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CONTENTS

FOREWORD

The six papers, one with a discussion, contained in this RECORD reflect the spectrum from theory to practice that must be covered by research on concrete for highway applications. The papers by Ingram and Furr, Stratfull and Spellman, Carrier and Cady, and Newlon, Davis, and North, as well as the discussion by Chamberlain, Amsler, and Jaqueway, all treat various aspects of concrete performance in bridge decks. The last two papers and the discussion reflect conclusions based on visual surveys made on large numbers of bridge decks in service in Pennsylvania, Virginia, and New York. Although the three surveys were completely independent, the results are complementary and in basic agreement on certain performance characteristics (both good and bad) and possible causative factors. Although limited, the data relating to the influence of stay-in-place forms are of particular interest because of current questions related to the use of the former and a lack of previously published quantitative information.

A critical influence on the performance of concrete in bridge decks is the moisture content of the concrete. Factors influencing this important parameter are discussed by Ingram and Furr.

The paper by Stratfull and Spellman supplements earlier work from their extensive and valuable studies of the corrosion of steel in bridge decks. The methods they have developed are being evaluated on a broad scale by highway departments and national standardization bodies.

Essential to a valid understanding of observed performance is an appreciation of the fundamental materials behavior and mechanical properties of the concrete. The paper by Palotás presents an analytical method for quantifying the influence of non-load-induced factors on the stress and cracking of concrete. Johnston relates anisotropy, a perplexing but somewhat conceptual characteristic of concrete, to a specific performance characteristic, strength.

The diversity of backgrounds and affiliations of the authors also reflects the spectrum from academia to operations, along with the international effort to bridge the gap between the two. This diversity continues to be both the challenge and the hope of concrete research.

—Howard H. Newlon, Jr.

ANALYSIS OF THE RESIDUAL STRESSES IN CONCRETE DUE TO TEMPERATURE VARIATION, SHRINKAGE, AND CREEP

L. L. Palotás, Technical University, Budapest

Because the physical properties (thermal expansion and shrinkage) and mechanical properties (strengths and moduli of elasticity) of the components of concrete are different, concrete is considered a multiphase material. For the sake of simplicity, it is assumed in this paper that there are two phases: the cement stone (i.e., the hardened cement paste) as the matrix and the aggregate particles as the discrete phase. Because the phases are prevented from free deformation by their strong bond, hampered strains impose balanced stresses on cement stone and aggregate; thus, the concrete gets in a residual stress condition. Three elementary models (disk, circular cylinder, and sphere) simulating a cement disk of outer shell and an aggregate disk of inner core will be applied to illustrate the evaluation of residual stresses in concrete due to temperature variation, shrinkage, and creep. Diagrams are also presented for easier applicability of the method.

•CONCRETE is a building material consisting of two phases, the viscoelastic cement stone (i.e., hardened cement paste) and the elastic aggregate, where differential phase properties affect the development of strains and stresses in a definite manner.

Such nonhomogeneous, non-isotropic materials cannot freely develop motions upon physical and mechanical influences because the identical deformations on phase interfaces necessarily impose inner constraints and the concrete gets in an equilibrium condition of residual stresses.

In what follows, some ideas will be presented that are related to the development of residual stresses in each phase, due to phase differences in concrete. An exact solution is very difficult; therefore, it has to be approximated by introducing certain elementary assumptions. Phases are assumed to fit together without slip and gaps to meet the principle of compatibility. This leads to the assumption that, in any point of the interface, the strains of cement stone and aggregate in a given direction are equal.

The main difficulty affecting concrete residual stress calculations is that knowledge of the mechanical properties of the concrete and its phases is required, as well as the particular physical characteristics of each phase and the resultant concrete characteristics. Although the strains can be measured directly as deformations, the stresses should at most be deduced from strain values. The effective moduli of elasticity of the cement stone and aggregate (E_s and E_a) required for the determination of the stresses in cement stone and aggregate are by no means the ones obtained by some of the usual methods from uniaxial loading but are complex characteristics obtainable only indirectly from certain measurements in a concrete of given composition and quality. Thus, the effective E_s and E_a cannot be determined by independent tests on the cement stone or aggregate, or, if they are, such results can only be applied with approximate corrections. Such a simplified method, based on the standard modulus of concrete E_c and on the absolute volumes of cement stone and aggregate v_s and v_a, is presented below.

Sponsored by Committee on Mechanical Properties of Concrete.

Concrete stress σ_c is assumed to be composed of stresses σ_s and σ_a in the cement stone and the aggregate respectively, leading to the relationship

$$\sigma_c = c\sigma_s + (1 - c)\sigma_a \tag{1}$$

Strains ϵ being identical, Eq. 1 leads to

$$E_s = E_c/[c + (1 - c)n] \tag{2}$$

where $n = E_a/E_s$ and $c = f(v_s)$.

E_c, v_s, and v_a are known for a given case, whereas n can be determined by the suitable deformation condition as follows. In the case of low stresses, E_c may be replaced by the initial tangential modulus

$$\begin{aligned} E_{co} &= 550{,}000\rho \\ \rho &= f_c/[f_c + 200] \end{aligned} \tag{3}$$

where f_c, in kips/cm^2, is the concrete cube strength. In general, the modulus of deformation E_c for stress at any time t is

$$E_c = \nu E_{co} \text{ kip/cm}^2 \tag{4}$$

where

$$\begin{aligned} E_{so} &= \frac{E_{co}}{c + (1 - c)n} \\ E_s &= \nu E_{so} \\ \nu &= \frac{\nu_o}{1 + \varphi} \\ \nu_o &= \frac{1}{2}\left[1 + \left(1 - \frac{\sigma}{\sigma_\mu}\right)^{1/2}\right] \end{aligned} \tag{5}$$

The rate of creep φ at any time t is given as

$$\varphi = k_o k_r \delta_{\varphi_n} \tag{6}$$

k_o, the factor characterizing the time of beginning of application of the sustained load, can be expressed as

$$k_o = 3.9 \times e^{0.77t^{1/6}} \tag{7}$$

k_r is a factor expressing the effect of the moisture content n_r (in percentage) of the surroundings:

$$k_r = \frac{115 - n_r}{100 - 0.7\, n_r} \tag{8}$$

δ is a process function of the creep, which is

$$\delta = 1 - 3^{-0.1t^{1/2}} \tag{9}$$

φ_n is the final value of creep, and σ is either the prism or cylinder strength σ_p or the tensile strength σ_t of the concrete in kip/cm^2.

To provide uniformity of treatment, the average concrete strain ϵ_c is assumed to be of the form

$$\epsilon_c = c \times \epsilon_s + (1 - c)\epsilon_a \tag{10}$$

which is obtainable from the free deformations of cement stone and aggregate. Here c is a function of the absolute cement volume in the concrete, expressed approximately by

$$c = v_s^{3/2} \tag{11}$$

This last equation is supported by experimental values on thermal deformations (α_c, α_s, and α_a), on shrinkage (ϵ_{cs}, ϵ_{ss}, and ϵ_{as}), and on creep of concretes.

RESIDUAL STRESSES DUE TO TEMPERATURE VARIATION

It is assumed that the thermal expansion coefficients of concrete and its components (α_s, α_a, and α_c) and the Poisson's ratios (μ_s and μ_a) are known. Similarly, the concrete composition (absolute volumes v_s and v_a) and the concrete modulus of elasticity (E_c) should be given. α_c is simply calculated by means of Eq. 10 (1). Approximate calculations of the residual stresses will be attempted by the following three model types:

1. Plane disk or linear model consisting of intercrossing cement stone and aggregate disk elements of absolute volumes v_s and v_a respectively (Fig. 1);
2. Circular cylinder model (Fig. 2); and
3. Spherical model (Fig. 3) with the cement stone as outer shell and the aggregate as core.

Starting from certain accepted equations of statics, kinematics, and strength of materials, some important relationships between the $n = E_a/E_s$ ratio and the stresses σ_s and σ_a in cement stone and aggregate due to 1 deg C of temperature variation have been compiled and are shown in Figures 1, 2, and 3. Because temperature variation may be considered as an instantaneous effect, zero rate of creep φ has been assumed.
The important relationships on the basis of the elasticity equations are given below:

1. For disks (Fig. 1),

$$p = p_s = \sigma_{sd}\, v_s = -p_a = -\sigma_{ad}\, v_a \tag{12}$$

$$p = \frac{(\alpha_c - \alpha_s)}{1 - \mu_s} E_{sd}\, v_s = \frac{(\alpha_a - \alpha_c)}{1 - \mu_a} E_{ad}\, v_a \tag{13}$$

$$\sigma_{sd} = \frac{p}{v_s} \Delta t = -\frac{(\alpha_s - \alpha_a)}{1 - \mu_s} (1 - c) E_{sd}\, \Delta t$$

$$\sigma_{ad} = -\frac{p}{v_a} \Delta t = \frac{(\alpha_s - \alpha_a)}{1 - \mu_s} c\, E_{ad}\, \Delta t \tag{14}$$

$$E_{sd} = \frac{E_c}{c + (1 - c)n_d} \quad E_{ad} = n_d E_{sd} \tag{15}$$

$$n_d = \frac{1 - c}{c} \frac{v_s}{v_a} \times \frac{1 - \mu_a}{1 - \mu_s} n_o \beta_d \tag{16}$$

$$n_o = \frac{1 - c}{c} \frac{v_s}{v_a} \quad \beta_d = \frac{1 - \mu_a}{1 - \mu_s} \tag{17}$$

2. For cylinders (Fig. 2),

$$p = \frac{(\alpha_c - \alpha_s)}{1 + \mu_s + (1 - \mu_s)/v_a} \times E_{sc}\, v_s = \frac{(\alpha_a - \alpha_c)}{1 - \mu_a} E_{ac} \tag{18}$$

$$\left.\begin{aligned}
\sigma_{ts,i} &= p\frac{1+v_a}{v_s}\Delta t & \sigma_{rs,i} &= -p\Delta t\\
\sigma_{ts,o} &= -p\frac{2v_a}{v_s}\Delta t & \sigma_{rs,o} &= 0\\
\sigma_{ta,I} &= -p\Delta t & \sigma_{ra,i} &= -p\Delta t\\
\sigma_{ta,o} &= 0 & \sigma_{ra,o} &= 0
\end{aligned}\right\} \tag{19}$$

$$n_c = n_o \beta_c \tag{20}$$

$$\beta_c = \frac{(1-\mu_a)v_a}{1+\mu_s+(1-\mu_s)/v_a} \tag{21}$$

3. For spheres (Fig. 3),

$$p = \frac{(\alpha_c - \alpha_s)}{1+\mu_s+2(1-2\mu_s)/v_a} 2E_{ss} v_s = \frac{(\alpha_a - \alpha_c)}{1-\mu_a} E_{as} \tag{22}$$

$$\left.\begin{aligned}
\sigma_{ts,i} &= p\frac{1+2v_a}{2v_s}\Delta t & \sigma_{rs,i} &= -p\Delta t\\
\sigma_{ts,o} &= p\frac{3v_a}{2v_s}\Delta t & \sigma_{rs,o} &= 0\\
\sigma_{ta,i} &= -p\Delta t & \sigma_{rs,i} &= -p\Delta t\\
\sigma_{ta,o} &= 0 & \sigma_{rs,o} &= 0
\end{aligned}\right\} \tag{23}$$

$$n_s = n_o \beta_s \tag{24}$$

$$\beta_s = \frac{(1-2\mu_a)/2v_a}{1+\mu_s+2(1-2\mu_s)/v_a} \tag{25}$$

Figure 4 shows the n and E_s values versus v_s, whereas Figure 5 shows the variation of mean σ_s values based on the three models for $\alpha_s = 20.10^{-6}$ deg C, $\alpha_a = 10.10^{-6}$ deg C, $\nu = \nu_o = 1$, $\mu_s = \mu_a = 0$, $\mu_s = 0.2$, and $\mu_a = 0.1$. Because interfaces have been assumed to contact each other slipping-free, approximate values based on the linear model can also be obtained by assuming $\mu_s = \mu_a = 0$.

RESIDUAL STRESSES DUE TO SHRINKAGE

Residual stresses due to shrinkage can be determined by applying the relationships obtained for residual thermal stresses. Evidently, because shrinkage stresses are of a permanent character, the calculation of stresses has to take into account the creep rate φ for the given concrete grade and time. In general, aggregate shrinkage is practically negligible as compared to that of the cement stone; thereby, the second term in Eq. 10 can be omitted. Thus the concrete shrinkage at time t is expressed by

$$\epsilon_{cs} = c \times \epsilon_{ss}$$

At an arbitrary time, shrinkage of concrete or that of the cement stone is given by the process function corresponding to the given conditions

$$\epsilon_{cs} = \delta(t)\epsilon_{cs,\infty}$$

or

$$\epsilon_{ss} = \delta(t)\epsilon_{ss,\infty}$$

The process function can be written as

$$\delta(t) = 1 - e^{-a}$$

where a depends on the given circumstances (cement grade, water-cement ratio, degree of hydration, ambient humidity, and mean drying thickness) and on the time. For relative humidities of about 90, 70, and 40 percent, as well as 10 cm of mean drying thickness, a may be assumed as $0.07t^{1/3}$, $0.10t^{1/3}$, and $0.13t^{1/3}$ respectively (2).

Final shrinkage values of concrete for high early strength portland cement, 10 cm of mean drying value, relative humidities of 90, 70, and 40 percent, and water-cement ratios of 0.4, 0.6, and 0.8 are shown in Figure 6. Figure 7 shows characteristic values (E_c, f_c, φ, ν, ν_o, E_{so}) for 28-day portland cement of $f_o = 245$ kip/cm^2 (3,500 psi) based on the plane disk (linear) model for $\mu_s = \mu_a = 0$. Hence, among characteristics needed to calculate stresses, the ratio n and the modulus of elasticity E_s have been calculated by means of Eqs. 16 and 2 respectively. Tensile and compressive stresses in cement stone and aggregate were obtained with the following formulas:

For tensile stresses,

$$\sigma_{s,s} = (\epsilon_{s,s} - \epsilon_{c,s})E_s = \epsilon_{s,s}(1 - c)\nu E_{so} \tag{26}$$

and for compressive stresses

$$\sigma_{as} = - \epsilon_{cs} nE_s = - \epsilon_{ss}(1 - c)(v_s/v_a)E_s = - \sigma_{ss} v_s/v_a \tag{27}$$

Because there is no uniform temperature or shrinkage along the concrete cross section, in addition to the stresses obtained by assuming uniform distribution, residual stresses develop in elementary fibers due to temperature gradients and variable shrinkage (moisture). It is evident from the informative stress values presented that, under the combined effect of cooling and shrinkage, the tensile strength of cement stone may be exceeded and inner microcracks as well as surface cracks may develop.

STRESS REARRANGEMENT IN CONCRETE DUE TO CREEP

For simplifying the calculation of the stress rearrangement, the effect of diversity of the different kinds of rocks will be neglected. It will also be assumed that their creep may be ignored. However, it should be stressed that the results of the latest experiments clearly show the effect of the different kinds of rocks on the development of shrinkage and creep (17, 18, 19).

The sustained load, in the moment of application, induces initial stresses in the cement stone and aggregate in the concrete. In the course of time, the values of these stresses change as the deformations increase, but still the new stress state developed is a state of equilibrium of stresses.

The eigenstresses developed are determined—by assuming the cross section to be planar—by satisfying the equilibrium of deformations with the help of a simple model wherein the cement stone and skeleton of the aggregate are of plane disk lamination. A more accurate model has been applied by Baker (20) wherein he tried to describe the stress pattern developed within the concrete by a lattice consisting of vertical, horizontal, and diagonal bars.

The experiments conducted on creep showed that the aggregate, possessing no significant anelastic properties, behaves as a large system of elastic rigidity. As a matter of course, the stresses originally developed in the cement stone will decrease owing to its creep, and their major part will be transferred to the skeleton of aggregate.

In order to clarify this question, let us start from the simple model already mentioned above (Fig. 1), where a plane disk lamination of each of the elements (cement stone and aggregate) was assumed.

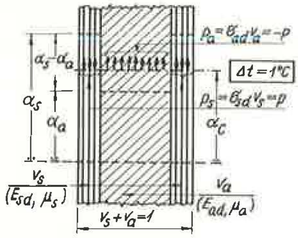

Figure 1. Plane disk model.

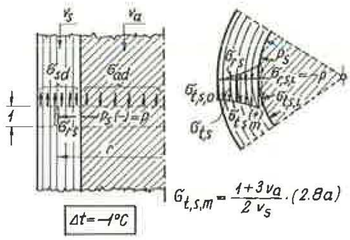

Figure 2. Circular cylinder model.

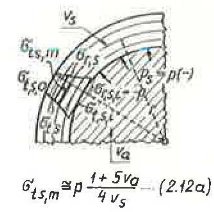

Figure 3. Spherical model.

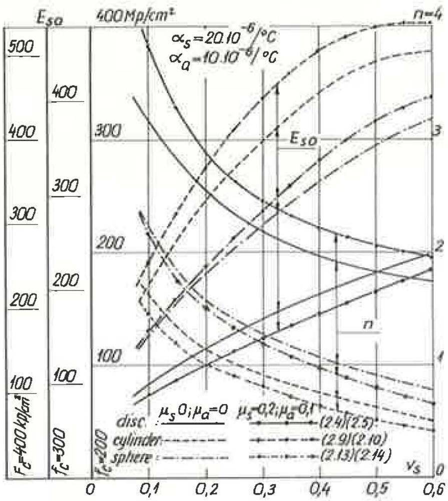

Figure 4. Values of n and E_s versus v_s.

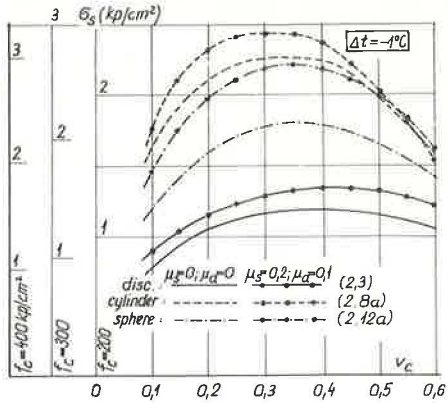

Figure 5. Mean σ_s values versus v_s.

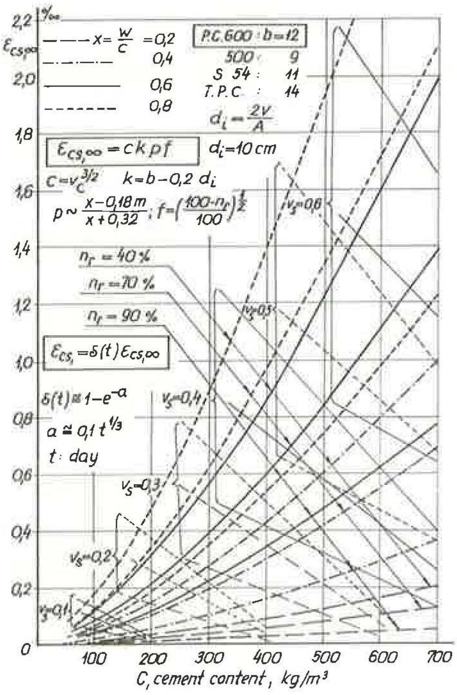

Figure 6. Final concrete shrinkage versus v_s.

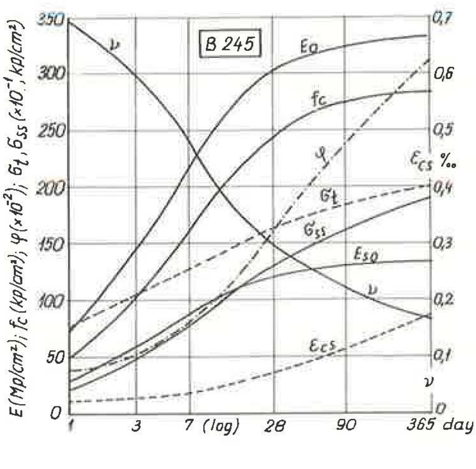

Figure 7. Effects on shrinkage of a concrete of cube strength f_c = 245 kip/cm^2 (3,500 psi).

Naturally, a model in which both vertical and horizontal and even diagonal laminations are assumed would strongly entangle the problem. In general, the Poisson's ratio has been supposed in the calculations to be different from zero.

Compressive Force

At an instant $t = 0$, i.e., at the moment of application of the load, the compressive force induces the stresses σ_{co} and σ_{ao} in the cement stone and aggregate respectively. The compressive force may be calculated from the formula

$$F = F_{so} + F_{ao} \tag{28}$$

where the following relationships may be assumed:

$$F_{so} = v_s A_c \sigma_{so} = A_s \sigma_{so}$$
$$F_{ao} = v_a A_c \sigma_{ao} = A_a \sigma_{ao}$$
$$v_s A_c + v_a A_c = A_c$$

The common deformation is

$$\epsilon_o = \epsilon_{so} = \frac{F_{so}}{A_s E_{co}} = \frac{F_{ao}}{A_a E_{ao}} = \frac{F}{A_c E_c} \epsilon_{ao}$$

from which

$$F_{ao} = F_{co} \frac{E_{ao}}{E_{co}} \times \frac{v_a}{v_s} = F_{so} n_o \gamma_a = \frac{1 - c}{c} F_{so} \tag{29}$$

where

$n_o \gamma_a = \dfrac{1 - c}{c}$,

$\gamma_a = \dfrac{v_a}{v_c}$, and

$E_{ao} = n_o E_{co}$.

From Eq. 28,

$$F_{so} = cF$$
$$F_{ao} = (1 - c)F$$

From these

$$\left.\begin{aligned} \sigma_{ao} &= \epsilon_o E_{ao} = n_o \sigma_{so} = \frac{(1 - c)F}{A_a} \\ \sigma_{so} &= \epsilon_o E_{so} = \frac{cF}{A_s} = c\sigma_{co} \\ \sigma_{co} &= \frac{F}{A_c} \end{aligned}\right\} \tag{30}$$

Effect of Creep

As a result of creep, the initial compressions increase with the time. The increment of deformation, i.e., the creep, is

$$\epsilon_\ell = \epsilon_o \varphi$$

where φ is as given.

Due to the creep, the σ_{co} initial compressive stresses decrease in the cement stone, first (probably due to a self-compaction) with a lower, later with a higher, and then again with a lower rate of change in stress. This latter slowdown may be due to the development of $\sigma_{s\varphi}$ tensile stresses. Meanwhile, only a very small (commonly negligible) $\sigma_{a\varphi}$ compressive stress increment is added to the σ_{ao} initial compressive stresses in the aggregate. During this transfer of compressive stresses, the cross section is in a state of equilibrium. At this state the deformation ϵ_{ϱ} is the value of the slow deformation of the cement stone $\epsilon_{\varrho c}$ reduced by mechanical resistance of the aggregate.

At an instant t, due to the state of eigenstress, $\sigma_{c\varphi} = -\sigma_{a\varphi}\gamma_a$. In this equation, $\sigma_{a\varphi}$ is compressive stress (a negative value) and $\sigma_{c\varphi}$ is tensile stress.

For the simplified estimate of the increment of the compressive stress induced in the aggregate, it seems convenient to apply the reduction factor φ_r that is based on Dischinger's theorem. This factor is as follows:

$$\varphi_r = \frac{c}{1 - c}\left[1 - e^{-(1-c)\varphi}\right] \tag{31}$$

For the determination of the stresses due to shrinkage, the deformation equilibrium equality can be used. According to that, the shrinkage of the concrete may be assumed as being composed of the shrinkages of the cement stone and aggregate.

Because the shrinkage of the aggregate, just like its creep, is very small in comparison to the similar deformation of the cement stone, it is negligible. Thus, the shrinkage of the concrete is governed, first of all, by the cement stone.

The free deformation of the cement stone, as one of its basic characteristics, may be obtained from Eq. 10 as

$$\epsilon_s = \frac{\epsilon_c}{c} = \frac{\epsilon_o}{c}$$

The reduced creep of the concrete is given by the relationship

$$\epsilon_{c,s} = \epsilon_s \frac{\varphi_r}{\varphi_n}$$

Accordingly, the stresses in the aggregate and cement stone due to the creep are

$$\sigma_{a\varphi} = \epsilon_{cs} E_a \tag{32}$$

and

$$\sigma_{s\varphi} = -\sigma_{a\varphi}\gamma_a = -\epsilon_{sc}\gamma_a n_o E_s = -\epsilon_{cs}\frac{1 - c}{c} E_c \tag{33}$$

respectively where, at an instant t, E_c and E_a are as given in Eq. 2, $E_s = \nu E_{so}$, $E_a = n_o E_s$, and $\nu = \frac{\nu_o}{1+\varphi}$.

