subgrade: Subgrades should be compacted to a minimum of 100 percent standard AASHTO (T-99) density, with proof-rolling. The density should be corrected for material retained on the No. 4 sieve. To effect adequate compaction control, the one-point compaction curve method (17) is recommended. Adoption of quality control procedures is also recommended.
2. Subbase (Graded Aggregate): For gravel, crushed stone and slag subbases the following recommendations are offered relative to:

   a. Compaction:
      1. 100 percent Standard AASHTO (T-99) effective minimum.
      2. 100 percent Modified AASHTO (T-180) desirable.
      3. Density values to be corrected for material retained on 3/4 in. sieve.
      4. Use of control curves and quality control techniques.

b. Stability: Minimum CBR of 100 percent.
c. Permeability: Minimum K = 1000 ft/day. Provision of effective sub-drainage (collector drains, filter courses as needed, trenches, and marked pipe outlets), in accordance with FHWA Guidelines (5).

3. Subbase (Bituminous Stabilized): Definitive criteria was not obtained in this study. There were indications that Indiana Type 1 subbase (cold mixed with asphalt emulsion) does not have sufficient stability.
4. Concrete: Paving concrete should have:
   a. Dynamic modulus of elasticity: 5.0-6.0 million psi.
   b. Splitting tensile strength: greater than 450 psi.
   c. Slump: a minimum slump of 1.5 in.; a 2-in. minimum slump would be more desirable.
5. Reinforcement: To ensure adequate vibration of concrete it is recommended that size No. 6 deformed bars or equivalent sizes in deformed wire fabric be used for longitudinal steel. Transverse steel may be omitted if conditions warrant.

Construction

With respect to method of construction, the following recommendations are in order:

1. Use of pre-set steel on chairs should be discontinued.
2. Performance of tied bar mats should be critically examined by further field observations. There are indications that this type of steel fabrication is related to a higher incidence of failures than wire fabric or loose bars.
3. Sufficient vibration and compaction of concrete is essential to good performance of CRCP.
4. To permit good construction control, slower paving rates should be used.
5. Construction joints are points of weakness in CRCP, needing considerable care in their construction.

Pavement Response Functions

The response of continuously reinforced concrete pavement was evaluated in terms of dynamic deflection, surface curvature index (inversely related to pavement stiffness), crack width, crack spacing, and bifurcated cracks. For each pavement response function evaluated in this study, the values associated with good pavement condition are presented in Table 4.

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Table 4. Values of Pavement Response Functions Associated with 9 in. CRCP in Good Condition

Pavement Response Function	Values Associated with Good Pavement Condition
Dynamic Deflection*	Less than 0.90 milli-inch at 1.0 ft. from the pavement edge. Less than 0.60 milli-inch at 6.0 ft. from the pavement edge.
Surface Curvature Index*	Less than 0.07 milli-inch at 1.0 ft. from the pavement edge. Less than 0.05 milli-inch at 6.0 ft. from the pavement edge.
Crack Width*	Less than 0.015 in.
Average Crack Spacing	4.0 to 10.0 ft.
Crack Spacing Variance**	Less than 9 $ft.^2$
No. of Crack Intersections per 100 ft. length of Pavement	No more than 4-5

* Based on summer measurements (June) at an average temperature of $77^{o}F$.

** Variance, $V_c = \frac{\sum_{i=1}^{n} (X_{ic} - X_c)^2}{n - 1}$

where X_{ic} = individual crack intervals, i = 1, ... n.

X_c = mean crack interval (spacing).

n = number of crack intervals per 100 ft. length of pavement.

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