Effect of Moment

It is assumed that the moment M applied together with a compressive force F is so low that it causes only a low tensile stress at both edges of the specimen, so that no crack occurs in the cross section. Then the deformation equilibrium constraint equation states that the rotation of the cross section calculated on the basis of either the aggregate or cement stone is the same. The amount of traverse of the stresses may be obtained by the same deduction by the respective application of the formulas.

All that has been said is to be applied also to other kinds of materials.

CONCLUSIONS

Concrete can be considered as a two-phase composite material for the description of both its physical (thermal expansion, shrinkage) and its mechanical (compressive, tensile, and flexural strains, creep) properties. One of the phases is the viscoelastic cement stone, i.e., the porous hardened cement paste, as the matrix; the other is the elastic aggregate particles as the discrete phase. The influence of these two phases on the concrete properties differs significantly.

The two phases in the concrete are forced to deform together causing a mutual restriction, i.e., a state of residual stresses in both the paste and the aggregate. Approximate descriptions of these residual stresses can be obtained by applying formulas of the theory of elasticity (Eqs. 1 through 11) on any of the three elementary models: disk, circular cylinder, and sphere. In these models the cement paste is an outer shell and the aggregate is an inner core. It appears that the results obtained by the disk model (Eqs. 14, 26, 27, 32, and 33) provide the best estimates for practical purposes.

Stresses capable of causing cracking in the concrete can be produced by shrinkage or temperature change without any external loading.

Finally, it is demonstrated that the compression creep causes a rearrangement of the stresses in concrete by reducing the compressive stresses in the paste and by increasing those in the aggregate particles.

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ANISOTROPY OF CONCRETE AND ITS PRACTICAL IMPLICATIONS

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The influence of anisotropy induced by different methods of casting on the uniaxial tensile and compressive strength of concrete is illustrated, and it is shown that the strength of concrete cast with the axis of loading vertical is about 8 percent less for tension and 8 percent more for compression than that of corresponding concrete cast horizontally. Consequently, the ratio of tensile to compressive strength for concrete cast with the axis vertical is about 15 percent less than that for corresponding concrete cast horizontally. Some practical situations where a knowledge of these effects should influence the evaluation of concrete quality from tests on standard molded specimens, drilled cores, and sawed beams are also discussed.

• THE anisotropic behavior of concrete with respect to its compressive strength has been mentioned in papers published over the past 35 years. However, the opinions and conclusions expressed are not unanimous, different investigators having reached directly opposite conclusions on the sense of the anisotropy. Moreover, none of these investigations indicates whether anisotropic behavior occurs with respect to tensile strength. Only very recently has this problem received any attention. Nevertheless, a knowledge of the effects of anisotropy in both tension and compression is essential to an understanding of the relationships among the strengths of the various standard molded specimens, the strengths of specimens sawed or cored from in situ concrete, and the in situ structural strength in a particular direction. The present paper reviews and supplements previous information on anisotropy in compression, provides additional information on anisotropy in tension, and attempts to assess the influence of mix parameters. The more important practical implications of the conclusions are also discussed.

LITERATURE REVIEW

Neville in a 1959 report (1) illustrated the uncertainty regarding anisotropy at that time. He referred to the work of Gilkey and Leavitt (2), who reported the strength of mortar cubes cast with the axis vertical to be 9 to 13 percent less than that of cubes loaded in the standard manner, and to his own results for $^3/_8$-in. aggregate concrete cubes, which suggested the difference to be 4 to 7 percent in the same sense, although in some cases it was not statistically significant. Neville also mentioned contrary conclusions reached by Mercer (3), who reported a difference of 10 to 20 percent in the opposite sense for mortar cubes, and Johnson (4), who reported the value to be about 5 percent for concrete cylinders. Thus, four separate investigations were equally divided between opposing conclusions at this time. The cause of the discrepancy is not obvious, although it seems possible that capping in the three investigations involving cubes cast with the axis of loading vertical may have been a contributory factor. However, work by L'Hermite (5), who reported a similar difference of 13 percent for cubes, by Bloem (6), who reported an average value of 15 percent for

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cylinders regardless of whether the specimens were capped on both ends, and more recently by Petersons (7), who reported a value of 12 percent for cores, has substantially supported the conclusions of Mercer (3) and Johnson (4) that specimens cast with the axis of loading vertical are stronger in compression than those cast with the axis of loading horizontal. Moreover, it has added weight and generality to their conclusion because of the similar trends observed for cubes, cylinders, and cores.

TEST PROGRAM

The schedule of 23 mixes, given in Table 1, was adopted to permit investigation of the individual influence, if any, of slump, water-cement ratio, and aggregate maximum size on the results. Six 30-in. long prisms made using type 1 cement and gravel aggregate (except as marked) were cast from a single batch of each mix, three with the axis of loading vertical and three with the axis horizontal. The cross sections were 6 x 6 in. for mixes with $1\frac{1}{2}$-in. aggregate and 4 x 4 in. for mixes with $\frac{3}{4}$- or $\frac{3}{8}$-in. aggregate. After the mixes had been moist-cured for 28 days, the uniaxial tensile strength was determined by using a friction grip technique described and analyzed in detail by Johnston and Sidwell (8). The uniaxial compressive strength was determined by using prisms of a height-width ratio 2.0 sawed from fractured sections of the tension specimens, thus eliminating the problem of capping and its possible influence on the results. All strengths quoted are mean values calculated from three tests.

RESULTS AND DISCUSSION

The results for uniaxial tension (Fig. 1) show that the tensile strength of specimens cast with the axis of loading vertical is generally less than that of corresponding specimens cast horizontally. In compression, on the other hand, the strength of specimens cast with the axis vertical is generally greater than that of specimens cast horizontally, as shown in Figure 2. Thus, the effects of anisotropy in tension and compression are opposite in sense. However, the magnitude averages 8 percent in both cases for the 23 mixes. Although no comparative data are available for uniaxial tension, the value of 8 percent for compression is considerably less than the 18 percent calculated from the 8 mixes tested by Bloem (6). The difference is possibly attributable to inadequate compaction associated with the special horizontal cylindrical molds used in his work.

Comparison of mean strengths for both directions of casting to determine the statistical significance of the strength differences shows that the variances do not differ significantly in the F-test, a condition that must be fulfilled before applying either form of the t-test. Whereas the general t-test for difference of means fails to show a difference with a reasonably high probability of being correct, the more discriminating t-test for paired related data shows that the mean difference between the strengths of the two sets of specimens is highly significant, the probability level exceeding 99.9 percent for both tension and compression. Inasmuch as the mean strengths compared for each mix were derived from a single batch of concrete, the latter test is clearly valid, and the result adds strong statistical support to the conclusions visually evident in Figures 1 and 2.

The generality of these conclusions can be qualitatively illustrated for a much wider variety of mixes by comparing the ratio of tensile to compressive strength for specimens cast with the axis of loading vertical to the corresponding ratio for specimens cast horizontally. This comparison tends to accentuate the dissimilarity in the strength characteristics of the two types of specimens because the strength differences for tension and compression are of opposite sense. Thus, if the mean percentage difference of 8 percent for both tension and compression is assumed, the strength ratio for specimens cast with the axis of loading vertical should be about 15 percent $[(1 - 0.08)/(1 + 0.08) = 0.85]$ less than for corresponding specimens cast horizontally. Comparison of the best-fit curves representing the two visibly distinguishable bands of data shown in Figure 3 reveals that this is essentially true over the normal range of compressive strength. And it is significant that, although the data from other investigations (8, 9, 10, 11) do not represent corresponding mixes and include a wide variation in water-cement ratio, aggregate maximum size, grading, and type, parameters that have been

Table 1. Mixes used in investigation.

Aggregate Size (in.)	Slump (in.)	w-c = 0.35		w-c = 0.45		w-c = 0.55		w-c = 0.65	
		Tension	Compression	Tension	Compression	Tension	Compression	Tension	Compression
1½	0	1.015	1.123					0.902	1.147
	2	0.989	1.036	0.958	1.086	0.988	1.000	1.049	1.157
	4	1.010	1.122					0.937	1.051
¾	0	0.841	1.047					0.924	1.169
	2	0.907	1.070	0.844	1.082	0.928	1.037	0.899	1.052
	4	0.856	1.072					0.878	1.049
⅜	2	0.931	1.107	0.972	1.094	0.872	1.098	0.852	1.025
¾[a]	–[b]	0.881	1.177	0.779	0.945	0.900	1.116		

[a]Crushed basalt aggregate. [b]Not recorded.

Figure 1. Comparative tensile strengths of specimens cast vertically and horizontally.

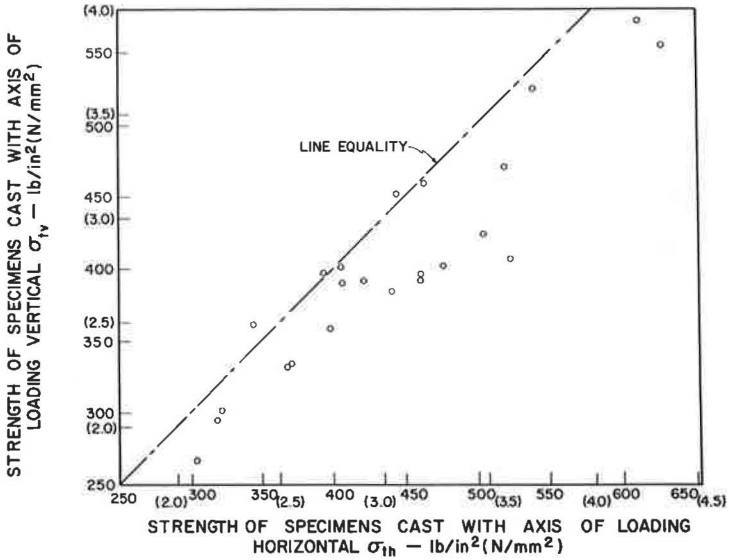
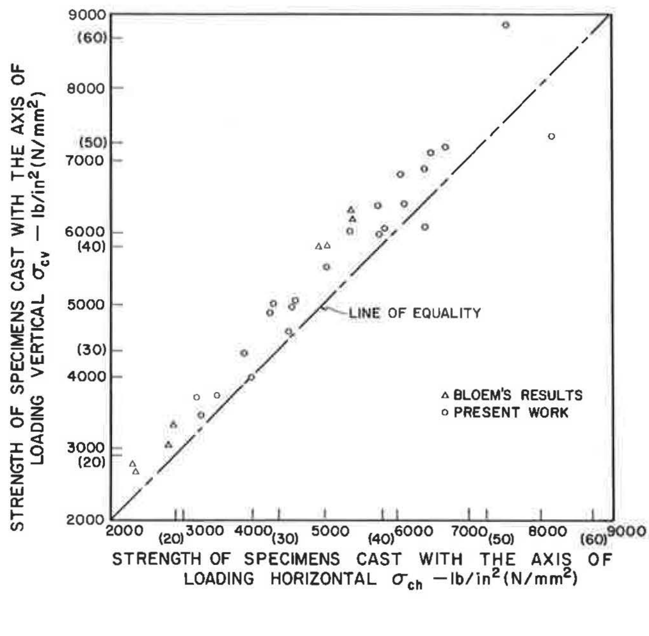


Figure 2. Comparative compressive strengths of specimens cast vertically and horizontally.



shown to influence the ratio (8), the overriding influence of direction of casting is still apparent. Also, the inaccuracy of using general rule-of-thumb factors to estimate tensile strength from the results of compression tests is again emphasized.

The strength differences associated with each mix are given in Table 1. The range of the data is quite large because both mean strengths are subject to a coefficient of variation that averaged 5.4 percent for tension and 6.4 percent for compression, and it is only when averages are calculated with respect to each mix parameter, as given in Table 2, that the influence of aggregate size, slump, and water-cement ratio can be assessed. From these values, it is evident that the magnitude of the strength differences associated with anisotropy is not clearly dependent on any of these mix parameters and for practical purposes can be regarded as constant and equivalent to 8 percent for normal weight structural concretes. Water gain, or the tendency of water to concentrate underneath the aggregate particles as cast (thus creating areas of weak cement-aggregate interface), a phenomenon first observed by Gilkey (12), seems to explain the observed trends. His statement that it occurs "even in relatively dry and stiff mixtures" is compatible with the lack of influence of mix parameters, and the opposite sense of the strength differences in tension and compression can be explained as follows. In a tension specimen cast with the axis of loading vertical, the weak interface is primarily parallel to the failure surface, thus lowering the strength relative to that of a corresponding specimen cast horizontally in which the interface is perpendicular to the failure surface. In contrast, in a compression specimen cast with the axis of loading vertical, the weak interface is primarily perpendicular to the longitudinal cracks that induce failure, thus tending to increase the strength relative to that of a corresponding specimen cast horizontally in which the interface is parallel to the failure cracks. These areas of weak interface are clearly visible as whitish zones on the failure surface of the lower portion of a vertically cast prism tested in tension and are not present on the failure surface of the upper portion, as shown in Figure 4.

PRACTICAL IMPLICATIONS

Assuming that the strength difference associated with anisotropy is about 8 percent for both tension and compression, as reported above, its influences on the evaluation of concrete properties in practice are as follows:

1. The relationship between the compressive strength of drilled cores and standard molded cylinders is subject to the effect of anisotropy when the direction of coring is horizontal, as is normal for vertical walls and columns, but not when it is vertical as in the case of slabs and pavements. Therefore, application of a correction factor of about 0.92 is appropriate in the former case, not merely a report of the direction of loading with respect to the horizontal as required by ASTM C 42. Furthermore, if the core strength is to relate to in situ strength in the direction of applied stress, rather than standard cylinder strength, a correction factor of 0.92 should be applied to the core strengths in the case of slabs and pavements and a factor of 1.08 in the case of walls and columns.

2. The relationship between the splitting tensile strength of cores and standard cylinders is not subject to the influence of anisotropy when the cores are drilled vertically, as for slabs and pavements, but could be affected by it in varying degrees depending on the test orientation when the cores are drilled horizontally, as for walls and columns. This latter situation is probably rare (e.g., the vertical walls of a pressure vessel), inasmuch as the splitting test is normally applied only to pavement work.

3. The relationship between the flexural strengths of sawed and molded beams with their longitudinal axis parallel to the slab or pavement is not subject to the influence of anisotropy. However, the value for compressive strength measured using portions of broken beams, as described in ASTM C 116, depends on whether the specimen is loaded top-to-bottom or side-to-side, the latter case giving a lower value. The specification allows either condition for square cross sections and requires the top-to-bottom condition when the depth-breadth ratio is greater than unity and the side-to-side condition when the depth-breadth ratio is less than unity. Realistic interpretation of

Figure 3. Influence of direction of casting on the ratio of uniaxial tensile to compressive strength [Komlos' data (11) based on uncorrected cube strengths].

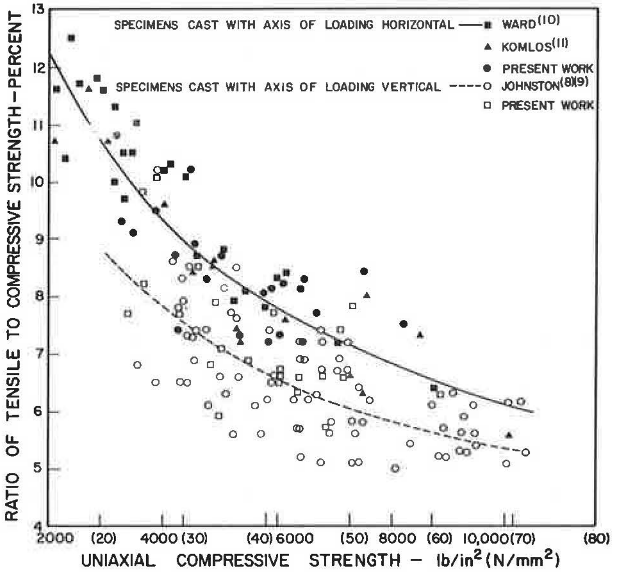


Figure 4. Failure surfaces in a vertically cast prism after testing in uniaxial tension.

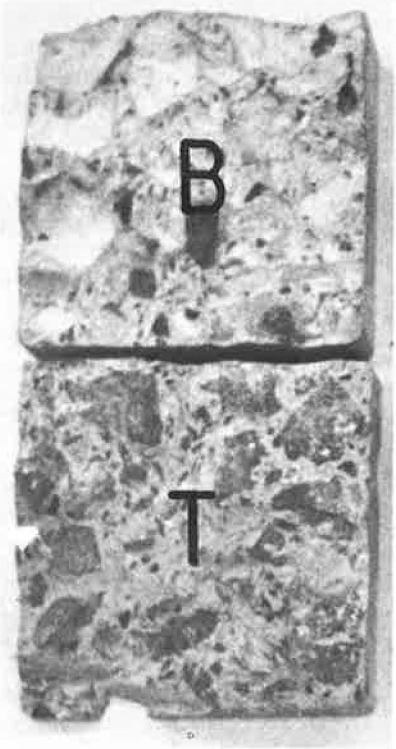


Table 2. Average values of ratio of the strengths of vertically cast prisms to those of horizontally cast prisms.

	By Aggregate Size (in.)			By Water-Cement Ratio				By Slump (in.)			
Test	$1\frac{1}{2}$	$\frac{3}{4}$	$\frac{3}{8}$	0.35	0.45	0.55	0.65	0	2	4	Overall
Tension	0.981	0.875	0.907	0.929	0.888	0.922	0.920	0.921	0.932	0.920	0.918
Compression	1.090	1.074	1.081	1.094	1.052	1.063	1.093	1.060	1.070	1.074	1.080

the results therefore requires making appropriate corrections to account for the effects of both height-width ratio and anisotropy in each particular case.

4. With regard to the cube specimen used as a standard in other countries or in research work to measure compressive and splitting strength, the relationship between cube and cylinder compressive strength is subject to the opposing effects of anisotropy and height-width ratio, the latter being strength-dependent. In addition, the splitting strength depends on whether the cube is loaded top-to-bottom or side-to-side, the latter case giving a lower value, as shown recently by Soshiroda (13).

CONCLUSIONS

1. The strength of concrete cast with the axis of loading vertical averages 8 percent less for tension and 8 percent more for compression than that of corresponding concrete cast horizontally.
2. The magnitude of the strength difference is independent of aggregate size, water-cement ratio, and slump.
3. The ratio of tensile to compressive strength for concrete cast with the axis of loading vertical is about 15 percent less than that of corresponding concrete cast horizontally.

ACKNOWLEDGMENT

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MOISTURE PENETRATION IN CONCRETE WITH SURFACE COATINGS AND OVERLAYS

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Tests were made on concrete specimens coated with four waterproofing materials to determine how deeply the coatings penetrated into the concrete. Also, coated surfaces and overlaid specimens were ponded with salt water and tap water to determine the effectiveness of each in preventing the penetration of moisture into the concrete specimens. Freeze-thaw tests were made on asphaltic overlays to determine the effect of freeze-thaw cycling on the overlays and the portland cement concrete beneath the overlays. Shear tests were made to determine the shear strength of concrete overlays bonded to concrete test blocks. It was found that the deepest penetration of coatings, 0.054 to 0.062 in., was made by a mixture of linseed oil and kerosene. No damage was found under the asphaltic overlay after 59 freeze-thaw cycles. Shear bond strengths ranged from 61 to 578 psi when the cube surfaces were treated with surface coatings and from 367 to 597 psi when the surfaces with coatings were sandblasted before overlaying.

•MANY of the problems associated with durability of concrete bridge decks begin with the entry of water into the concrete through cracks and pores. Soluble chlorides are sometimes carried by the water. Electrochemical corrosion of reinforcing steel is enhanced by concentration of chloride ions resulting from chloride entry with water. De-icing salts and seawater are two common sources of chlorides. The depth of penetration of water into concrete is of interest because salt solutions that reach reinforcing steel will probably cause corrosion. The accumulation of corrosion products is sometimes so great that spalling (tensile failure of surface concrete) occurs in the vicinity of the corrosion.

Scaling of concrete surface mortar is the result, too, of water penetration. Alternate freezing and thawing of the water in pores and cracks cause gradual surface deterioration by flaking away the mortar.

Concrete that is kept dry is almost certain to be free of corrosive and freeze-thaw damage sometimes found in concrete bridge decks. This report covers a study to determine the depth that water penetrates concrete with and without protective coatings and with overlays. Tests were made also to determine how deeply the coating materials penetrated into the concrete. The depth of penetration is of interest particularly where abrasion from traffic is expected.

Tests and test results that were used in selecting the surface treatments for this study have been reported elsewhere (1). The overlay systems selected for this study are some of those that have been or are being considered for use by the Texas Highway Department.

Concrete specimens that had been coated with waterproofing materials were examined to determine how deeply the coatings penetrated. In other tests, coated surfaces and overlaid specimens were ponded with 5 percent salt water (the salt solution contained 5 percent sodium chloride and 95 percent tap water by weight) and with tap water

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to determine the effectiveness of each to resist the penetration of moisture into the concrete specimen. Table 1 gives the tests made in the study and the purpose of each test.

It was found that the overlay systems were in general effective in resisting moisture penetration, that the waterproofing materials served to slow down moisture penetration, and that the depth of penetration of the waterproofing materials differs. No correlation between depth of penetration and protection provided against freeze-thaw action was evident in the laboratory study.

MATERIALS AND TESTS

The portland cement concrete slabs, 10 × 10 × 2 in., were made of natural sand and gravel. The maximum size of aggregate was $^3/_4$ in. The concrete mix design for the slabs was as follows: gravel, 1,950 lb; sand, 1,295 lb; type 3 cement, 516 lb; and water, 300 lb.

Test Series 1

In laboratory tests, several materials were investigated as possible depth indicators. Those materials include oil-base dyes, sulfuric acid, printer's ink, and phenolphthalein.

Oil-Base Dyes—Three colors of oil-base dyes (red, orange, and blue) were used. One gram of the dye was added to 100 ml of coatings 2-a and 8-a prior to applications. After the coatings had dried, eight to 10 measurements of penetration depth were made over the 10-in. long broken and sawed surfaces.

Sulfuric Acid—A 50 percent solution of sulfuric acid was used as an indicator of depth of penetration with coating 2-a (method suggested by William Kubie, Oilseed Crops Laboratory, U.S. Department of Agriculture). It was applied evenly over the sawed and broken faces at room temperature, and the specimens were then baked in a 270-deg oven for $2\frac{1}{2}$ hours. After removal from the oven, the specimens were allowed to cool before observations were made.

Printer's Ink—A mixture of 20 percent printer's ink was made with coatings 2-a, 7, and 8-a, and the mixture was applied to the top of the blocks. After drying, transverse surfaces, both broken and sawed, were made from each block. These surfaces were then observed under ultraviolet light to determine the penetration of the flourescent ink.

Phenolphthalein—Phenolphthalein has the property of reacting with alkaline substances to indicate a pink color. Thus, in intimate contact with alkaline concrete particles, a pink color is seen. If the particles are coated to prevent intimate contact with alkali, no such color is indicated. An indicator was prepared by mixing 5 grams of phenolphthalein crystals, 500 ml of isopropyl alcohol, and 500 ml of distilled water. This solution was mixed with coatings 2-a and 8-a to produce a mixture of 20 percent indicator and 80 percent coating. Because the indicator would not mix with coatings 7 and 9, a mixture of indicator was made with coating 2-a. The resulting mixture was applied evenly on the sawed and broken surfaces of the blocks coated with coatings 7, 9, and 2-a. A mixture of coating 8-a and indicator was applied evenly to the sawed and broken surfaces of the blocks receiving coating 8-a.

Both sawed and broken surfaces were observed by the aid of a variable-power microscope. Specimens were mounted on a traversing table, and the distance traversed from the surface to the observed penetration limit was read from a micrometer on the traversing mechanism. Depth measurements to 0.001 in. were recorded for the coatings given in Table 2 by using phenolphthalein as the depth indicator.

Test Series 2

Specimens containing Monfore (2) moisture gauge wells were used in this test series. A relative humidity probe inserted into the moisture wells was used to determine relative humidity at depths of $^1/_8$, $^2/_8$, and $^3/_8$ in. below the top surface.

The specimens in this test series were coated with coatings 0, 2-a, 7, and 8-a. After the coatings had dried, 8-in. diameter rings were bonded to the tops of the blocks.

All surfaces not ponded were exposed to laboratory air of approximately 70 F and 50 percent relative humidity. One set was ponded with a ½-in. depth of tap water and an identical set with 5 percent salt water. Relative humidity versus time was recorded until each moisture well had reached 100 percent relative humidity.

Test Series 3

In this series of tests, overlays were bonded to the surfaces of 10-in. square blocks to determine their effectiveness in resisting the passing of tap water and 5 percent salt water into the concrete base. All surfaces not ponded were exposed to laboratory air as in test series 2. The following overlays were included:

1. Epoxy mortar, ½ in. thick, made up of 15 percent GuardKote 250 epoxy and 85 percent natural sand was used for this overlay. The epoxy and sand were mixed in a 5-gal can using a beater type of blender attached to an electric drill. After being mixed for 5 min, the mortar was poured onto the block surfaces and smoothed. When the mortar lost its tackiness, it was rolled to provide compaction. After 1 day, the 8-in. diameter rings were bonded to the overlay, and relative humidity data were recorded until the overlays were 60 days old.
2. A ½-in. thick polyester resin mortar made of 15 percent commercial polyester resin and 85 percent natural sand was used for this overlay. The procedure for preparing the overlay was the same as for the epoxy except that a wood maul was used for compaction. A wood screed was then used for strike off.
3. Asphaltic concrete, 1½ in. thick, was used for this overlay. A seal coat of 120 to 150 penetration asphalt cement and intermediate-grade synthetic lightweight aggregate was applied to 10-in. square blocks. After 3 days, a tack coat of EA-HVMS (emulsified asphalt, high viscosity medium setting) with 2 percent latex rubber solids was applied to the seal coat. Time was allowed for the EA-HVMS to break, i.e., to permit water to evaporate; then the hot mix was applied on top of the tack coat. The hot-mix overlay was compacted 1 min by static pressure from a hydraulic ram with a force of 14,000 lb over the 10-in. square area.
4. A 1½-in. thick portland cement concrete overlay was bonded to 10-in. square block surfaces with a portland cement grout. The overlays were cured in a 73 F, 100 percent relative humidity chamber for 7 days, after which the overlaid blocks were placed in a 73 F, 50 percent relative humidity chamber. After 7 days in this chamber it became apparent that the drying would not be complete in time for the tests. The specimens were then removed to a 140 F, 25 percent relative humidity chamber where they remained 25 days until the relative humidity in the specimens reached 65 to 70 percent. The containing rings were then bonded to the overlay surface after the 25-day drying period. Tap water and 5 percent salt water were ponded in the rings, and relative humidity data were recorded. When the relative humidity in the moisture wells reached approximately 95 percent, the rings were removed and the overlaid blocks were returned to the 140 F, 25 percent relative humidity chamber for 14 days to reduce the relative humidity to about 70 percent. Coating 2-a was then applied to the overlays, and the test was repeated.

Test Series 4

In this series of tests, asphaltic overlays were bonded to the surfaces of the 10-in. square blocks as described in test series 3. Eight-in. diameter rings were bonded to the surfaces of the overlays, and 5 percent salt water was ponded within the rings. All surfaces not ponded were exposed to ambient laboratory conditions. The blocks were frozen in a 0 F chamber and were thawed in a 40 F chamber. One complete freeze-thaw cycle required 12 hours, and the cycling continued throughout the test. Once each week the blocks were removed to the laboratory where the old 5 percent salt water was discharged. The surfaces were flushed with tap water and brushed to remove any loose particles. They were then visually inspected for any signs of deterioration. If no signs of deterioration were found, the blocks were recharged with 5 percent salt water and the cycling was begun again. Cycling continued until 59 freeze-thaw cycles were completed.

Test Series 5

This series of freeze-thaw tests was made to determine the effect of coatings 0, 2-a, 7, and 8-a on the shear bond strength of old and new portland cement concrete.

TEST RESULTS

Results of tests are given in this section in the form of tables, charts, and discussion.

Test Series 1

Several methods were investigated in measuring the depth of penetration of various coatings. The oil-base dyes mixed well only with coatings 2-a and 8-a. On the broken and sawed surfaces, orange was the only effective indicator.

The 50 percent solution of sulfuric acid applied evenly to broken faces of blocks that had received coating 2-a produced carbon, black in color, when it reacted with the oil. The color gradually faded with depth into the block, making the limit of penetration difficult to identify under close inspection. Gast, Kubie, and Cowan (3) have reported using a 50 percent solution of sulfuric acid as an indicator of coating penetration. In tests on concrete with a sand and gravel to cement ratio by weight of 3, they reported sharp, even lines of penetration. In tests reported here, concrete with a sand and gravel to cement ratio by weight of 5.4 was used. Different coatings and different concretes possibly account for the difference in the results of the separate investigations.

When faces of blocks coated with mixtures of printer's ink and coatings 2-a and 8-a were viewed under ultraviolet light, fluorescence of the ink was evident. The depth was not well defined, however, making measurements difficult and accuracy subject to question. This test was not used for record measurements.

A chemical reaction that produces a distinctive pink color takes place when phenolphthalein and clean portland cement concrete come in contact. The sensitive solution caused no color change in that portion of the concrete penetrated by the coatings used in these tests, and it proved to be a good indicator of penetration depth. Because the phenolphthalein could not be made to mix with coatings 7 and 9, the mixture of phenolphthalein and coating 2-a was used to determine the penetration depths of those coatings.

Table 3 gives the penetration data recorded for coatings 2-a, 7, 8-a, and 9. Each of the coatings was applied to two blocks. Each block was then cut twice at random positions to give four faces from which penetration data were recorded. A statistical analysis of the data showed that there was no significant difference between blocks with a given treatment, no significant difference between faces within blocks, and a significant difference between coatings. Confidence limits for the mean penetration depth at the 95 percent level are given for each of the coatings.

Stewart and Shaffer (4) found no apparent correlation between depth of penetration of the sealer and the final rating of the concrete. Data presented here along with data from an earlier report (1) concur with that finding. In the Stewart and Shaffer test, penetration of linseed oil and mineral spirits was found to be less than 0.01 in. Penetration for linseed oil and kerosene reported above was found to be about 0.05 in. The difference in concrete used in the two tests and the difference in indicators possibly account for the difference in depth of penetration.

Heskin and Rheineck (5) have reported using a 2 percent paste of phenolphthalein indicator in a cellulose gel for determining depth of penetration of coatings. The paste became pink on the concrete areas and remained unchanged on the area penetrated by the coating.

Verbeck (6) discussed a phenolphthalein color test for estimating the depth of penetration of carbon dioxide in various types of portland cement specimens. He concluded that there appears to be no readily apparent physicochemical basis for this test and presumably indicates a combined net effect of extensive carbonation and leaching of alkalies.

In the tests reported here, a comparison of coated and uncoated specimens was not made because it was believed that the indicator would react with clean concrete, i.e., an uncoated concrete. If the concrete were coated, on the other hand, there would be no contact between the alkaline material and the phenolphthalein and, hence, no reaction. Later, tests were made in response to the suggestion of a reviewer that the neutralized products of carbonated alkalies would not react with the phenolphthalein indicator. In those tests, specimens of the same size and concrete mix as the original ones were prepared. After 7 days' moist curing and 21 days' dry curing at 73 F and 50 percent relative humidity one-half of the top surface of each of three specimens was coated with coating 2-a and the other half was left uncoated. The phenolphthalein indicator was applied on both halves of the specimens after they were broken to expose the interior of the blocks. The uncoated blocks displayed the pink color all the way to the top surface, whereas the coated ones had a line of uncolored material along the top surface. That uncolored line was taken to be material coated by the penetrating coating, and the interpretation given to the color on the uncoated blocks was that all of the material was still alkaline and, thus, reactive with the phenolphthalein.

Test Series 2

Moisture penetration into dry portland cement concrete occurs quickly when no protective coating is provided. The moisture migration to the depth of a moisture well was evident when free moisture appeared on the gauge when inserted in that well. Tap water and salt water both can penetrate to a depth of $^3/_8$ in. within 3 hours, the penetration by 5 percent salt water being somewhat faster than that of tap water. Data shown in Figure 1 represent only one set of data and are intended to show a trend. The trend was the same for other data, but the magnitudes were different. When a protective coating is provided, the time required to penetrate $^3/_8$ in. can be extended to several days. Coating 7 provided approximately the same resistance to both tap and salt water, whereas coatings 2-a and 8-a were less resistant to tap water than they were to salt water. Of the three coatings tested, coating 8-a was the most resistant to both tap and salt water. Table 4 gives the times of penetration of 100 percent relative humidity to depths of $^1/_8$, $^2/_8$, and $^3/_8$ in. when various coatings are used.

Test Series 3

Results from this series of tests indicate that thin, bonded overlays can be effective in resisting moisture penetration into the concrete beneath the overlay. Generally, the ponded water remained on the overlays approximately 60 days. It is not likely that water would stand for any appreciable length of time on a well-drained bridge deck and even in low spots for more than 2 or 3 weeks. The 60-day time period then may be taken as a safe upper limit in time for surfaces to be covered with tap or salt water.

Thin, bonded overlays generally were effective in resisting moisture penetration into the concrete beneath them. The blocks covered with epoxy mortar overlays showed no appreciable change in relative humidity when subjected to ponded tap water, but there was an 8 to 10 percent increase in relative humidity over the test period when the blocks were subjected to ponded 5 percent salt water (Fig. 2). In blocks covered with polyester mortar overlays with a 30-day age, relative humidity of 90 percent gradually decreased to 85 percent at 60 days under ponded water. The only reason that can be offered for this decrease in relative humidity is that the sides and bottom of the concrete blocks were not sealed. In their exposure to the laboratory air, some moisture near the surfaces was lost, no doubt, but the moisture well nearest any exposed surface was covered with approximately 4 in. of concrete (Fig. 2).

The blocks covered with the $1\frac{1}{2}$-in. thick portland cement concrete overlays (Fig. 3) reacted practically the same when subjected to ponded tap and 5 percent salt water. The relative humidity had reached 90 to 95 percent after 7 days and remained almost constant at that level for an additional 20 days. The blocks were then stored in a controlled environment of 140 F and 25 percent relative humidity for 14 days. The relative humidity of the blocks was reduced from approximately 95 percent to approximately 70 percent. The blocks were then coated with coating 2-a. Results after the coating was applied are shown in Figure 4.

Table 1. Summary of tests and their objectives and results.

Test Series	Test	Objectives	Results
1	Penetration	To determine indicator(s) that may be used to ascertain depth of penetration and to determine the penetration of selected coatings	A mixture of phenolphthalein crystals, isopropyl alcohol, and distilled water was selected for an indicator. Penetration depths of the coatings ranged from 0.013 to 0.054 in.
2	Moisture penetration	To determine the effectiveness of waterproofing materials in retarding moisture penetration	Tung mix and LO mix[a] were most effective. Both more resistant to saltwater solution.
3	Moisture penetration	To determine the effectiveness of overlay systems in retarding moisture penetration	All overlay systems except the untreated concrete system were effective in resisting moisture penetration.
4	Freeze-thaw	To determine the effect of freeze-thaw action on asphaltic concrete overlay systems	No ill effects were noted.
5	Shear	To determine the effect of waterproofing materials on bond stress	All materials tested reduced the bond stress. Sandblasting of treated surfaces prior to overlaying helped to restore bond strength.

[a]LO mix is boiled linseed oil mixed with kerosene, 50 percent of each by volume.

Table 2. Description of coatings.

Coating Number	Description
0	No coating
2-a	Two coats of 50 percent linseed oil and 50 percent kerosene
7	Two coats of Thompson's Water Seal
8-a	Two coats of 50 percent tung oil and 50 percent kerosene
9	One coat of EpoXeal with a touch-up coat

Table 3. Indicated penetration of various surface coatings in concrete.

Coating Number	Description	95 Percent Confidence Limits for Mean Penetration Depth (in.)
2-a	Two coats of 50 percent linseed oil and 50 percent kerosene	0.054 to 0.062
7	Two coats of Thompson's Water Seal	0.013 to 0.017
8-a	Two coats of 50 percent tung oil and 50 percent kerosene	0.041 to 0.045
9	One coat of EpoXeal with a touch-up coat	0.027 to 0.031

Note: N = 160, using the phenolphthalein indicating solution.

Figure 1. Penetration time for ponded tap and salt water on coatings 0, 2-a, and 8-a.

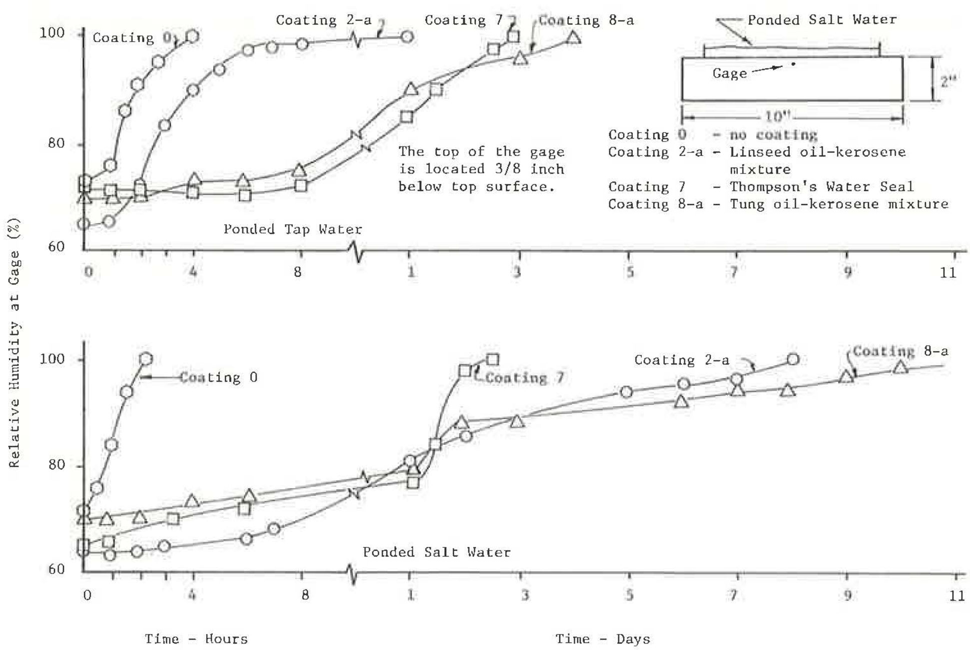


Table 4. Time for penetration to reach 100 percent relative humidity.

Coating Number	Type of Ponded Water	Penetration Time at Various Depths		
		$\frac{1}{8}$ In.	$\frac{2}{8}$ In.	$\frac{3}{8}$ In.
0	Tap	60 min	110 min	175 min
	Salt	20 min	60 min	150 min
2-a	Tap	30 min	7 hours	1 day
	Salt	90 min	$1\frac{1}{2}$ days	8 days
7	Tap	21 hours	27 hours	$2\frac{3}{4}$ days
	Salt	24 hours	27 hours	$2\frac{1}{2}$ days
8-a	Tap	2 hours	1 day	5 days
	Salt	8 days	10 days	13 days

Figure 2. Penetration time for ponded salt water on epoxy mortar overlay.

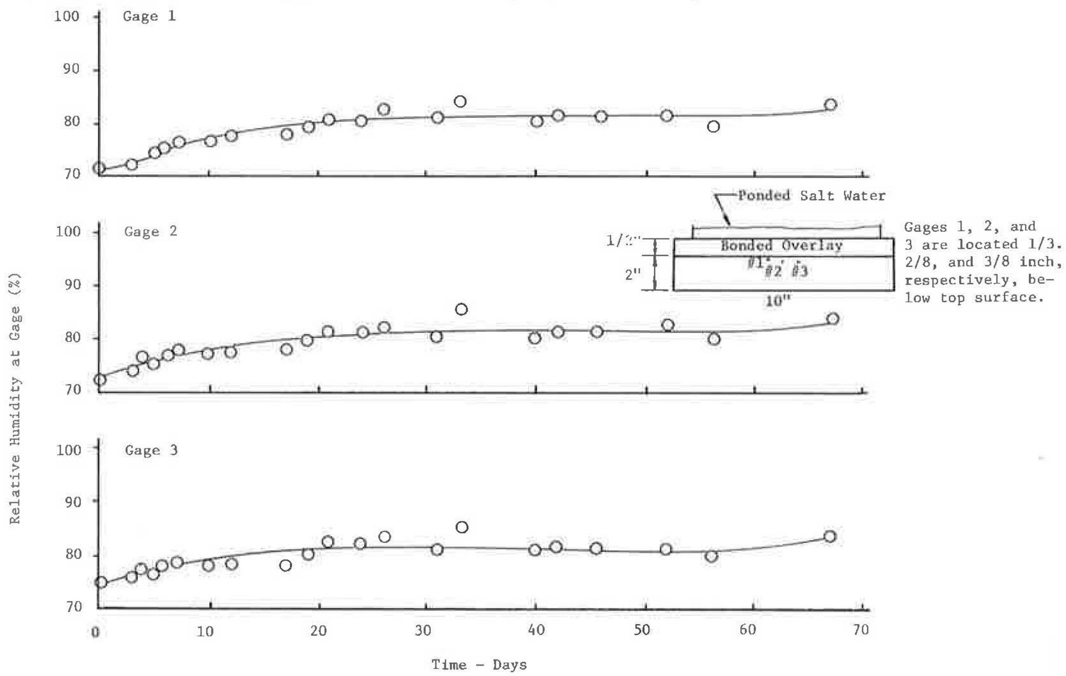
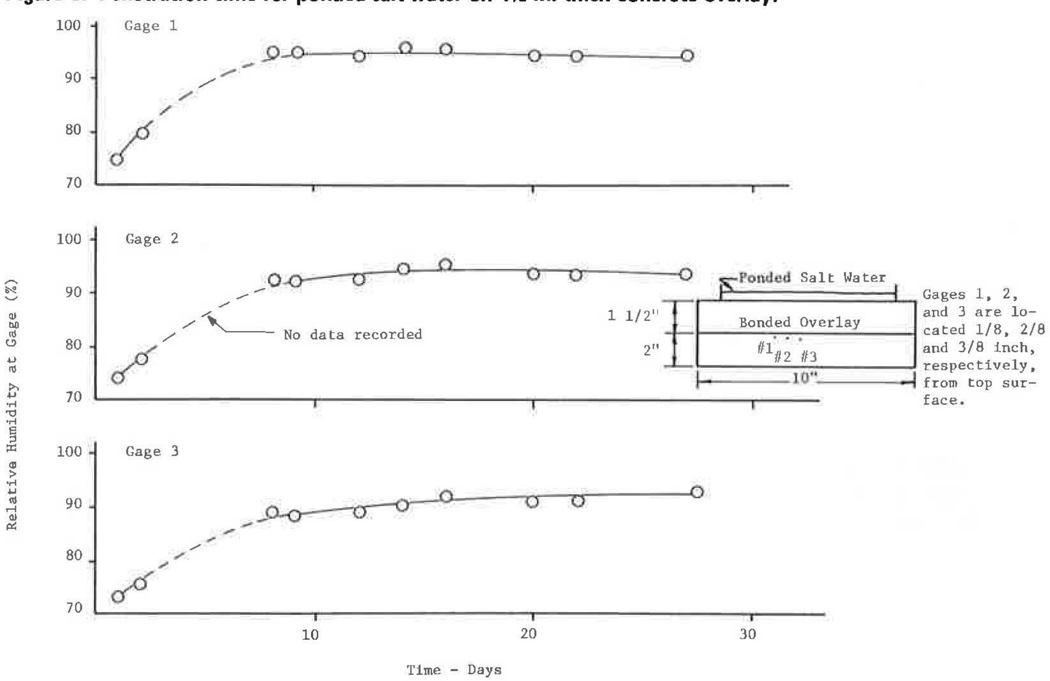


Figure 3. Penetration time for ponded salt water on 1½-in. thick concrete overlay.



The 1½-in. thick asphaltic overlays were tested in the same manner as the other overlays. The relative humidities remained practically constant for the full 60-day period under ponded salt water. Figure 5 shows a typical plot of those data. It was assumed that the relative humidity in the moisture wells, being located near the ponded surface, was not influenced by exposed surfaces of the specimen.

Test Series 4

Tests reported in an earlier report (1) have shown that deterioration can develop under a seal coat when alternately frozen and thawed. In tests of this series, freeze-thaw cycling of the asphaltic overlays continued for 59 cycles. After 25 and 38 cycles, one of each type of overlay was removed in order to examine the concrete surface, which showed no signs of deterioration. The penetration of the 5 percent salt water into the broken overlay was observable from its damp and darker appearance, as compared to the remaining overlay, and, when measured with a ruler, penetration varied between ¼ and ½ in. No apparent distress was evident when the blocks were removed from the test.

Test Series 5

Surface condition plays a very important part in bonding of a portland cement concrete overlay. Sinno and Furr (7) showed that motor oil on the overlaid surface greatly reduces the bond capacity between new and old concrete when little or no surface preparation is made.

Overlays were sheared from cubes that had received coatings 0, 2-a, 7, and 8-a with no surface preparation and from cubes that had received coatings 2-a and 7 followed by sandblasting as the surface preparation (Table 5). The cube was fixed in a jig, and a shearing force was applied to the overlay. This force, parallel to the interface between overlay and base specimens, was gradually increased until the specimen failed. The bond stress, in force per unit area of bonded surface, was found for each specimen. When no surface preparation was made, the average bond stress was 578 psi for uncoated cubes, 61 psi for coating 2-a, 88 psi for coating 7, and 267 psi for coating 8-a. Cubes coated with coating 2-a and others coated with coating 7 were sandblasted after the coatings had dried. The resulting average bond stress for coating 2-a cubes was 367 psi and 597 psi for the cubes with coating 7.

Coating 8-a was not included in these tests on sandblasted surfaces. The shear bond strength of overlay over this coating without sandblast treatment was 267 psi. It has been established by Gillette (8) that 200 psi is an acceptable bond strength value for pavement overlay. Theoretical calculations (9) have shown that the bond between a 2-in. overlay and a 7-in. thick bridge slab when flexed under an AASHO H20 wheel is approximately 64 psi. Inasmuch as the 267 psi strength is greater than either of these values, further tests were not made. Had the test been made, it seems reasonable to assume that the bond stress would not have been lowered but increased.

The sandblasting seemed to have removed all of coating 7 but not all of coating 2-a. This is supported by data from test series 1 in which the depths of penetration were measured. Sandblasting removed approximately $^1/_{32}$ in. of the surface that included all of the concrete penetrated by coating 7 but only about 60 percent of that of coating 2-a.

CONCLUSIONS

The following conclusions are made on the basis of tests performed in this investigation. An explanatory note follows each conclusion.

1. Surface coatings of a mixture of 50 percent linseed oil and 50 percent kerosene penetrated almost $^1/_{16}$ in. into the portland cement concrete used in these tests. The two-coat application of boiled linseed oil and 50 percent kerosene penetrated to a depth of 0.05 to 0.06 in.; the mixture of 50 percent tung oil and 50 percent kerosene applied in two applications penetrated approximately 0.04 in.; EpoXeal, an epoxy penetrant, to 0.03 in.; and Thompson's Water Seal, to approximately 0.015 in.

Figure 4. Penetration time for ponded salt water on 1½-in. concrete overlay coated with coating 2-a.

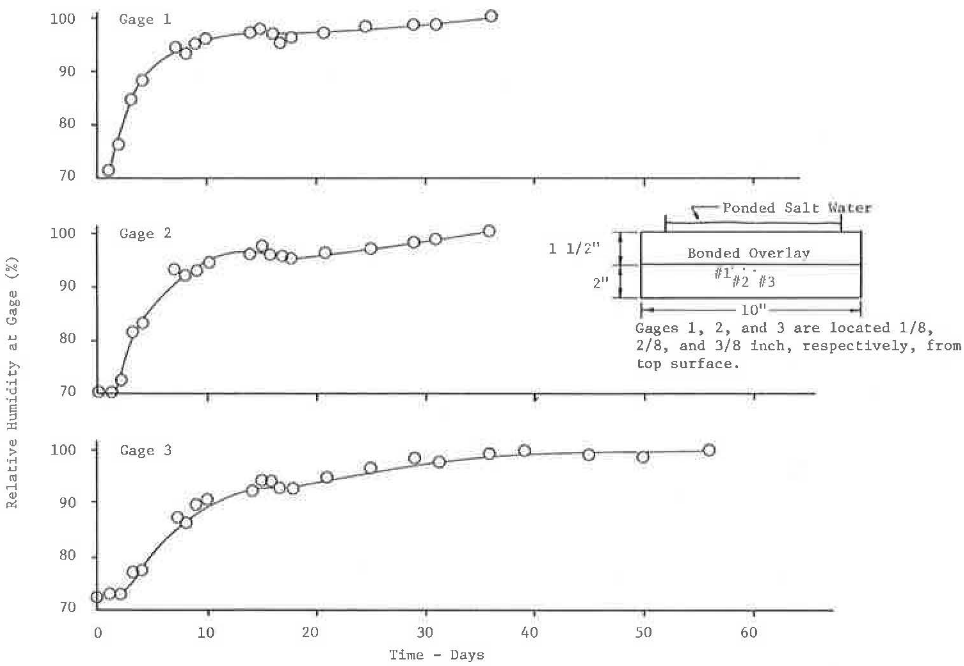


Figure 5. Penetration time for ponded salt water on asphaltic concrete overlay.

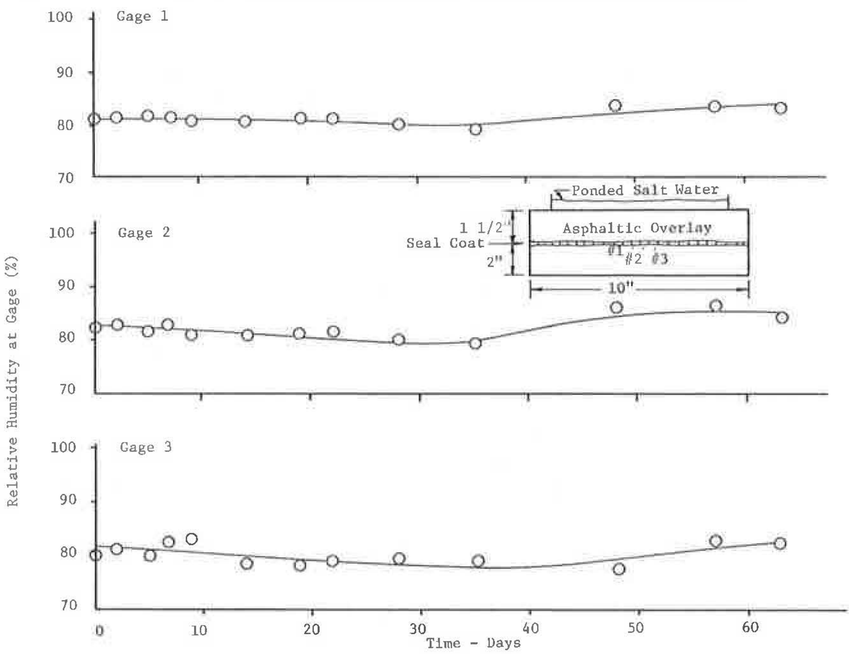


Table 5. Shear bond strength.

Coating Number	Surface Preparation	Bond Stress[a] (psi shear)	Range of Bond Stress (psi)
0	None	578	506 to 658
2-a	None	61	34 to 81
7	None	88	65 to 100
8-a	None	267	235 to 314
2-a	Sandblast[b]	367	255 to 650
7	Sandblast[b]	597	530 to 694

[a]Average of five specimens.
[b]Sandblasted after coating had been applied and curing.

2. Surface treatment of concrete with a mixture of kerosene and either boiled linseed oil or tung oil (50-50 basis) delays the penetration of both tap water and 5 percent salt water into concrete. Relative humidity measurements taken at $^3/_8$ in. from the horizontal surface of the portland cement concrete showed that tap water penetrated $^3/_8$ in. of uncoated concrete within 3 hours, whereas from 1 to 5 days were required for tap water penetration of the linseed-oil- and tung-oil-treated surfaces. Penetration of 5 percent salt water to $^3/_8$ in. was delayed up to 13 days.

3. Thin, uncracked, nonporous resinous and rubberized asphaltic overlays provide effective barriers to entry of moisture into concrete, and a portland cement concrete overlay retards the entry of moisture. Overlays of epoxy mortar and polyester mortar concretes $^1/_2$ in. thick were effective in maintaining the relative humidity below about 85 percent when ponded over with 5 percent salt water for the 60-day period of the test. The $1^1/_2$-in. thick portland concrete overlay retarded the flow of water for a period of about 10 days. A rubberized asphaltic overlay filled with expanded shale, limestone, and field sand provided a nearly complete seal to the laboratory concrete.

4. Surface treatments designed as moisture barriers can measurably reduce the shear bond strength of an overlay applied to the freshly treated concrete surface. The shear bond strengths of a portland cement concrete overlay placed on a concrete surface treated with the linseed oil-kerosene mixture, tung oil-kerosene mixture, and Thompson's Water Seal are reduced from about 578 psi for the untreated surface to 61, 267, and 88 psi respectively for the freshly treated surfaces. Sandblast conditioning of the linseed-oil- and Thompson's Water-Seal-treated surfaces restored them to 367 and 597 psi shear strength respectively.

5. No freeze-thaw deterioration developed after 59 freeze-thaw cycles on concrete surfaces overlaid with the rubberized asphalt used in the tests.

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CONCRETE VARIABLES AND CORROSION TESTING

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We partially submerged 710 reinforced concrete blocks in a saturated sodium chloride solution. Based on the test criterion that a sufficient quantity of chloride-absorbed concrete causes the steel to change from a passive to an active or corroding half-cell potential, the test results showed that increasing the cement factor and increasing the length of water curing increased the time to an active half-cell potential. Steam curing of the concrete resulted in a reduction of the time to an active half-cell potential as well as a reduction in absorption as compared to just water curing. The test procedure used verified capillary action as the primary mechanism of water absorption. The chloride content of the concrete was determined. Of the three tested admixtures and corrosion inhibitors, only pozzolan appeared to result in a significant benefit even though this concrete had the greatest absorption and also the greatest drying shrinkage.

•THE California Division of Highways reported in 1957 the causes of corrosion of steel in concrete (1). In 1963, an empirical equation was developed from field and laboratory data for estimating the time to corrosion of embedded steel (2). These works indicated that additional data were required regarding the influence of curing, admixtures, and other concrete design variables when a structure is designed for a specific maintenance-free life (2).

Based on a literature survey, it was found that at least 15 different methods have been used for testing the corrosion of steel in concrete (3). In general, some of the methods are as follows:

1. Exposure to tidal water (4, 5),
2. Normal outdoors (5, 6, 7, 8, 9, 10),
3. Laboratory, high humidity (6, 8, 11, 12),
4. Laboratory, low humidity (6, 13),
5. Alternate immersion (6, 12),
6. Alternate but partial immersion (14, 15),
7. Variable salt, moisture, temperature (16),
8. Salt spray cabinet (17),
9. Partial covering with a wet towel (13),
10. Flow of water vapor (18),
11. Periodic spraying with salt water (18),
12. Immersion in water (12),
13. Partial immersion (9),
14. Dry cellar (19), and
15. Impressed voltages (1).

In effect, a test method for obtaining precisely reproducible results even by the same investigator (20, 21) has not been clearly established.

In the selected partial-immersion corrosion test of the reinforced concrete blocks, there are two ways to measure the corrosion phenomenon: One is to measure the half-

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cell potential of the steel, and the other is to visually inspect the concrete for cracks that result from the rusting of the steel.

In a previous study (21), it was found that one could electrically measure a change from a noncorroding passive half-cell potential (no salt at the steel-concrete interface) to an active half-cell potential, which occurs when there are sufficient chlorides present to cause corrosion of the steel. With proper instrumentation, it is less costly to measure the half-cell potential of the steel than to make periodic visual inspections for corrosion-caused concrete cracking; therefore, electrical measurements were used in this study as the sole criterion for the determination of the time to corrosion.

This study is a report on observations of the effects of curing method, curing time, cement factor, admixtures, and corrosion inhibitors and the test method used to determine the influence of these variables on the time to corrosion when the concrete was partially immersed in a saturated sodium chloride solution.

FABRICATION AND TESTING OF BLOCKS

The variables of concrete manufacture as used in this series of tests are given in Table 1. The river run aggregate was $^3/_4$ in. maximum, and the gradation complied with the 1964 Standard Specifications of the California Division of Highways. The cement used was type 2 modified, low-alkali, which also complied with the California specifications.

The reinforced concrete specimens were $4^1/_2$ in. wide, $2^1/_2$ in. thick, and 15 in. long. The No. 4 reinforcing steel was sandblasted and cast in the concrete to provide a minimum of $^7/_8$-in. concrete cover at any point. Figure 1 shows a typical test specimen.

Ten corrosion test specimens, four 6- × 6-in. cylinders and six 3- × 3- × $11^1/_4$-in. shrinkage bars, were cast from a single batch. Except for the test variations in batching, all concrete was batched in 1 day for each test variable. The 10 corrosion specimens were then either water-cured or steam- plus water-cured. Water curing was the complete submersion of the block in water at a temperature of approximately 72 F.

Specimens to be steam-cured were subjected to approximately 16 hours of steam curing at 138 ± 5 F and then water-cured for the length of time that related to that test variable of curing time. All steam-cured specimens were allowed to stand in their casting mold for a minimum of 4 hours. In all cases, the specimens were steam-cured on the day they were cast.

In all cases, the concrete slump was maintained at approximately $3^1/_4 \pm {}^1/_4$ in. by adjusting the amount of mixing water. The vibration of the concrete was accomplished by placing the steel molds on a Packer type of vibrating table.

After casting, the blocks were subjected to curing variables. In all cases, after the appropriate time of submerged water curing, the blocks were immediately transferred to the partial-immersion tanks. Figure 2 shows the blocks in place in the tanks. The blocks were partially immersed at an empirically selected depth of $3^1/_2$ in. on the 15-in. dimension in a saturated sodium chloride solution, the steel reinforcement rod being in a vertical position. A data acquisition system automatically obtained the half-cell potentials of the steel and also printed the results on tape.

The absorption tests were made in accordance with Test Method Calif. 538-A. Essentially, this test method first requires oven drying at 230 ± 9 F for a minimum of 3 days and a continuation of drying until the concrete water loss is less than 0.05 percent by weight. Prior to soaking, the concrete is stored overnight at room temperature. For the 6- × 6-in. absorption cylinders used in this test program, the approximate drying time was 14 days. In soaking, the concrete is required to be submerged at least 1 in. below the surface of the water. The weight gain is recorded at approximately 1, 3, 5, 7, and 24 hours and 2, 3, 8, 16, and 28 days. The weight gain versus time data are then plotted on log-log paper, and the best fit of the weight gain curve at 28 days is either visually obtained or calculated by a regression analysis. The weight gain is then computed and reported as percentage of concrete by volume.

RESULTS

Absorption and Cure Method

The absorption values given in Table 2 were obtained by means of Test Method Calif. 538-A dated April 6, 1970. The values were obtained after 28 days of soaking of the initially oven-dried 6- × 6-in. cylinders. (The specimens contained 6 sacks of cement per cubic yard.) The results for three repetitive tests are also given in the table. Table 3 gives the statistical significance of the data in Table 2. In Table 4, the absorption values given are the average for three cycles of concrete absorption. The statistical significance of these data are given in Table 5.

Figure 3 shows the average values for three cycles of absorption testing for each batch of concrete of the various cement factors. All concrete had 28 days of water curing prior to absorption testing. When the data shown in Figure 3 were analyzed by the method of least squares, the following relationship was obtained:

$$A_s = 1.01\ A_w - 0.422 \qquad (1)$$

where

A_s = absorption of steam-cured and 28-day water-cured concrete, in percentage of concrete by volume, and
A_w = absorption of 28-day water-cured concrete, in percentage of concrete volume.

For this relationship of 20 pairs of values, the correlation coefficient was 0.9175 and the standard error of estimate was found to be 0.33 percent.

As indicated by the results for plain non-air-entrained concrete containing five through nine sacks of cement per cubic yard, the process of steam plus water curing reduces the 28-day concrete absorption value roughly by 0.3 percent by volume. However, as given in Table 3, in the case of pozzolan, steam curing did not result in a significant reduction in absorption.

Concrete Sorption

As a preliminary confirmation of the means whereby water is absorbed into concrete, 12 concrete blocks of 3 × 3 × 11¼ in. were cast of ¾-in. maximum size aggregate. Originally, the blocks were moist-cured for 7 days and measured for drying shrinkage. These blocks had been on hand for a few years and, for the purpose of confirmation, were reused in this test. However, all were subjected to Test Method Calif. 538-A. After oven drying at 230 F, six of the room-temperature, oven-dried blocks were placed in a closed container so that the blocks were partially but horizontally immersed in water for ¼ in. of length. The other six blocks were also placed in the same container but were elevated above the water's surface so that they were only exposed to approximately 100 percent relative humidity. The results of this test are shown in Figure 4. As indicated by this figure, gain in weight by the initially oven-dried concrete is contingent neither on complete immersion nor on being in direct contact with water.

These results seem to confirm previous work (2) indicating that the major control on the movement of water into saturated concrete that results in a buildup in the concentration of chloride is the continued evaporation of the water from some part of the concrete surface. Also, this reaffirms work (2, 22) that demonstrated that the major mechanism controlling the passage of water into good-quality concrete is capillarity and evaporation and not hydrostatic pressure as related to permeability measurements.

Half-Cell Potentials

The half-cell potential of the steel was referenced to a saturated calomel electrode and was normally measured thrice weekly with a data acquisition system of 10 megohms of input impedance.

The normal trend of the half-cell potential behavior in this test is shown in Figure 5. As determined in a previous study (21), the half-cell potential of the steel abruptly

Table 1. Concrete mix variables.

Admixture	Cement Factor	Slump (in.)	Air (percent)	Unit Weight (lb/cu ft)	Mixing Water (lb/cu yd) Gross	Net
None	6.00	$3^1/_4$	1.9	153.1	329	286
Pozzolan, 15 lb/sack	5.99	$3^1/_4$	1.7	152.1	339	298
Pozzolan, 30 lb/sack	5.99	$3^1/_4$	1.7	150.4	363	325
2 percent hydrated lime, 1.89 lb/sack	6.00	$3^1/_8$	1.8	153.3	324	282
4 percent hydrated lime, 3.76 lb/sack	5.98	3	1.8	153.1	326	283
1 percent sodium benzoate, 0.94 lb/sack	5.96	$3^1/_4$	4.9	149.4	301	269
2 percent sodium benzoate, 1.88 lb/sack	6.04	$3^1/_2$	4.9	149.1	302	260
None	5.02	3	2.3	151.5	333	289
None	7.51	$3^1/_4$	1.7	154.4	322	282
None	9.01	3	1.6	154.7	334	297

Figure 1. Corrosion test specimen.

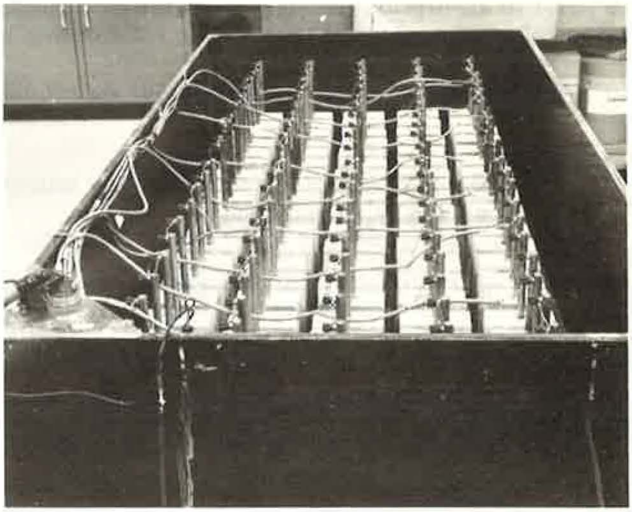


Figure 2. Test specimens placed in partial-immersion tanks.

Table 2. Percentage of concrete absorption.

Batch	Cycle	5 Sacks/Cu Yd WC[a]	SWC[a]	6 Sacks/Cu Yd WC	SWC	$7^1/_2$ Sacks/Cu Yd WC	SWC	9 Sacks/Cu Yd WC	SWC
1	1	14.63	14.20	13.66	13.53	12.79	12.41	13.13	12.87
	2	14.74	12.34	13.79	13.66	12.80	12.57	12.95	12.92
	3	14.82	14.36	13.68	13.70	12.79	12.56	13.01	12.94
2	1	14.69	14.71	13.95	13.24	12.94	12.29	12.84	12.51
	2	14.84	14.86	14.12	13.48	13.09	12.52	12.84	12.68
	3	14.89	14.99	14.09	13.42	13.13	12.62	12.85	12.71
3	1	14.19	14.41	13.58	13.49	13.17	12.70	13.10	12.65
	2	14.31	14.53	13.68	13.63	13.11	12.81	13.08	12.73
	3	14.37	14.70	13.60	13.67	13.22	12.92	13.03	12.85
4	1	14.87	14.97	14.18	13.55	13.13	12.87	13.15	12.64
	2	14.77	14.81	14.15	13.55	12.98	12.84	12.95	12.66
	3	14.94	15.01	14.22	13.57	13.02	12.94	12.96	13.06
5	1	14.81	14.50	13.89	13.63	13.48	12.48	13.01	12.61
	2	14.81	14.67	13.93	13.76	13.38	12.68	12.89	12.80
	3	14.81	14.75	13.89	13.72	13.35	12.72	12.89	12.83

[a]WC = water-cured concrete; SWC = steam- plus water-cured concrete.

Table 3. Statistical significance of data given in Table 2.

Sacks/Cu Yd	Mean WC	SWC	Standard Deviation WC	SWC	Coefficient of Variation WC	SWC
5	14.700	14.654	0.228	0.255	1.55	1.74
6	13.894	13.573	0.222	0.133	1.60	0.98
$7^1/_2$	13.092	12.662	0.213	0.193	1.63	1.52
9	12.979	12.764	0.103	0.148	0.79	1.16

Table 4. Percentage of concrete absorption of admixtures and inhibitors.

	Pozzolan				Sodium Benzoate				Hydrated Lime			
	15 Lb/Sack		30 Lb/Sack		1 Percent/Sack		2 Percent/Sack		2 Percent/Sack		4 Percent/Sack	
Batch	WC	SWC	WC	SWC	WC	SWC	WC	SWC	WC	SWC	WC	SWC
1	14.97	14.94	16.86	16.97	12.66	12.56	13.05	13.23	13.77	13.33	13.97	13.77
2	15.40	15.14	17.25	17.15	13.09	12.37	13.36	12.61	14.07	13.88	14.13	13.69
3	14.99	15.11	17.11	17.60	12.59	12.18	12.74	12.47	13.73	13.64	13.90	13.51
4	14.94	15.03	17.14	17.10	13.39	13.06	12.39	12.01	13.68	13.75	14.23	13.87
5	15.11	14.95	17.08	17.13	12.46	11.81	13.47	12.87	13.75	13.79	13.95	14.20

Table 5. Statistical significance of data given in Table 4.

Admixture-Inhibitor	Mean		Standard Deviation		Coefficient of Variation	
	WC	SWC	WC	SWC	WC	SWC
Pozzolan						
15 lb/sack	15.08	15.03	0.189	0.091	1.25	0.60
30 lb/sack	17.09	17.19	0.143	0.240	0.84	1.39
Sodium benzoate						
1 percent/sack	12.84	12.40	0.389	0.463	3.03	3.74
2 percent/sack	13.00	12.64	0.445	0.455	3.42	3.60
Hydrated lime						
2 percent/sack	13.79	13.68	0.166	0.213	1.20	1.56
4 percent/sack	14.04	13.81	0.138	0.256	0.99	1.85

Figure 3. 28-day absorption of steam- and water-cured concrete.

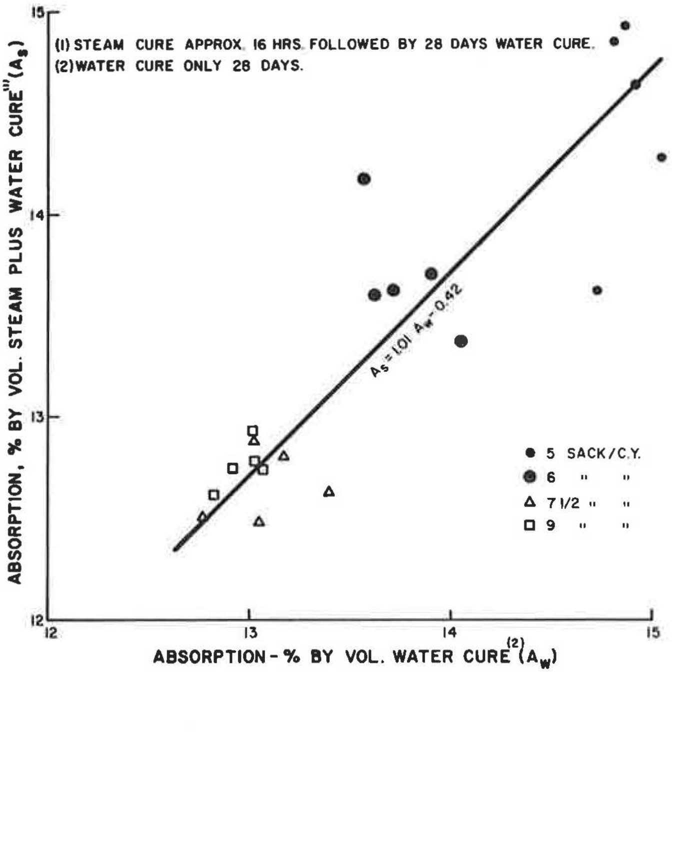


Figure 4. Weight gain of partially immersed specimens.

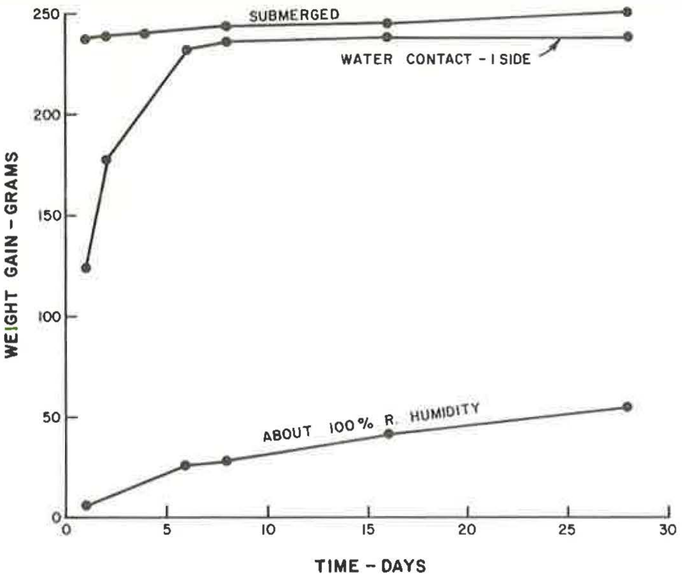
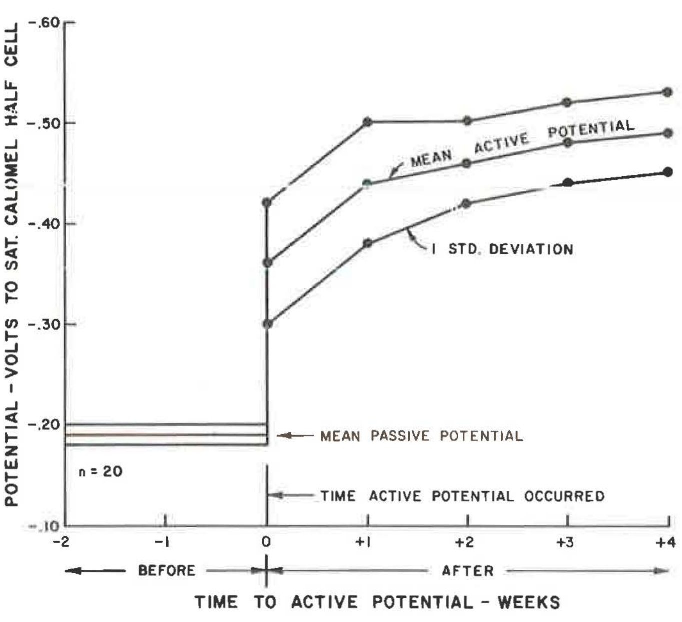


Figure 5. Typical half-cell potential of steel in concrete.



changes from passive to active when sufficient chlorides come in contact with the steel to cause corrosion. This change in potential has been measured to have a rate of change of about 0.15 volt in 3 hours; however, this rate is not known to be an average rate of change.

As indicated in Figure 5, the passive or noncorroding potential of the steel has a mean value of about -0.19 volt as referenced to the saturated calomel half-cell. In a previous study (21), the mean passive potential of 200 specimens was found to be approximately -0.11 volt, so these fall into a normal statistical distribution about the mean.

As shown in Figure 5, the mean change in potential at the time when it was observed was from -0.19 to -0.36 volt to the calomel cell. With increasing time, the mean active potential continues to increase to an average value of about -0.49 volt.

The observed value of an active potential of -0.36 volt seems to compare fairly well with the previously observed value of -0.33 volt (21). However, in this study as well as in another, it was observed that the steel in about 5 percent of the test blocks would have an abrupt change from a passive potential of about -0.19 volt to an active potential of about -0.36 volt and then drop back down to a passive potential. This phenomenon of shifting from passive to active for a small percentage of blocks is cyclic and has been observed in this and a previous study (21).

In a previous study (21), we showed that the time to the active potential of steel in concrete that is partially immersed in a saturated sodium chloride solution is mathematically related to the time to concrete cracking due to corrosion. Therefore, we did not include the test parameter of observing the surface of the concrete for cracks or rust stains. Visual observations not only are of questionable accuracy depending on the observer but also are a more time-consuming and expensive procedure than is the measuring of half-cell potentials.

Batch Variations

An attempt was made to determine the influence, if any, of batching procedures on the corrosion test results. In this test series, as given in Tables 6 and 7, the six-sack concrete was batched as follows:

1. Series 1 concrete was batched at a rate of one batch per week, and
2. Series 2 concrete was all batched in 1 day.

Each batch contained 10 blocks, of which one-half were water-cured for 28 days and the other steam- plus water-cured.

By an analysis of variance (25), it was determined that, for the steam-cured concrete, there was an F-ratio (for N = 50) of 0.0671, which does not indicate there was any significance between the test results for the different batching procedures.

For the moist-cured concrete, the analysis of variance was calculated to have an F-ratio of 10.01 (for N = 50), which is considered to represent a significant difference at the 95 percent confidence level as a result of the methods of batching.

In effect, the data show that batching methods both do and do not affect the results.

Effect of Cement Factor

To determine the influence of cement content on the time to corrosion of the steel, we cast concrete blocks containing 5, 6, 7½, and 9 sacks of cement per cubic yard. The test specimens were either water-cured or steam- plus water-cured for 28 days. Table 6 gives the days to an active half-cell potential for the cement variables (Fig. 6). The statistical significance of the results is given in Table 7. As shown in Figure 6, time to an active half-cell potential depends on the cement content of the concrete.

By the method of least squares, a regression analysis indicated the following relationship between cement factor and time to an active potential for the steam- plus water-cured concrete:

$$P_s = 0.157(C)^{3.34} \tag{2}$$

Table 6. Days to active half-cell potential with various cement contents.

Batch	5 Sacks/Cu Yd WC[b]	5 Sacks/Cu Yd SWC[b]	6 Sacks/Cu Yd[a] Series 1 WC	6 Sacks/Cu Yd[a] Series 1 SWC	6 Sacks/Cu Yd[a] Series 2 WC	6 Sacks/Cu Yd[a] Series 2 SWC	7½ Sacks/Cu Yd WC	7½ Sacks/Cu Yd SWC	9 Sacks/Cu Yd WC	9 Sacks/Cu Yd SWC
1	49	42	104	56	107	82	287	158	394	151
	71	41	78	71	118	77	287	326	406	393
	39	29	32	71	98	61	369	139	362	270
	34	48	18	64	100	91	317	174	518	286
	56	28	67	61	77	103	53	146	394	403
2	38	44	55	73	63	68	132	174	416	237
	64	20	70	62	135	49	251	129	367	181
	62	34	80	45	152	55	276	70	388	185
	45	31	111	50	128	61	223	120	351	185
	35	24	146	64	128	44	199	118	388	185
3	56	29	62	37	93	55	201	145	553	390
	34	33	79	44	89	61	177	166	322	210
	41	33	62	72	40	61	177	124	488	56
	40	40	56	58	70	68	177	135	504	194
	56	26	79	72	55	48	208	112	399	320
4	54	25	54	104	114	57	316	207	316	270
	54	54	48	64	98	70	316	124	396	253
	26	43	48	71	131	55	176	162	512	165
	57	33	35	39	121	55	109	188	405	272
	54	32	61	71	61	63	232	125	316	272
5	29		118		110	77	213		307	
	35		85		75	77	204		385	
	35		84		30	44	206		276	
	27		77		112	77	171		449	
	31		97		91	77	66		68	

Note: Batch 6, cast in 1 day, was done at 6 sacks/cu yd. For water-cured concrete, the results were 137, 110, 96, 77, and 103 days to active potential; and, for steam- plus water-cured concrete, the results were 34, 72, 48, 51 and 65 days to active potential.

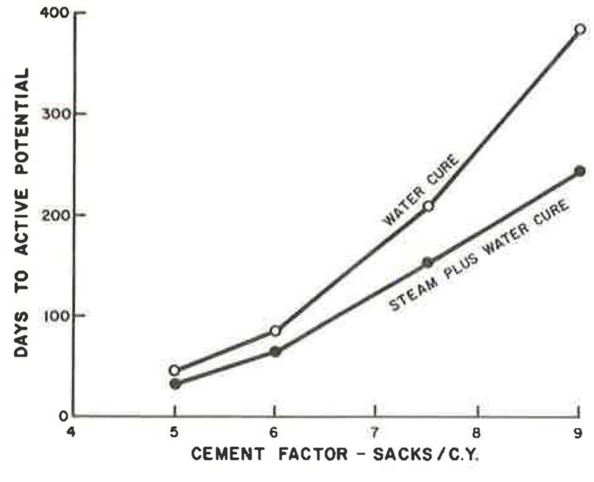
[a]Series 1 samples were cast at one batch per week, and series 2 were all cast in 1 day.

[b]WC = water-cured concrete; SWC = steam- plus water-cured concrete.

Table 7. Statistical significance of data given in Table 6.

Variable	5 Sacks/Cu Yd WC	5 Sacks/Cu Yd SWC	6 Sacks/Cu Yd Series 1 WC	6 Sacks/Cu Yd Series 1 SWC	6 Sacks/Cu Yd Series 2 WC	6 Sacks/Cu Yd Series 2 SWC	7½ Sacks/Cu Yd WC	7½ Sacks/Cu Yd SWC	9 Sacks/Cu Yd WC	9 Sacks/Cu Yd SWC
Mean	44.9	62.5	72.2	62.5	97.3	63.5	213.7	152.1	387.2	243.9
Standard deviation	12.7	15.2	28.5	15.2	29.5	15.0	77.8	51.2	97.3	88.2
Coefficient of variation	28.4	24.3	39.5	24.3	30.3	23.7	36.4	33.6	25.1	36.1

Figure 6. Effect of cement factor on time to active potential.



where

P_s = days to an active potential, steam plus 28 days of water curing of concrete, and
C = cement factor, in sacks per cubic yard.

For this relationship, the correlation coefficient was 0.910 and the standard error of estimate was 0.1306 $\log_{10}$ for the 110 pairs of data.

When we analyzed the data for the 28-day water-cured concrete, a regression analysis indicated the following relationship:

$$P_w = 0.104(C)^{3.72} \tag{3}$$

where P_w = days to an active potential, water-cured concrete, and C is as described above. For this relationship, the correlation coefficient was 0.8797 and the standard error of estimate was 0.1765 $\log_{10}$ for the 130 pairs of data.

To determine the average fit between the water- and steam- plus water-cured concrete, we combined the data by mathematically equating the water- to the steam-cured concrete (Eq. 8). The result of this analysis was

$$P_{ws} = 0.125(C)^{3.42} \tag{4}$$

where P_{ws} = days to active potential of mathematically combined water- and steam-plus water-cured concrete. For this relationship, the correlation coefficient was 0.8915, and the standard error of the estimate was 0.1504 $\log_{10}$ for the 240 pairs of data.

Concrete Curing Time

To study the effect of the length of curing on the time to an active potential of the embedded steel, we water-cured concrete for 2, 4, 8, 16, and 32 days. As given in Tables 8 and 9, and also Figure 7, the length of water curing has a definite influence on the time to corrosion. The longer the curing time is, the longer the time to an active potential is.

By the method of least squares, a regression analysis was made to determine the effect of water curing time on the time to an active potential. For the steam- plus water-cured concrete, results of the analysis were

$$P_s = 6.23(D)^{0.66} \tag{5}$$

where

P_s = days to an active potential, steam- plus water-cured concrete, and
D = days of post-underwater curing.

For this relationship, the correlation coefficient was 0.8165, and the standard error of estimate was 0.2014 $\log_{10}$ for the 100 pairs of data.

The regression analysis of the effect of water curing time of water-cured concrete on the time to an active potential of the steel produced the following equation:

$$P_w = 6.0(D)^{0.90} \tag{6}$$

where P_w = days to active potential, water-cured concrete. For this relationship, the coefficient was 0.826, and the standard error of estimate was 0.2651 $\log_{10}$ for the 100 pairs of data.

To combine the effect of the two curing methods, we corrected the time to an active potential of the water-cured concrete to that of the steam- plus water-cured concrete by use of Eq. 8. The results of this analysis indicate the following influence of water time on the time to an active potential:

$$P_{ws} = 6.1(D)^{0.78} \tag{7}$$

Table 8. Days to active half-cell potential at various underwater curing times.

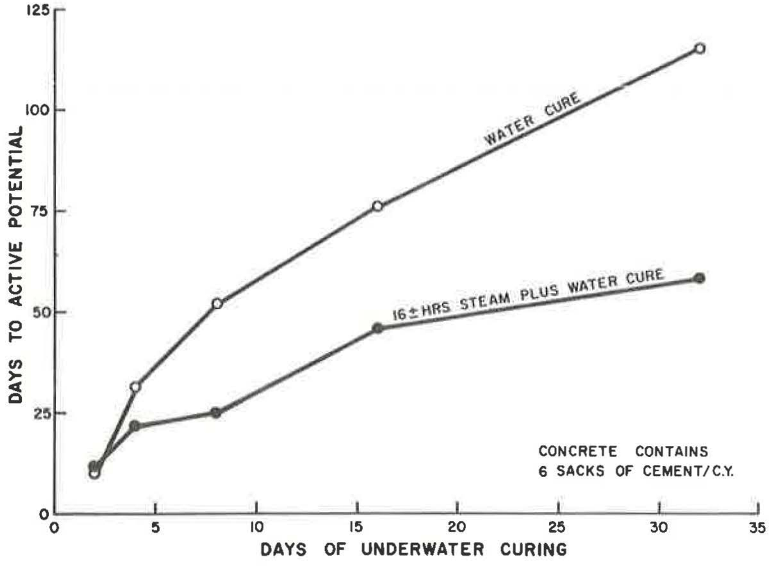
Batch	Days of Underwater Curing of WC[a]					Days of Underwater Curing of SWC[a]				
	2	4	8	16	32	2	4	8	16	32
1	9	21	29	58	115	23	56	23	55	94
	5	18	47	63	60	9	6	8	34	49
	23	41	83	47	129	6	10	28	22	53
	12	19	68	84	117	23	19	37	30	49
	8	24	50	72	103	7	4	12	29	56
2	5	3	26	47	101	13	10	29	61	53
	21	38	76	61	129	19	27	19	54	62
	12	31	48	63	138	12	21	33	28	72
	6	21	65	84	183	8	38	37	26	80
	26	24	65	69	80	8	14	33	23	89
3	5	59	42	86	117	5	18	20	40	70
	2	63	44	112	140	13	17	20	35	70
	22	74	48	55	154	9	11	28	33	63
	5	59	49	75	143	5	12	16	47	61
	55	59	48	91	109	5	35	28	34	59
4	19	40	38	44	122	19	10	21	51	52
	2	63	44	86	119	26	7	34	36	75
	5	38	55	91	61	13	7	16	55	59
	2	38	52	149	75	5	31	34	68	101
	5	27	52	84	109	13	19	29	55	52

Note: All specimens composed of six sacks of cement per cubic yard.
[a]WC = water-cured concrete; SWC = steam- plus water-cured concrete.

Table 9. Statistical significance of data given in Table 8.

Days of Underwater Curing	Mean		Standard Deviation		Coefficient of Variation	
	WC	SWC	WC	SWC	WC	SWC
2	10.0	12	7.8	6.7	78.8	55.4
4	38	19	19.2	13.1	50.5	70.5
8	51.5	25	14.2	8.5	27.7	33.5
16	76.1	41	24.7	13.7	32.4	15.3
32	115.2	66	30.5	15.3	26.5	23.2

Figure 7. Effect of curing time on time to active potential.



where P_{ws} = days to active potential, water-cured and steam- plus water-cured concrete. For this relationship, the correlation coefficient was 0.7894, and the standard error of estimate was 0.2603 $\log_{10}$ for the 200 pairs of data.

Curing Method and Time to Active Potentials

The average time in days to an active potential for the two curing methods is shown in Figure 8. By the method of least squares, a regression analysis indicated the following relationship between water and steam plus water curing to the time to an active potential of the steel:

$$P_s = 0.595\ P_w + 4.21 \qquad (8)$$

where

P_s = days to active potential, steam- plus water-cured concrete, and
P_w = days to active potential, water-cured concrete.

For this relationship, the correlation coefficient was 0.9680, and the standard error of estimate was 18.1 days for the 40 pairs of averaged data points.

Admixture-Inhibitor Study

In a preliminary evaluation of using admixtures in concrete to forestall the time to corrosion of steel in concrete, three were selected. Hydrated lime was added to the concrete mix at the rate of 2 and 4 percent by weight of the cement. Sodium benzoate, a reported corrosion inhibitor for steel in concrete (23), was added to the concrete mix at a rate of 1 and 2 percent by weight of the cement, and pozzolan, a material that is reported to enhance some properties of concrete (24), was added at a rate of 15 and 30 pounds per sack of cement.

The pozzolan was a calcined volcanic tuff tested under ASTM C 618. All concrete used in the admixture-inhibitor study contained 6 sacks of cement per cubic yard and had a total underwater curing time of 28 days before corrosion testing. The results are given in Table 10, and are shown in Figure 9. The statistical significance of the data are given in Table 11.

Concrete Shrinkage

At the time the concrete corrosion specimens were batched, three 3- × 3- × $11\frac{1}{4}$-in. bars were cast to measure the 14-day drying shrinkage at 50 percent relative humidity and 73 F. As given in Table 12, without altering the ratio of coarse and fine aggregate, we found that shrinkage of the concrete increased with increasing cement factor apparently because of the increase in mixing water. Also, when pozzolan was added to the mix, there was an increase in drying shrinkage associated with an increase in mixing water needed to maintain constant slump. When hydrated lime was added to the concrete mix, the mixing water and shrinkage were not significantly changed.

The addition of the corrosion inhibitor (23) sodium benzoate did not result in a highly significant change in shrinkage. However, it is of interest that, even with reduction in mixing water, the concrete containing sodium benzoate did not show any significant reduction in its drying shrinkage over the concrete without admixture. During the mixing of the concrete containing sodium benzoate, the mix was observed to have a frothy or bubbly appearance.

Chloride in Concrete

As an indication of the quantity of salt that could be absorbed by the concrete at the steel-concrete interface, 40 six-sack, 2-day water-cured blocks were analyzed. Half of these blocks were water-cured for 2 days, and the other half were first steam-cured for about 16 hours and then water-cured for 2 days.

Fourteen days after the reinforcing steel had a measured active half-cell potential, the blocks were mechanically split to expose the concrete mortar interface with the

Figure 8. Water curing versus steam plus water curing.

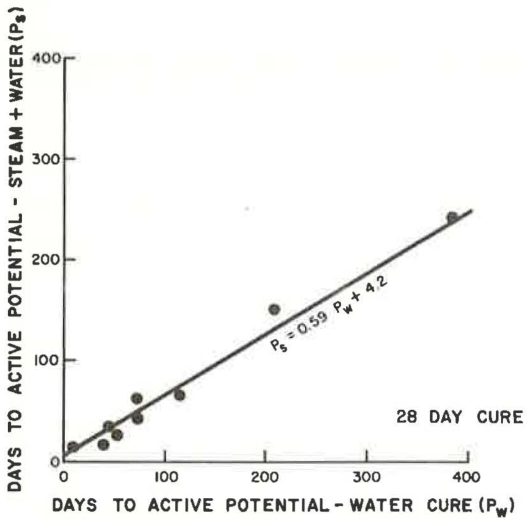


Table 10. Days to potential half-cell with various admixtures and inhibitors.

	15 Lb Pozzolan		30 Lb Pozzolan		2 Percent Lime		4 Percent Lime		1 Percent Sodium Benzoate		2 Percent Sodium Benzoate	
Batch	WC[a]	SWC[a]	WC	SWC	WC	SWC	WC	SWC	WC	SWC	WC	SWC
1	387	281	67	566	90	49	120	54	193	95	174	88
	130	211	137	515	106	48	137	47	169	88	197	60
	485	172	139	517	82	50	69	71	151	75	172	75
	134	176	64	268	82	60	116	64	69	88	75	104
	61	274	67	71	67	69	75	54	71	137	145	67
2	55	195	62	265	143	55	87	55	141	101	185	70
	62	181	62	36	66	66	106	37	146	62	162	73
	66	164	561	220	77	64	70	55	188	101	181	35
	146	159	62	468	99	59	104	48	136	55	118	70
	174	97	62	48	77	50	62	64	176	64	92	66
3	54	177	62	516	89	51	119	84	147	89	168	96
	63	138	54	376	93	62	107	160	151	98	177	124
	58	170	53	247	56	75	62	58	177	126	257	107
	62	56	441	448	110	33	100	62	128	51	226	98
	62	247	54	322	98	58	86	62	161	54	243	151
4	46	26	477	316	148	55	85	39	109	179	237	190
	50	165	477	431	120	85	95	61	307	181	179	139
	46	190	48	272	129	64	29	57	151	151	218	176
	46	90	54	517	61	61	109	50	158	151	270	124
	319	218	57	57	134	43	124	61	57	83	61	134
5	475		69		69		106		120		188	
	349		43		104		90		125		106	
	225		69		84		111		141		120	
	80		69		86		78		132		183	
	69		77		106		69		55		84	

Note: Specimens composed of six sacks of cement per cubic yard.

[a]WC = water-cured concrete; SWC = steam- plus water-cured concrete.

Table 11. Statistical significance of data given in Table 10.

	Mean		Standard Deviation		Coefficient of Variation	
Admixture-Inhibitor	WC	SWC	WC	SWC	WC	SWC
15 lb pozzolan	148	169	141	65.3	95	38.5
30 lb pozzolan	135	324	160	174	118	53.8
2 percent lime	95	58	25.2	11.5	26.5	19.9
4 percent lime	92.6	62	24.6	25.3	26.6	40.7
1 percent sodium benzoate	142	101	51.4	40.2	36.1	40.8
2 percent sodium benzoate	169	102	57.3	40.8	34.0	39.9

Figure 9. Effect of admixtures-inhibitors on time to active half-cell potential.

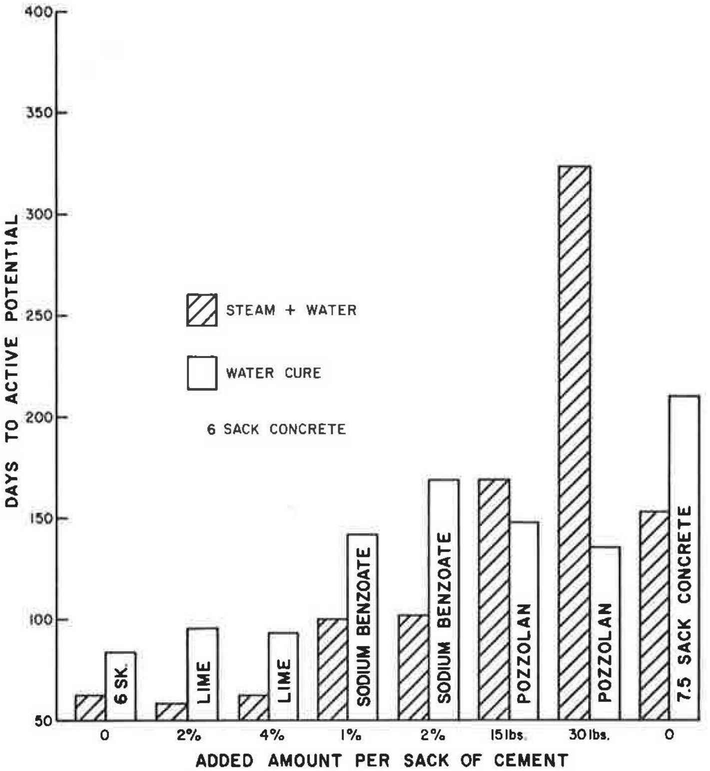


Table 12. 14-day drying shrinkage of concrete.

Cement Factor	Admixture	Percentage of Shrinkage[a]	Percentage of Control[b]
6.00	None	0.023	100
5.99	Pozzolan, 15 lb/sack	0.033	143
5.99	Pozzolan, 30 lb/sack	0.047	205
6.00	2 percent hydrated lime, 1.89 lb/sack	0.025	109
5.98	4 percent hydrated lime, 3.76 lb/sack	0.025	109
5.96	1 percent sodium benzoate, 0.94 lb/sack	0.027	117
6.04	2 percent sodium benzoate, 1.88 lb/sack	0.021	92
5.02	None	0.021	92
7.51	None	0.025	109
9.01	None	0.030	130

[a]3- x 3- x 11¼-in. bars [b]Average of three bars.

steel. Then the mortar, for a depth of about $\frac{1}{16}$ in., was mechanically removed for about 3 in. This section of mortar would have been in that part of the concrete block that was immersed in the sodium chloride solution.

The concrete mortar was then chemically analyzed for chloride content, and it was estimated that the sand-cement mortar portion of concrete would weigh 2,040 lb/cu yd.

For the 20 water-cured blocks, the absorbed chloride in the concrete at the mortar-steel interface had a calculated mean value of 17.9 lb/cu yd, with a standard deviation of 2.38 lb/cu yd for the 20 data points. In the case of the steam plus 2 days of water curing, the mean chloride content at the mortar-steel interface was 21.7 lb/cu yd, and the standard deviation was 3.82 lb/cu yd for the 20 data points.

The computed chloride content of the concrete in this test is less than that found in bridge piles that were continuously submerged for about 40 years in seawater (28).

DISCUSSION OF RESULTS

Corrosion Testing

Although 15 different test methods have been identified, it might be well to discuss the differences that have been obtained in three separate tests with approximately the same six-sack concrete mix and test method by the same laboratory (20, 21).

In study 1 (20), the six-sack, non-air-entrained concrete blocks were fog-cured for 14 days and then air-dried by laboratory storage for 60 days, prior to being partially immersed in a saturated salt solution. The average time to corrosion-caused cracking of these concrete blocks was 22 days.

In study 2 (21), the six-sack concrete blocks were moist-cured in a fog room for 7 days and stored in the laboratory under controlled conditions of about 72 F and 50 percent relative humidity for 10.5 months. These blocks were then subjected to varying depths of partial immersion in fresh water for about $5\frac{1}{2}$ months before being partially immersed in the saturated salt solution.

In study 1 concrete cracking was observed after 22 days, whereas in study 2 the time was 309 days, a time ratio of 14 to 1 for about the same quality of concrete.

In comparing results from the two previous tests, it is apparent that the moisture content of the concrete immediately prior to partial immersion of the blocks in the salt solution was of great significance. In test 1, the concrete had air-dried for 60 days, and, in test 2, the blocks were saturated with fresh water immediately prior to partial immersion. It is obvious that the partially air-dried blocks could almost immediately absorb the salt solution, whereas the blocks saturated in fresh water in test 2 had to transpire some of the absorbed water before the salt water could enter the concrete. Also, as shown by this study, the influence of concrete curing time prior to testing was also probably significant.

In this latest, or third test series, even though the concrete was saturated with fresh water immediately prior to saltwater exposure, the days to an active potential of the steel of the six-sack, water-cured concrete varied from an average of 10 to 115 days, depending on the length of water curing.

In addition to the variables of curing time and initial moisture content of the concrete, in study 1 it was observed that, when the atmosphere was about 54 percent relative humidity, the length of time to corrosion-caused cracking of a seven-sack concrete increased by approximately 50 percent when the relative humidity of the atmosphere was increased to 95 to 97 percent. Therefore, it is apparent that corrosion testing of steel in concrete can revolve about two basic concrete parameters: absorption by a dry concrete and combination of desorption by evaporation and absorption by capillary action of the aggressive water. Also, when considering desorption as a test parameter, it is obvious that the relative humidity of the atmosphere should be of significance.

Variation in Results

In study 2, and even in this third study, the coefficient of variation of the time to corrosion is roughly 30 to 35 percent. A coefficient of variation of this magnitude has a great bearing on the number of samples that must be tested to determine whether the test results are real or accidental. For example, at the 95 percent confidence level and a coefficient of variation of 35 percent, it could be expected that the average test value for two blocks could have a maximum mean error of about 55 percent. Therefore, if two samples were used for testing a variable, a difference of 50 percent in the average test time would look good on a chart, but it could just as well be an accident.

Chloride Accumulation

It is acknowledged that, when the chloride ion is either mixed with (1, 11, 18) or absorbed by the concrete (2, 16, 17, 20, 27), this ion is primarily responsible for the overwhelming majority of the reported cases of corrosion of steel. Although this study has directed a maximum amount of attention toward the influence of concrete variables on the time to corrosion of steel, only a minor amount of attention was directed toward the actual mechanism of chloride accumulation by absorption or desorption of water in the concrete.

For example, this study shows that concrete absorption is not the single controlling mechanism for the time to corrosion. However, absorption seems to relate to concrete cement factor in this test, which does relate to the time to corrosion. In a previous study (2), it was empirically determined that the quantity of water that evaporated out of concrete could be related not only to concrete composition but also to the time to corrosion of the steel. It was assumed that chlorides that accumulated in the concrete by the process of water evaporation would leave a residual of salts.

In this regard, it was reported (28) that the computed concentration of chloride in concrete piling that was continually submerged for approximately 40 years was greater than that found in the seawater. Therefore, it is inferred that chlorides are transferred into the concrete by capillary action and concentrated by a mechanism of evaporation of the water at the atmospherically exposed concrete surface. If this accumulation of chloride is by absorption and desorption by water vapor, then we could infer that water vapor can travel within a seemingly water-saturated pile for a longitudinal distance of at least 10 ft (28). Apparently there could be voids in concrete where there is free passage of water vapor even though the capillary pores are filled with water.

If the indicated mechanism of chloride accumulation in concrete can be confirmed, then laboratory corrosion testing can be directed toward controlling those variables that can cause misleading test results. And, most important, the solution to economically preventing or forestalling the time to corrosion of steel in concrete can be determined in the laboratory.

SUMMARY AND CONCLUSIONS

Absorption

Concrete absorption, per se, does not appear to be a direct measure of ability of concrete to inhibit or prevent corrosion of embedded steel. However, it may be valuable in evaluating similar concrete mixes. That concrete absorption is not necessarily related to the time to corrosion of the steel is illustrated by the following two conflicting test results: (a) concrete absorption is reduced by increasing the cement factor, which increases the time to corrosion, and (b) concrete absorption is reduced by steam curing with a resulting decrease in the time to corrosion. Roughly, steam curing reduces volumetric concrete absorption by 0.3 percent (Fig. 3).

Concrete Sorption

A preliminary test of submerging one face of a concrete specimen in water was performed to confirm capillary action as the primary mode for transport of water into concrete. The concrete that had only one face in water absorbed nearly as much water as one that was entirely submerged. Therefore, capillary action must be a large factor in the movement of water into the atmospherically exposed area of concrete. In addition, the gain in weight of concrete that is simply exposed to high humidity confirms the hygroscopic adsorption properties of concrete (26) (Fig. 4). In effect, because all concrete samples were considered water-saturated at the beginning of the test, the variations in test results due to concrete variables are probably most related to the desorption characteristics of the concrete.

Active and Passive Potential

The total time that it takes for the half-cell potential of the steel to change from passive to active has been related to the total test period that it takes concrete to crack as a result of steel corrosion (21). In this study, the mean passive or noncorroding half-cell potential of the steel was about -0.19 volt to a saturated calomel reference cell. The mean value for the active or corroding potential on the day that the active potential was detected was -0.36 volt, which continued to increase after 4 weeks of measuring to a mean of -0.49 volt. As indicated by this and a previous study (21), the half-cell potential of steel follows a normal statistical distribution; therefore, there will be no absolute value that will precisely define an active or passive potential. However, based on the two studies, it appears that the potentials with a numerical value of less than about -0.22 volt to the calomel reference cell could empirically indicate a passive potential for about 95 percent of the measurements. Also, if the potential had a numerical value of greater than about -0.27 volt, then it could indicate an empirical active potential of the steel in approximately 95 percent of the measurements. The active potential does not indicate a rate of corrosion, merely that sufficient chloride is present to destroy the corrosion inhibiting or passivating effect of the concrete (Fig. 5).

Cement Factor

In all of the testing, as the cement factor was increased, the time to corrosion of the steel increased. The results of a regression analysis roughly indicate that the addition to the concrete mix of one sack of cement per cubic yard will increase the time to an active potential of the steel by roughly 70 percent for both the water- and the steam- plus water-cured concrete within the cement factor limits of 5 to 8 sacks per cubic yard (Fig. 6).

Length of Cure

Within the limits of the test, it was found that the length of water curing of the concrete had a pronounced influence on the time to an active half-cell potential of the steel. The longer the curing was, the longer the time to corrosion or to an active potential was. However, only a six-sack concrete mix was tested for this variable. Therefore, the length of concrete curing time may have more or less influence on the time to an active potential depending on the cement factor and even on extended times of water curing beyond the test period of 32 days.

The regression analysis of the data indicated that, by doubling the water curing time, the time to an active potential increased by 50 percent for both the water- and the water-plus steam-cured concrete (Fig. 7).

Water and Steam Curing

The steam curing of the concrete reduced the time to corrosion of the steel. With other variables equal, the regression analysis of the data indicated that the time to an active potential of the steel by steam plus water curing was about 60 percent of that by water curing alone (Fig. 8).

Admixture-Inhibitors

In this study, hydrated lime and sodium benzoate were used as corrosion inhibitors, and pozzolan was used as a concrete admixture. Test results did not clearly indicate that the lime had a significant influence, even though there was a small increase in the time to corrosion or active half-cell potential of the steel.

The use of sodium benzoate resulted in a marked increase in the time to corrosion. However, there was a significant reduction in mixing water requirements due to the entrainment or entrapment of air. Therefore, the overall benefit of sodium benzoate is questionable until it is compared to a plain concrete made with the same amount of mixing water.

Although there was an increase in mixing water required for a given slump, the addition of a calcined, volcanic-tuff pozzolan resulted in a significant increase in the time to corrosion. This effect was greatest when the concrete was steam-cured. Based on the large coefficient of variation of test results for the water-cured concrete containing pozzolan, it is speculated that the apparent benefits of pozzolan may be increased with longer water curing than the 28 days used for this particular test; i. e., give the water-cured pozzolan and calcium hydroxide chemical reaction a chance to reach the same "maturity" of the steam-cured mix (Fig. 9).

Concrete Shrinkage

The major significance in the 14-day drying shrinkage tests was the large increase in shrinkage of the concrete containing pozzolan. Perhaps a reduction of the shrinkage might be achieved by using a coarser sand and by reproportioning the mix so as to reduce the mixing water requirements (Table 12).

Chlorides in Concrete

Chemical analysis of the mortar at the steel-concrete interface showed a calculated concentration of 17.9 and 21.7 lb of chloride in the mortar portion of a cubic yard of concrete for the water- and steam-cured concrete respectively. Based on an assumed concrete absorption of 14 percent by volume, the evaporable water in the concrete could have contained salt in solution at a calculated chloride concentration of about 9 percent by weight at the steel-concrete interface.

Batch Variations

An analysis of variance was calculated for the batches of the six-sack, 28-day water- and the steam- plus water-cured concrete that was batched all in 1 day and those that were batched on a weekly basis.

For the steam-cured concrete, the test results indicated that there was no significant difference between the two procedures for casting concrete blocks. However, there was a significant difference found for the moist-cured concrete. It is speculated that for the moist-cured concrete the difference might be related to the vagaries of the test itself inasmuch as the difference between the means was about 22 percent of the 92-day average test time to active potential. This 22 percent variation in the means should also be compared to the approximate 30 percent coefficient of variation of the normal testing without regard to batching effect.

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DETERIORATION OF 249 BRIDGE DECKS

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•IN THE PAST decade, highway administrators have recognized the severity of concrete bridge deck deterioration and have requested that serious efforts be made toward solution of the problem. During that same period, largely due to the Interstate Highway program, new concrete bridge decks were constructed at an unparalleled rate in the United States. Already, some of these Interstate highway decks have shown severe deterioration. This paper describes a study of 249 bridge decks built 4 years ago in Pennsylvania. The study was initiated in an attempt to learn the extent of deterioration on fairly new decks and to establish the relative importance of factors commonly associated with deterioration.

The two most important mechanisms of deterioration of concrete bridge decks have been postulated and are generally accepted. These include deterioration by freezing and thawing (6) and spalling caused by corrosion of reinforcing steel (9). Spalling is probably the single most serious form of deterioration from the standpoint of repair cost. It is necessarily preceded by the occurrence of a fracture plane or plane of separation that forms approximately parallel to the traffic surface usually at about the level of the top reinforcing steel. The various phenomena involved in spalling are shown in Figure 1. Note the importance of cracking.

Another type of deterioration, surface wear, is closely related to highway safety. John Volpe, Secretary, U.S. Department of Transportation, named highway safety as one of the six major problems in the transportation field today (10). From a skid resistance standpoint, surface mortar deterioration, particularly wear, is probably the most serious form of deterioration on bridge decks. Traffic wear, aggravated by studded tires (3, 4, 7) polishes the surface, which frequently reduces skid resistance to dangerous levels.

OBJECTIVES AND SCOPE

The objectives of the study described in the paper were twofold: (a) to discover the extent and nature of deterioration of a large sample of fairly new bridge decks, and (b) to evaluate the relative importance of factors causing deterioration on these decks.

Three major forms of deterioration were considered in this study: spalls and fracture planes; surface mortar deterioration (SMD), including wear by traffic and the general disintegration of weak mortar; and cracking, including transverse, longitudinal, and diagonal. We chose 249 four-year-old decks for observation. Inasmuch as the effects of age have been carefully documented in previous studies, it was excluded as a variable in this study by including only those bridges built in Pennsylvania in 1966. That particular year was chosen because of significant upgrading of Pennsylvania's specifications in the 5-year period prior to 1966. Information from blueprints, construction and maintenance records, and individual contractors was gathered concerning 32 factors (Table 1) that have been related to deterioration by various investigators.

SURVEY AND ANALYSIS PROCEDURE

Survey Method

Each of the 249 decks was examined by a 2- or 3-man team. Quantitative measurements of spalled and fractured areas, length of cracks (including only cracks greater

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Figure 1. Spalling phenomena (5).

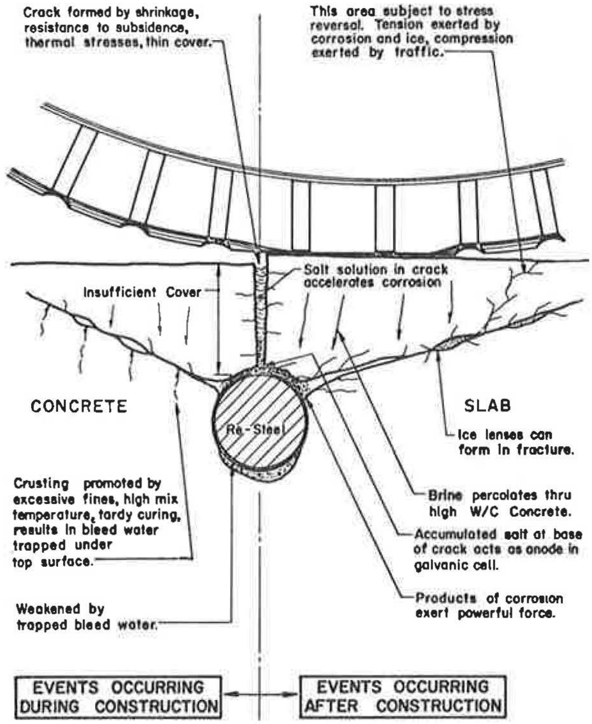


Table 1. Factors related to deterioration.

Factors	Source	Comments
Materials		
Average slump	Field construction records	
Average air content	Field construction records	
Average flexural strength	Actual field test values	
Aggregate source	Name of supplier	
Use of retarder		Yes or no
Design		
Superstructure type	Final approved designs	Eight types
Beam spacing		Usually 7 to 10 ft
Deck width		Curb to curb
Grade of deck	Slope of deck as related to drainage	
Skew	Angle of span with roadway	
Span length		Arbitrary minimum of 31 ft
Slab thickness		Usually 8 to $8\frac{1}{2}$ in.
Structural stiffness	Calculated with and without parapets for each span	
Form type		Usually stay-in-place or conventionally formed
Concrete cover over steel		Usually specified at 2 in. with an occasional case of $1\frac{1}{2}$ in.
Construction		
Contractor	By name	78 contractors
Mixing method		Central or transit
Placement method		Bucket, conveyor, buggy, or truck
Placement temperature	Field temperature data	
Month of construction		
Curing method		Usually wet burlap; sometimes heat and insulation
Finishing machine	By name	Seven types
Environment		
Average daily traffic	Statewide Statistical Logbook	
Maturity period		Month between end of construction and opening to traffic or de-icing, whichever occurred first
ASTM weathering index	(1)	
No. of freeze-thaw cycles/year	(1)	
Maintenance		
De-icer type		NaCl, $CaCl_2$, or combinations of the two
Rate of de-icer applications		Usually 200 to 800 lb/mile
No. of de-icer applications/year	Individual truck drivers made estimates for their routes	
Type of linseed oil applications/ year		Usually 0, 1, or 2
Month of linseed oil applications		Usually August, September, or October

than 1 ft long), and areas affected by mortar deterioration were made. Qualitative estimates (visual) of the type and extent of miscellaneous deterioration such as popouts, parapet damage, and others were also made. In cases where spalls occurred, the depth of steel reinforcing was measured with a ruler. Where fracture planes occurred but concrete had not yet become dislodged, a pachometer was used to measure the depth of steel. Steel depth was also measured in sound areas immediately adjacent to deteriorated zones. The deck surveys required about 10 months to complete and were performed between winter seasons.

Equipment Used

No special equipment was required for the deck surveys other than the aforementioned pachometer and a unique chain drag device used to detect fracture planes (Fig. 2). Although fractured areas are incipient spalls, they are usually not discernible at the road surface even upon close inspection. The chain drag device comprises a timber member with 2-ft lengths of ordinary snow-tire chains attached at 4-in. intervals along the member. As the device is traversed over a fracture plane on the deck, the chains produce a hollow sound. Tape recordings of several of these soundings were quite distinct. When a fracture plane was encountered, it was delineated, and its area and depth to reinforcing steel were measured (Fig. 3).

Data Analysis

All data, from three major sources (blueprints, questionnaires, and the field survey) as well as from other sources such as contractor interviews, were recorded in a single large logbook. Data from this logbook were punched and verified on computer cards. Five cards were necessary to include all the information for each span of each bridge deck, resulting in approximately 3,000 cards.

The analyses of data were performed in two fashions. First, the data in the logbook were perused for obvious relationships. When one was encountered, e.g., the relationship between certain sources of aggregate and the occurrence of concentrated popouts, it was evaluated by standard statistical techniques. Other relationships that had become obvious during the deck surveys were also evaluated in this manner.

Where relationships between deterioration and its causes were not immediately clear from the data, computer statistical analyses were employed. Because of the large number of possible causes of deck deterioration and the even larger number of possible combinations of interactions of causes, computer analyses were relied on quite heavily. Computer statistical techniques included the Pearson product moment correlation coefficients, upward multiple linear regression analysis, and the automatic interaction detection (AID) technique.

By far the most enlightening data evaluation was achieved with AID (8). Although the results of this study were verified by several statistical techniques, only the AID analysis is discussed in this paper. AID was designed specifically for research problems where the purpose of the analysis is more than the reporting of descriptive statistics but may not necessarily be the exact testing of specific hypotheses. The AID analysis splits variables into two categories based on variance in the dependent variable. It results in a tree diagram that can be easily interpreted for specific details concerning the important factors as well as the interactions of factors. An example of a tree diagram will be presented in the following discussions.

CONDITION OF THE DECKS

The overall condition of the 249 four-year-old decks was somewhat worse than anticipated. Fracture planes and spalls, the most serious form of deterioration from a repair cost standpoint, were found on 22 percent of the decks. Three decks, each havine very shallow reinforcing steel, were already in need of immediate repair; 11 decks had already been patched to various degrees. Fracture planes and spalls were found in many different positions on the decks. Some appeared to develop preferentially on the shoulders, at joints, and at the beginning or end span.

From the standpoint of skid resistance, wear is the most serious form of deterioration (Fig. 4). Ninety-five percent of these relatively young decks exhibited surface mortar deterioration (wear plus general mortar deterioration), with 97 percent of the affected area attributed to wear and only 3 percent attributed to disintegration of weak mortar (not polished and frequently not in the wheel zones).

In total, about 6.7 miles of cracks were encountered on the 21½ lane-miles of deck surface observed during the study. The distribution of the three types of cracks is shown in Figure 5. Note that, by number and length, transverse cracks occurred more frequently than did other types. In essentially every case, these cracks occurred directly over the transverse reinforcing bars.

Several types of miscellaneous deteriorations were observed to recur frequently. A total of 68.7 percent of the decks exhibited noticeable popouts, and 26.1 percent had one or more areas of severe popouts, indicating aggregate shortcomings. Most of these failures were linked to a few aggregate sources. Map cracking appeared in 20 percent of the decks especially near end spans (possibly related to Pennsylvania's wet-burlap deck-curing specification). There were frequent instances of finishing problems (failure to fully close the surface by screeding) and of debris incorporated in the surface (including mud balls, wires, and very frequently wood chips). Many curbs and parapets exhibited long striations, broken corners, and general disintegration, all of which were thought to have been initiated by snowplows. About 10 percent of the decks had clogged scuppers, and grass was observed growing on the shoulders of two decks.

RELATIVE IMPORTANCE OF CAUSAL FACTORS

Data collected for the 32 deterioration-related factors (henceforth called causal factors), after being analyzed for obvious relationships, were examined by the AID technique to detect important interactions. Quantitative data for each of the three types of deterioration were analyzed separately and therefore will be described separately in the following sections.

Fracture Planes and Spalls

Of the 55 decks, 53 exhibiting spalls had insufficient concrete cover (less than the specified design cover) over the reinforcing bars. Although an earlier study (5) had shown 1½ in. to be the critical depth of cover, 1¼ in. appeared to be the critical depth on these 4-year-old decks. The recurrent pattern of spalling associated with insufficient cover speaks poorly for construction practices on these decks.

Of the two decks having spalls where steel depth was adequate, one exhibited delaminations of a latex-modified concrete overlay. The second deck had 12 fracture plane areas, totaling 4½ sq ft, and eight spalls, totaling 10⅞ sq ft. These were distributed in many locations on the deck. The spalls were uniformly ¼ to ½ in. deep (Fig. 6), but the depth of steel in both deteriorated and sound areas was 2½ to 3 in. Many of the smaller fracture planes were at least initiated by patching of the screed rail holes, whereas others appeared to be due to overfinishing. Extensive areas of map cracking (perhaps associated with rapid drying) were also noted on this deck.

Inasmuch as fracture planes are incipient spalls, they were combined with spalls in the AID analysis.

As explained above, insufficient reinforcement cover was detected in nearly all cases where spalls were observed. Unfortunately, due to the large number of decks observed, it was possible to measure steel depth only on decks exhibiting spalls or fracture planes. Therefore, with data only for the deteriorated decks, steel depth could not be included as a variable in the AID analysis. Its importance, however, should be obvious to the reader.

After factors shown to be unimportant in a preliminary AID analysis were deleted, nine remained as predictor causal factors: form type, superstructure type, retarder use, concrete flexural strength, de-icing chemical type, number of salt applications per year, average daily traffic, contractor, and structural stiffness. The effects of these factors on spalls and fracture planes are shown in Figure 7. (Abbreviations used are given in Table 2.) Note that only six of the nine factors were sufficiently

Figure 2. Chain drag device.

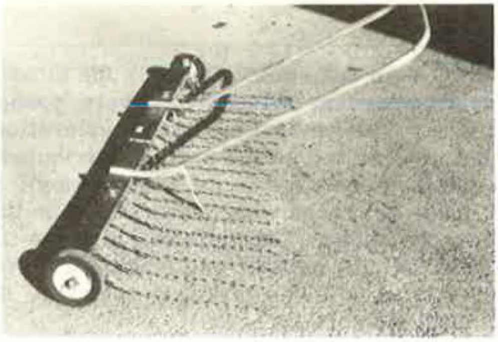


Figure 3. Delineation of fracture plane.

Figure 4. Surface wear deterioration.

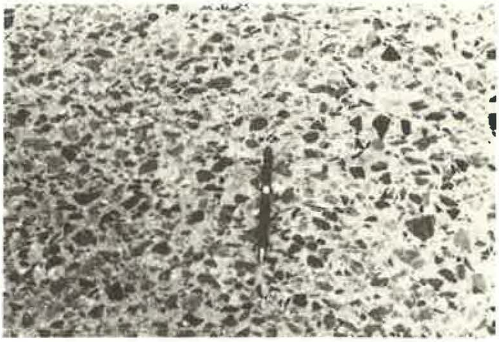


Figure 5. Distribution of types of cracks.

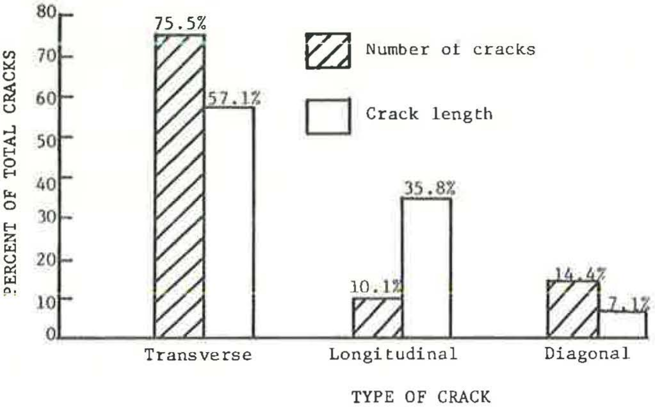


Figure 6. Example of ¼- to ½-in. deep spall.

Figure 7. Effects of six causal factors on spalls and fracture planes.

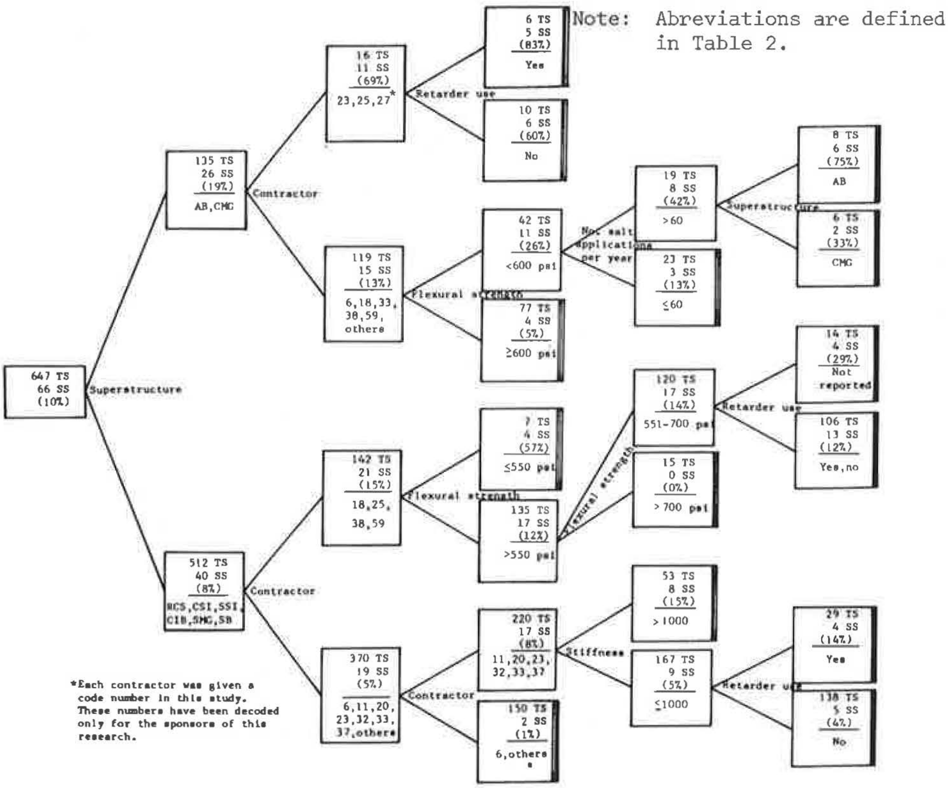


Table 2. Key to abbreviations.

Symbol	Definition
AB	Adjacent box beam (concrete) superstructure
C	Cinders
Ca	Calcium chloride
CI	Crack intensity (feet of cracks per 100 sq ft of deck)
CIB	Concrete I-beam superstructure
CMG	Continuous multigirder superstructure
CSI	Continuous steel I-beam superstructure
CV	Conventional removable (wooden) forms
md	Missing data
Na	Sodium chloride
RCS	Reinforced concrete slab superstructure
S	Sand
SB	Spread box beam (concrete) superstructure
SIP	Stay-in-place forms (metal corrugated forms)
SMD	Surface mortar deterioration (percentage of deck area affected)
SMG	Simple multigirder superstructure
SS	Spalled spans (including fracture planes)
SSI	Simple steel I-beam superstructure
ST	Crushed stone
TS	Total spans

important to appear on the tree diagram. (The AID program determined that the other three were not able to significantly reduce the unexplained variance.) This diagram for spalls and fracture planes will be explained in detail in the following paragraphs to provide insight into interpretation of the tree diagrams. (Similar tree diagrams for cracks and SMD will not be shown.)

It can be seen from the figure that the most important single variable explaining variation in the occurrence of spalls is superstructure type (recall that it was not possible to include steel depth as a variance). That is to say, of all the possible combinations in each of the nine predictor variables, the grouping of adjacent box and continuous multigirder superstructures against all other superstructures did the best job of explaining whether fracture planes or spalls or both occurred in the 647 total spans, i.e., explaining variance of the data. (Although 813 total spans were surveyed for deterioration, it was not possible to get complete data on each of the 32 causal factors. So we reduced the number of spans to 647 for the fracture plane and spall analysis, 623 for the transverse crack analysis, and 624 for the SMD analysis.) Of 135 adjacent box beams or continuous multigirder spans, 26, or 19 percent, had at least one spall, whereas all other superstructure types exhibited only 40 spalled spans of 512, or 8 percent. The grouping of two types against the remaining six allowed the greatest divergence between occurrences, i.e., 19 percent versus 8 percent. Any other combination of superstructure types would have given less than the 11 percent difference.

Of the 135 adjacent box beam and continuous multigirder spans, 16 spans, of which 69 percent had spalled, were built by only three contractors. Only 15 spalled spans, or 13 percent, occurred on the 119 remaining spans, built by all of the other contractors. Adjacent box beam and continuous multigirder bridges built by the better contractors performed well (about a 5 percent spall occurrence rate) when concrete with a flexural strength greater than 600 psi was used. Lower strength decks spalled more frequently, especially adjacent box beam decks when more than 60 salt applications per year were made. This last analysis is an excellent example of the ability of the AID technique to illustrate the effect of interactions. Of eight spans having adjacent box beam superstructures, greater than 60 salt applications per year, and a flexural strength of less than 600 psi, six spalled.

From the lower branch of the tree, 512 spans were built with superstructures other than adjacent box beams or continuous multigirders. Again, the contractor was the most important single factor associated with fracture planes and spalls. Contractor 6 and "others" (a group of 48 contractors who built fewer than six bridges in 1966) accounted for only two spalled spans out of 150. This excellent performance indicates that bridges resistant to fracture planes and spalls can be built. An intermediate group of contractors, 11, 20, 23, 32, 33, and 37, had 8 percent spall failures, and the poorest group, 18, 25, 38, and 59, had a spall occurrence rate of 15 percent. Among the spans constructed by the poorer contractors, four of seven with flexural test values of 500 psi or less exhibited spalls. It is also significant to note that no span having a flexural test strength greater than 700 psi spalled even though built by the poorer group of contractors. This speaks strongly for higher strength concrete (low water content or high cement factor or both) for bridge decks. The influence of retarder use on the 120 intermediate strength spans appears minimal, inasmuch as the AID program grouped "yes" and "no" against "not reported."

In the lowermost branch of the tree, of the 220 spans with 8 percent exhibiting spalls, stiffness appeared to correlate inversely to what one would expect. High stiffness should reduce flexibility and thus cracking, thereby reducing spall occurrence. In this analysis, however, very stiff spans appear to exhibit somewhat more spalling. A separate analysis on the entire 813 spans showed no significant correlation between stiffness and cracking at the 95 percent confidence level (2).

Retarder use appeared three times in the entire analysis. As stated previously for the group of spans having high flexural strength, retarder use was not a significant factor. However, in two of the three cases, there appeared to be a slight disadvantage to the use of a retarder.

Form type, de-icing chemical type, and average daily traffic did not appear as important factors in this analysis of spalling. Without question, superstructure type, contractor, and flexural strength are the primary factors controlling whether fracture planes and spalls occur. Unfortunately, the decisions to use adjacent box beam and continuous multigirder superstructures are usually dictated by physical constraints and are not subject to change. However, flexural strength can be altered by specification, and construction practices, as indicated by the contractor variable in this study, can and should be closely regulated. These analyses clearly show that, under the same specifications and at the same time, different contractors can produce durable and non-durable bridge decks.

Cracking

Similar analyses performed on crack data resulted in another tree diagram, shown in Figure 8. Although detailed interpretation of this analysis is left to the reader, note that again several variables proved important: contractor, form type (stay-in-place metal corrugated or conventional removable wooden forms), superstructure type, span length, and others.

Surface Mortar Deterioration

The surface mortar deterioration tree diagram is shown in Figure 9. Again the importance of the variable contractor is evident as is form type, different antiskid materials, average daily traffic, and others.

CONCLUSIONS

As stated earlier, one of the twofold objectives of the investigation was to provide a condition report of a large sample of fairly new bridges. A brief summary of the condition of the decks is presented in an earlier section of this paper [for a more detailed condition report, see Cady et al. (1)].

All of the causes of deterioration investigated in connection with the second objective are generally recognized today. Indeed, we chose to evaluate known causes of deterioration on a large sample of bridges in an attempt to determine the relative importance of these causes; for, although these causes of deterioration have been recognized for some time, we continue to experience inadequate deck performance.

Table 3 gives a summary of those factors that appeared most strongly related to deterioration in order of decreasing importance. Note the relationship between construction practices and all three major forms of deterioration. This suggests that improved inspection and quality control at the jobsite would yield maximum benefit, particularly with regard to providing adequate concrete cover.

RECOMMENDATIONS

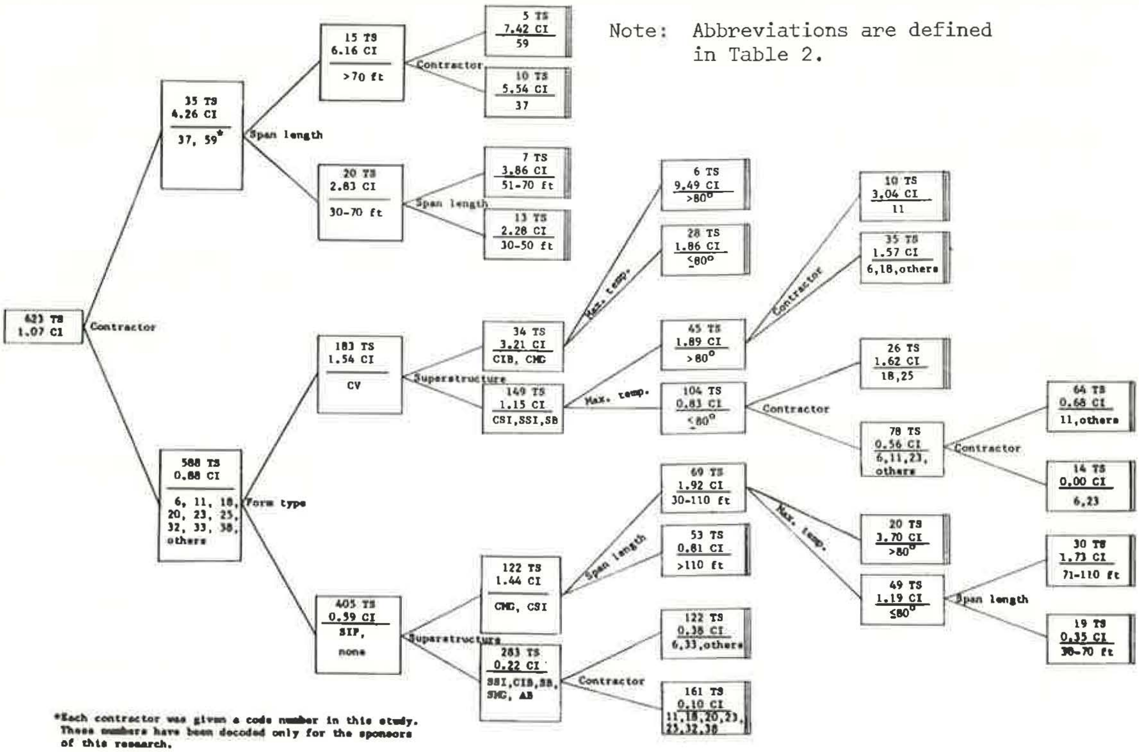
It is hoped that the information presented will allow policy and procedural changes to be directed toward elimination of the most serious causes of deterioration. Whereas some of the factors in Table 2 such as average daily traffic, de-icer usage, and superstructure type are constraints generally outside the control of engineers, two very strong relationships suggest changes that could be made.

For these study bridges, early spalling was almost always associated with inadequate concrete cover. Provision of sufficient cover is absolutely essential to durable bridge decks. The authors, therefore, recommend that provision of the specified cover be made a basis-of-payment item, similar to the slab thickness basis-of-payment for highway pavements. Also, in view of the improved performance of high-strength decks, the authors recommend that flexural strength requirements be increased to 700 psi for bridge deck concrete.

ACKNOWLEDGMENTS

This study was sponsored by the Pennsylvania Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration. The views presented are those of the authors and not necessarily those of the sponsoring agencies.

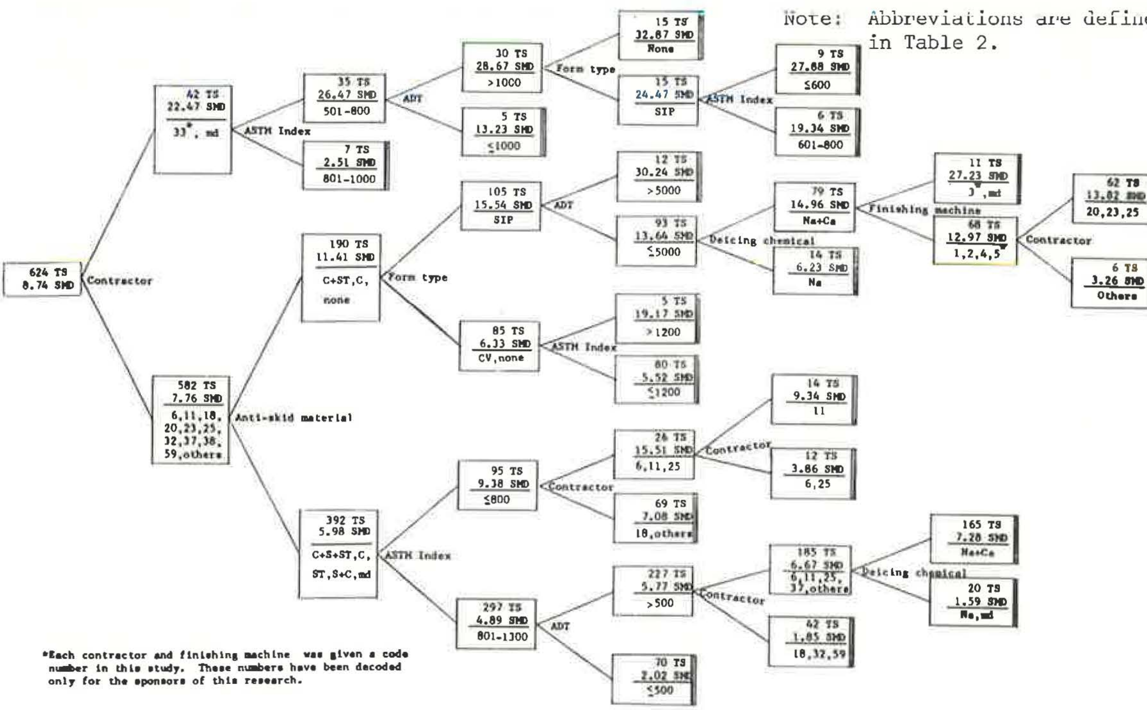
Figure 8. Effects of five causal factors on crack intensity.



Note: Abbreviations are defined in Table 2.

*Each contractor was given a code number in this study. These numbers have been decoded only for the sponsors of this research.

Figure 9. Effects of seven causal factors on surface mortar deterioration.



Note: Abbreviations are defined in Table 2.

*Each contractor and finishing machine was given a code number in this study. These numbers have been decoded only for the sponsors of this research.

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Page 55, in the Discussion by Chamberlin, Amsler, and Jaqueway, after the text table, insert the following: Data are in linear feet of transverse cracks per 100 ft^2 of bridge deck.

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DISCUSSION

W. P. Chamberlin, D. E. Amsler, and J. K. Jaqueway,
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The authors' observation on the association of major defects with the use or nonuse of corrugated metal stay-in-place (SIP) forms is welcome, particularly in view of the strong opinions that the subject evokes in the absence of factual information.

They have shown that 4-year-old, SIP-formed decks in Pennsylvania have far fewer transverse cracks than conventionally formed decks of the same age but that they exhibit somewhat more deterioration of surface mortar, primarily in the form of wear. They conclude from this evidence that form type is an important determinant of both transverse crack intensity and surface mortar deterioration.

A recent study of the condition of 716 bridge decks in New York State (11) supports the authors' findings with regard to the association of lesser transverse crack intensity with the use of SIP forms. Data given below show that SIP-formed decks in New York have less than one-half the amount of cracking of those formed by conventional methods.

Source	SIP	Conventional
(1)	0.46	1.50
(11)	0.55	1.27

The New York study includes most decks built in the state during the 7-year period, 1965 to 1971—about one-half with SIP forms (Fig. 10).

The lesser crack intensity found in SIP-formed decks in New York appears to result from a combination of two circumstances (Table 4): (a) substantially fewer cracked spans and (b) substantially fewer cracks in those spans that are cracked.

Table 3. Major forms of deterioration and their causes.

Type of Deterioration	Cause
Fracture planes and spalls	Depth of steel, superstructure type, construction practices, flexural strength, retarder use, number of salt applications per year
Cracks	Construction practices, form type, span length, superstructure type, maximum placement temperature
Surface mortar	Construction practices (especially finishing), antiskid material (may be related to use of $CaCl_2$ on stock piles), form type, average daily traffic, de-icing chemical type, finishing machine
Miscellaneous	Difference in elevation of approach slab and deck as related to broken slab edges, aggregate sources as related to popouts, snowplow damage as related to parapet deterioration

Figure 10. Distribution of SIP-formed decks in New York.

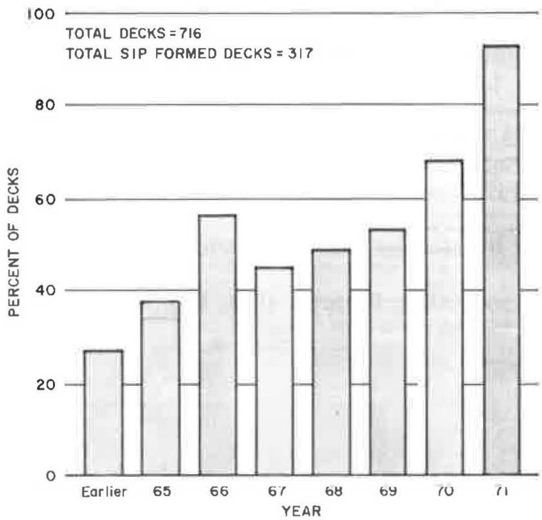

Figure 11. Frequency of transverse cracking by year of construction.

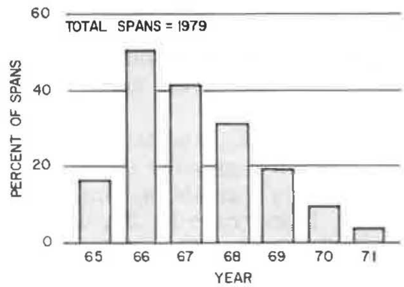

Table 4. Transverse cracks in New York bridge decks.

Feature	Forming Method: SIP	Forming Method: Conventional
Proportion of spans cracked, percent	22.1	32.9
Cracks per 100 sq ft of cracked span	0.22	0.35
Mean crack length, ft	11.5	11.2

Although it is tempting to infer a cause-and-effect relationship between deck cracking and forming method, on the basis of the association observed between these two factors, it is perhaps premature without either more statistical evidence to support the authors' finding or a better understanding of the mechanism through which the two observations are related or both. With regard to these deficiencies, the New York results do not wholly support the authors' conclusion of a causal relationship.

The New York experience appears at first glance to supply the additional statistical evidence. Yet the lower crack intensity in SIP-formed decks in New York also can be explained by factors other than forming method. As shown in Figure 10, SIP-formed decks in New York, as a group, are younger than those formed by conventional methods. This fact, combined with the strong age-dependence of transverse cracks (Fig. 11), supplies an alternate explanation for the higher crack intensity associated with conventionally formed decks; i.e., conventionally formed decks are cracked more intensely because more are older. Unfortunately (for this purpose) the objectives of the New York survey were primarily descriptive, and the information at hand does not permit a more exhaustive study of causal factors, which might resolve this question. The Pennsylvania study, of course, was confined to decks of the same approximate age and does not suffer from this dilemma.

Regarding mechanisms, the authors (1) have speculated that the lesser crack intensity associated with the use of SIP forms could result from slower water loss in the fresh concrete with more gradual shrinkage or increased stiffness of the deck due to composite action. If this water-loss hypothesis is true, then cracking associated with forming method probably occurs early in deck life when drying (and shrinkage) is more rapid and strength not fully developed. However, this seems contrary to the implication of Figure 11, and to the findings of other investigators of bridge deck cracking (12), that the occurrence of transverse cracks is age-dependent. Resolution of this matter may rest on the question of whether transverse cracks resulting from such causes really become more frequent with deck age or develop early and just become more apparent with age.

Our point in this discussion is that the association of SIP-formed decks with lower transverse crack frequencies, observed by the authors in Pennsylvania, is supported by experience in New York but that the acceptance of a cause-and-effect relationship between the two should await additional statistical support or a better understanding than is now available of the particular circumstances under which form-related bridge deck cracks occur.

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BRIDGE DECK PERFORMANCE IN VIRGINIA

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•ONE OF THE MOST comprehensive programs for evaluation and definition of the problem of premature deterioration of bridge decks was that undertaken cooperatively by the Portland Cement Association, the Bureau of Public Roads, and eight state highway departments. Detailed studies of a few bridges were made in four states and these have been reported (1, 3, 5, 6). As a part of this program, surveys were made on 100 to 150 bridges in each of eight states including Virginia. These bridges were randomly selected to represent all bridges within a state. The results of these random surveys have been reported (13), and the entire project has been summarized (14). The data from this research are voluminous but some important results are given in Table 1.

Compared with those in the other states, the bridges in Virginia showed substantially less cracking and spalling and significantly more scaling. The lower incidence of cracking was attributed to a greater proportion of simple spans in the Virginia sample. It was also concluded that the higher frequency of scaling was due to the comparatively late adoption of air entrainment in the state. It is interesting to note that the other southern state, Texas, which did not use air-entrained concrete, also showed a high incidence of scaling, whereas Minnesota, with a much more severe climate but also a longer utilization of air entrainment, showed the lowest incidence of scaling of any of the states (excluding California).

Although spalling was the least common of the three major defects, it was, in the words of the final report (14), "without doubt—the most serious and troublesome kind of bridge deck distress." No reasons for the very low incidence of spalling in the Virginia decks were offered. In the 10 years since the field surveys were conducted, concern has been expressed nationally that spalling is becoming more prevalent, and several significant cases in Virginia have required major repairs. Unpublished reports from California indicate that spalling, which was not a significant problem in 1961, now is one.

To determine the change in performance characteristics of the Virginia bridge decks, during the summer of 1970 we resurveyed the structures included in the PCA-BPR survey, last inspected in 1961. This was apparently the first such resurvey conducted in the eight states included in the random survey.

The purpose of the resurvey was to establish the change in condition of the decks, particularly with regard to the further development of surface spalling. Some insights concerning the reproducibility of the survey techniques were also anticipated.

PROCEDURE

Each state cooperating in the PCA-BPR survey used a standard investigation procedure. A copy of the survey form and definitions used are available in the previously referenced report and are listed in the Appendix.

The PCA-BPR method of inspection was meticulously utilized throughout the resurvey. Data were collected in a manner that permitted evaluation of the deterioration on each span of each bridge as well as on the bridge deck as a whole. In this procedure, scaling, spalling, cracking, rusting, and popouts are the types of deterioration emphasized.

Sponsored by Committee on Performance of Concrete—Physical Aspects

Scaling is reported as a percentage of the span's deck area for the average scaled condition, and simultaneously the most severe scaling condition on each span is noted. Surface abrasion is included in scaling where a mixture of the two occurs. Scaling is defined as light, medium, heavy, or severe. Surface spalling is recorded as the number of large or small spalls on the span. Generally, any spall smaller than a foot in length or diameter is considered small.

Cracking is recorded by type and severity. A few cracks per span is classified as light. A large amount of cracking is considered heavy, and an intermediate condition is defined as medium cracking. The types of cracking are transverse, longitudinal, diagonal, pattern or map, random, and "D" cracking. When the bridge span inspected has a skew, cracking parallel to the direction of the skew is classified as transverse.

Joint spalls were recorded by their position on the bridge deck and their size or length along the joint. Rusting was noted as the number of rust spots, and popouts were listed as few or many in accordance with the inspector's opinion of how numerous they appeared on the bridge span deck.

Before the bridges were inspected for signs of deterioration, several engineers, laboratory technicians, and the authors performed preliminary practice inspections on local bridge decks. The object of this was to try to achieve the most uniform and accurate rating of the deterioration by each person individually analyzing the bridge deck and then to compare respective estimates of the amounts and types of deterioration. Differences in interpretation were reconciled and observation techniques refined. In this way, a consistent definition of the several types of deterioration could be obtained.

Following these preliminary practice inspections, the junior authors performed all of the field inspections of these bridge decks. On the first inspection trip, an experienced concrete technician accompanied them to aid in the initial inspections and to refine the observation techniques.

In the 1961 PCA-BPR survey in Virginia, 140 bridges comprising 452 spans were randomly chosen. Of these, 84 bridges had 262 uncovered spans, which were inspected. In 1970, 66 bridges comprising 206 uncovered spans were available for the resurvey. The important characteristics of these bridges are given in Table 2.

After all the data had been collected, they were analyzed by a computer. Prior to analyzing the resurvey data, the computer program was applied to those data obtained for the 1961 survey and published in the PCA-BPR random survey report (13). The original field data sheets were available so that reproduction of the published results was used as a check on both the computer program and the data analysis. After this check was obtained, the new data from the resurvey were inserted into the program, and new results were obtained by the same procedure utilized in 1961.

RESULTS

The results are given in Table 3. The format is the same as that used in Table 2B of the PCA-BPR report (13) so that comparisons can easily be made. In Table 4, the same data are given, but spans with defects classified in the least severe category have been combined with those showing no defects. It is believed that this grouping permits a more realistic comparison between the results of the two surveys and minimizes the differences attributable to judgment of the individuals conducting the surveys. For example, the classifications light scaling and light pattern cracking depend strongly on the point of view of the inspector. It is reasoned that a defect of sufficient magnitude to be classified above the minimum level would have been recorded by both groups of inspectors and that valid comparisons could thus be made.

As would be expected, the observed deterioration increased in frequency and severity on all of the bridges. As reflected in Tables 3 and 4, by far the most prevalent defects were scaling and cracking. The relative order of the three most prevalent defects, i.e., scaling, cracking, and popouts, remained the same between 1961 and 1970. In 1970 surface spalling was the fourth most prevalent, whereas in 1961 it was of the same order as rusting and joint spalling. In 1970, only 5 percent of the decks were free of scaling and only 25 percent were free of cracking in some form. It should be noted, however, that much of this distress would be classified as light. Eighty-seven percent

Table 1. Percentage of three most common defects observed in PCA-BPR study and in Virginia study.

Defect	Seven States	Virginia
Cracking (all types)	69.7	33.2
Scaling	22.9	43.1
Spalling	8.1	0.4

Table 2. Comparison of Virginia bridge characteristics in PCA-BPR study and in Virginia study.

Bridge Characteristics	1961	1970
Number		
Covered	56	14
Uncovered	84	66
Total	140	80[a]
No. of spans		
Covered	190	42
Uncovered	262	206
Total	452	248[a]
Year built[a]		
1940 to 1947	8	7
1948 to 1955	112	87
1956 to 1961	142	112
Traffic volume[b], ADT		
1 to 750	115	113
751 to 7,500	114	66
>7,500	33	27
Span length[b], ft		
<45	139	111
45 to 90	118	90
>90	5	5
Air entrainment[b]		
Specified	134	96
Not specified	126	108
Unknown	2	2

[a]Four bridges comprising 14 spans were relocated or replaced between the two surveys.
[b]Uncovered spans only.

Table 3. Distribution of span defects.

	1961 Condition		1961 Condition of Covered Spans		1961 Condition of Uncovered Spans		1970 Condition	
Span Defects	Number	Percentage	Number	Percentage	Number	Percentage	Number	Percentage
No scaling	149	57	19	45	125	61	11	5
Scaling	113	43	23	55	81	39	195	95
No cracking	175	67	23	55	143	70	51	25
Cracking	87	33	19	45	63	30	155	75
Transverse	66	25	15	36	47	23	121	59
Longitudinal	12	5	4	10	7	3	30	15
Diagonal	7	3	2	5	4	2	9	4
Pattern	19	7	5	12	12	6	47	23
"D"	0		0		0	0	0	0
Random	13	5	4	10	9	4	105	51
No rusting	261	>99	41	98	206	100	206	100
Rusting	1	<1	1	2	0	0	0	0
No surface spalling	260	99	41	98	205	>99	186	90
Surface spalling	2[a]	1	1	2	1	<1	20	10
No joint spalling	260	99	40	95	206	100	200	97
Joint spalling	2	1	2	5	0	0	6	3
No popouts	233	89	35	83	184	89	169	82
Popouts	29	11	7	17	22	11	37	18

[a]Data published by PCA-BPR (13) erroneously listed 1 rather than 2 spalled spans.

Table 4. Distribution of more severe span defects.

	1961 Condition		1961 Condition of Covered Spans		1961 Condition of Uncovered Spans		1970 Condition	
Span Defects	Number	Percentage	Number	Percentage	Number	Percentage	Number	Percentage
No or light scaling	215	82	35	83	183	87	143	69
Medium, heavy, and severe scaling	47	18	7	17	23	13	63	31
No or light cracking	252	96	39	93	199	97	180	87
Medium or heavy cracking	10	4	3	7	7	3	26	13
Transverse	6	2	2	5	4	2	13	6
Longitudinal	0	0	0	0	0	0	1	<1
Diagonal	2	<1	0	0	2	1	1	<1
Pattern	0	0	0	0	0	0	4	2
"D"	0	0	0	0	0	0	0	0
Random	2	<1	1	2	1	<1	7	3
No rusting	261	>99	41	98	206	100	206	100
Rusting	1	<1	1	2	0	0	0	0
No or small surface spalls	261	>99	42	100	205	>99	193	94
Large surface spalls	1	<1	0	0	1	<1	13	6
No joint spalls	262	100	42	100	206	100	200	97
Joint spalls	0	0	0	0	0	0	6	3
No or few popouts	262	100	42	100	206	100	199	97
Many popouts	0	0	0	0	0	0	7	3

of the spans were free of medium or heavy cracking, while about two-thirds of the spans were free of the more serious forms of scaling.

Surface spalling has progressed to the point where 10 percent of the spans are affected, 6 percent in the more severe classification. Although this increase is substantial, it is interesting to note that the current frequency of spalling on the Virginia sample is equal to or less than that reported from three of the seven other states in 1961.

No rusting was observed in 1970. Apparently rusting or the conditions leading to rusting were the basis for resurfacing of the one span observed in 1961.

In general, the performance of these bridge decks is not alarming and would be considered adequate or above average when compared with other published information on performance of decks in other areas of the country.

In the analysis of the data from the 1970 survey, a primary concern was to determine and report the increase of concrete deterioration on the bridge decks during the interval between surveys. It must not be overlooked that decks on 14 of the 80 bridges that were inspected had been resurfaced with a bituminous surface. Thus, 42 spans were covered. Inasmuch as concrete deterioration often necessitates deck resurfacing, the inspectors tried to discover whether this was the reason for the placement of bituminous concrete on the bridges. The district bridge engineers were requested to convey any information pertaining to the conditions of these deck slabs prior to the placement of the overlays.

From the replies, it would seem that for the most part these resurfacings were for reasons other than concrete distress. This is consistent with the data given in Table 4, which show that the condition of spans in 1961 was close to the average for the entire sample.

As judged by the data given in Table 4, it appears that the performance of the decks has been adequate with the possible exception of the resistance to surface scaling. The increase in spalling, though limited to a few spans, should be viewed with concern because of the difficulty associated with its correction.

SPECIFIC DEFECTS

The classes of defects will now be discussed individually in more detail, using comparisons shown in Figures 1 to 9. The data designated as "1961—all uncovered spans" are the same as those in the PCA-BPR report (13).

Scaling

The distribution of the maximum and average scaling conditions is shown in Figures 1 and 2. The progression of scaling increases with both age and traffic volume. The progression with traffic volume is shown in Figure 3. The degree of progression with age was very similar to that with traffic volume. [In the full report (14) from which this paper was taken, a more extensive discussion of the specific factors is given.] It is likely that these increases represent the combined effects of several factors including (a) more applications of de-icing chemicals on roads with high traffic volumes, (b) greater numbers of freezing and thawing cycles with age, and (c) difficulties of separating abrasion from light scaling. In any event, the differences due to age and traffic are not particularly significant.

The combined influence of air entrainment and age is shown in Figure 4. This influence was less obvious in 1970 than it was in 1961. The reason is at least twofold. One is the difficulty in distinguishing between light scaling and abrasion. Whereas reduction of scaling is attributed to entrained air, at the same time slight reductions in strength accompanying its use might be associated with increased abrasion. It is much more likely that the lack of significant reduction in scaling from air entrainment reflects the fact that the amount of air specified was not sufficient to provide resistance to scaling. The specifications in force at the time of construction required 3 to 6 percent air. Based on the observations from the Virginia Highway Research Council's (VHRC) special study of deck construction, there was a strong tendency to work to the lower limit rather than to the center of the range. It is thus probable that the majority of the spans do not contain what is currently deemed as adequate air entrainment. The Virginia specifications were revised in 1965 to require an air content of $6\frac{1}{2} \pm 1\frac{1}{2}$ percent.

Figure 1. Distribution of maximum scaling condition.

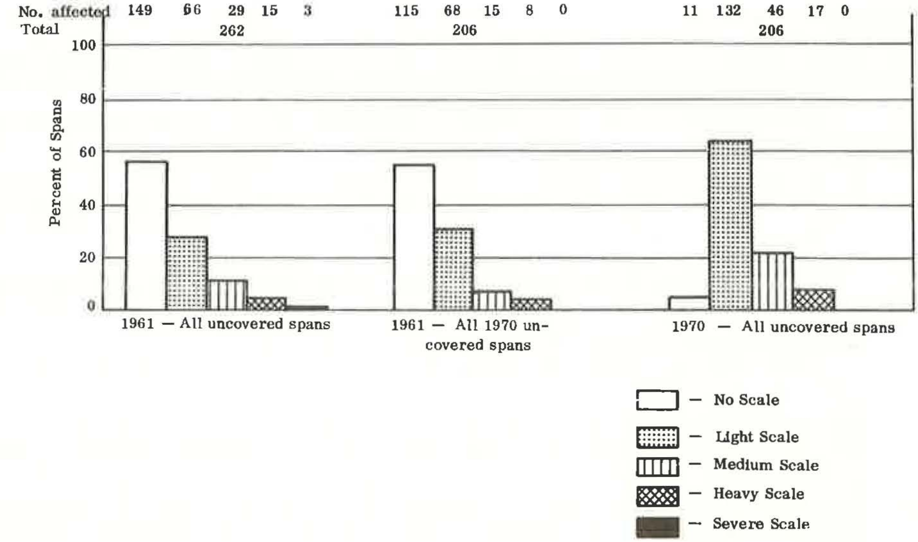


Figure 2. Distribution of average scaling condition.

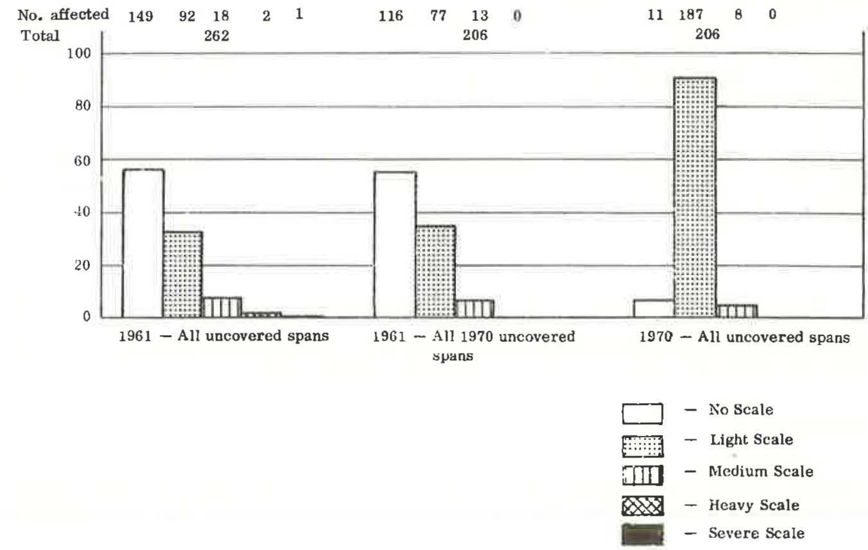
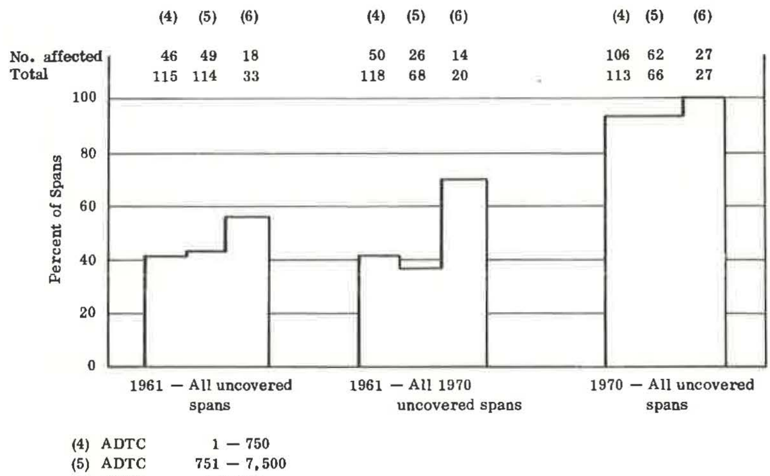


Figure 3. Influence of traffic volume on occurrence of scaling.



It is significant to note that the 11 spans that were free from scaling were all air-entrained. At the same time, the average daily traffic counts of all of these unscaled spans is less than 7,500, and more than half have an average daily traffic count less than 750. Spans on low-volume roads would be expected to have received few if any applications of de-icing chemicals.

Cracking

Cracking was the second most prevalent defect in both 1961 and 1970. Most of the cracking is light and of little comparison (Table 4). The largest increase, tenfold, was in the category of random cracking. Most of this was light and is believed to reflect differences between the ratings of the inspection teams rather than in actual performance. This probability is discussed later.

The most prevalent type of cracking was transverse, and its occurrence is shown in Figure 5. In 13 spans (6 percent) the severity is classified as medium or heavy. Approximately three times as many spans showed this level of cracking in 1970 as did in 1961.

The influence of span length on cracking is shown in Figure 6. The effect of traffic volumes on the occurrence of transverse cracking is shown in Figure 7. Both figures reflect the expected trends, namely, an increase with span length and traffic volume. In 1961 these trends were not evident because the longer spans and heaviest traveled bridges were also the youngest. In 1970 there was little difference between the two highest classifications of either volumes or span lengths, but spans shorter than 45 ft and carrying fewer than 750 vpd showed significantly less transverse cracking than did the other two classes. Transverse cracking can result from either loads or long-time volume changes, both of which are time-dependent. The effect of age on the occurrence of transverse cracking was also as expected although there is little difference in the occurrence on the spans in the two youngest categories. Because cracking and surface spalling are often related phenomena, the relationship between these two defects was evaluated and is discussed later.

In addition to transverse cracking, only random cracking showed a substantial change (Fig. 8). In all cases the severity was not alarming. No "D" cracking was observed in either survey. "D" cracking is a serious problem in pavements placed in some areas of the country, but the conditions necessary for its occurrence are not operative except in slabs-on-grade.

In general, cracking is not a serious problem and is believed to be within expected limits.

Spalling

Because of the recent nationwide concern over the development of surface spalling, particular attention was given to this defect. Its occurrence is shown in Figure 9. As noted earlier, spalling was more prevalent in 1970 than in 1961 and has shown proportionally a greater increase than other defects. The number of spans with spalling increased from one to 20. At the same time spalling is still much less of a problem on the bridges in the Virginia sample than on those in many of the other states included in the random survey. On 13 spans (6 percent), spalling was recorded as large. One bridge contained 33 spalls, another had 24, one had 17, and the remainder had less than 10. Thus, only three bridges were significantly affected.

The magnitude of the problem did not warrant and the scope of this project did not permit an extensive evaluation for incipient spalls or "hollow areas," but the comparative absence of spalls suggests that a further, more intensive study might shed some light on why some decks spall and others do not.

The data were studied for relationships between the simultaneous presence of spalling and transverse cracking. The formative mechanism of spalls is such that transverse cracking could be expected to precede the appearance of spalls. As noted earlier, 121 of the 206 spans (59 percent) contained transverse cracks. Of these 121 spans, 107 (89 percent) had no spalling, whereas 14 (11 percent) did. Spalls were found in 20 spans, and, of the 20, six had no transverse cracking while 14 had both. From a study of these

Figure 4. Influence of air entrainment and age on occurrence of scaling.

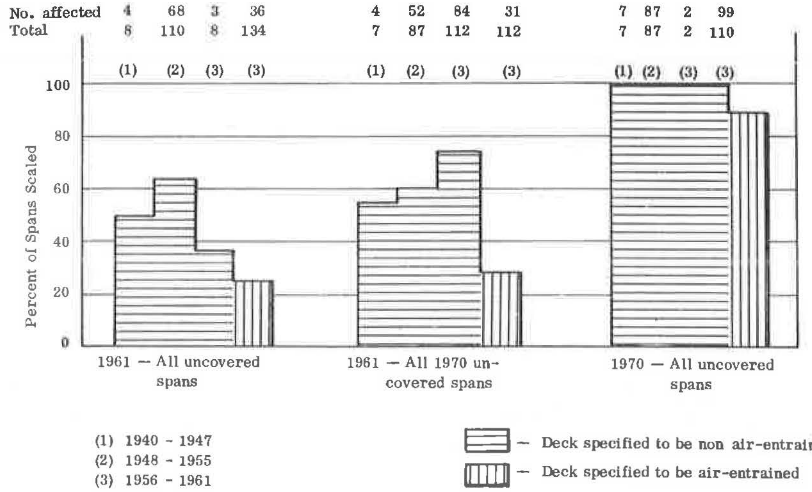


Figure 5. Distribution of transverse cracking.

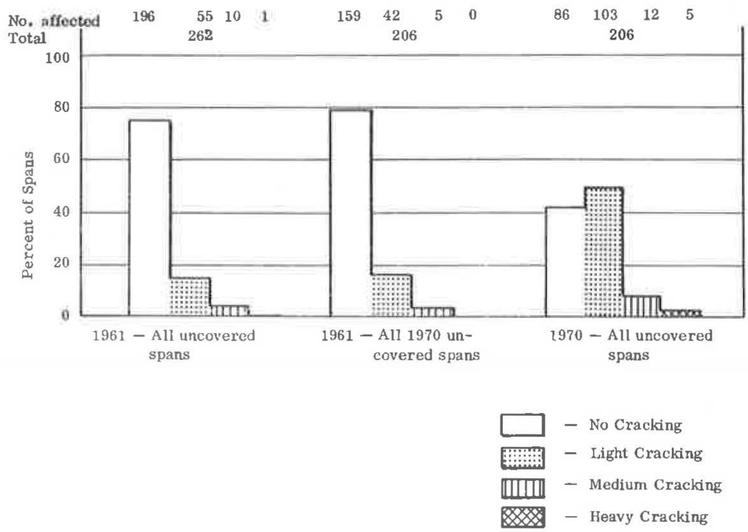
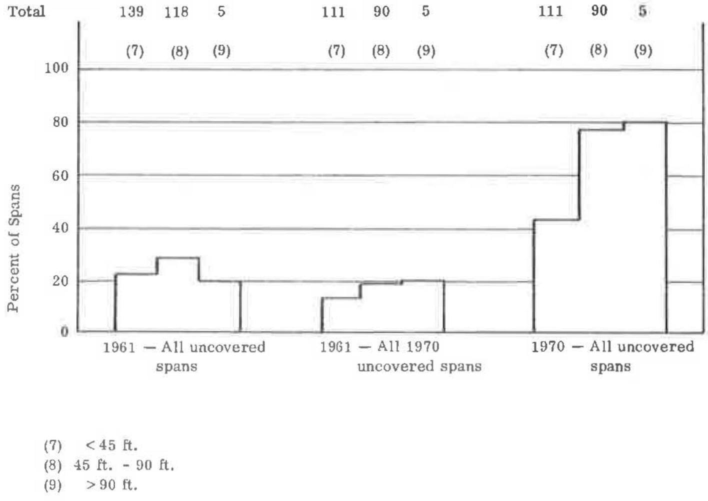
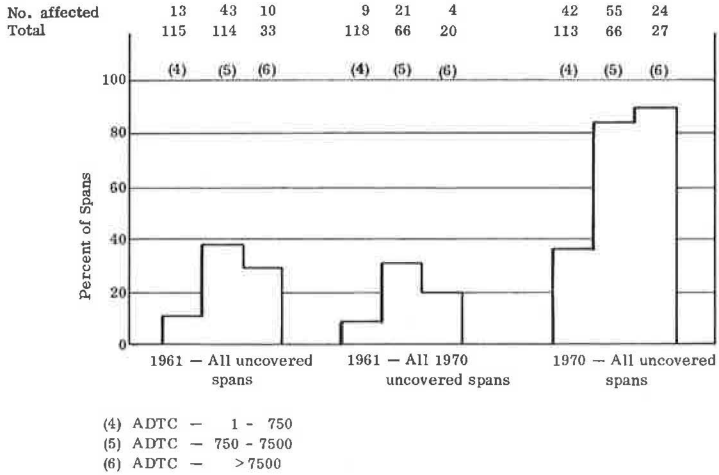


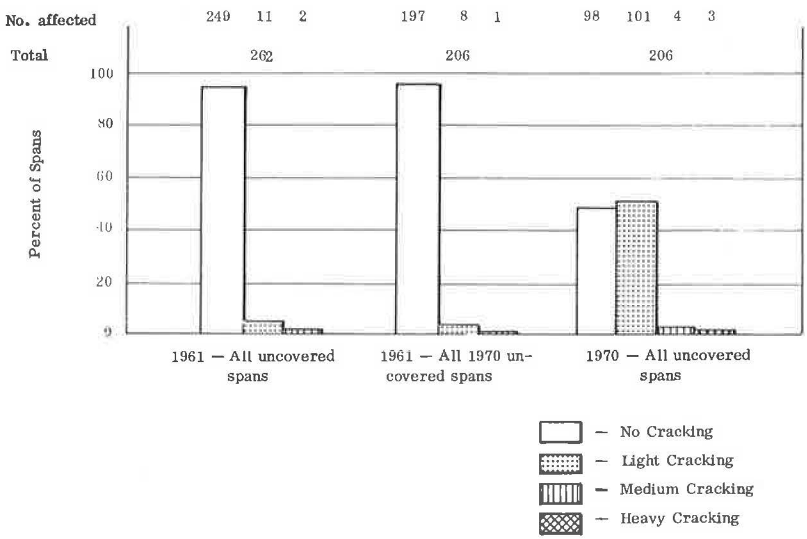
Figure 6. Influence of span length on occurrence of transverse cracking.



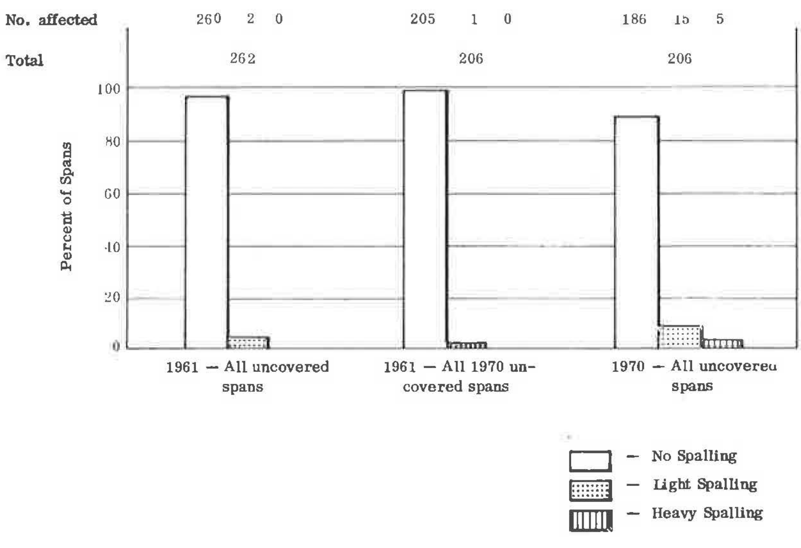
igure 7. Influence of traffic olume on occurrence of ransverse cracking.



igure 8. Distribution of random racking.



igure 9. Distribution of spalling.



data, there seems to be no particularly consistent relationship between spalling and transverse cracking, although the two defects often occur simultaneously.

Popouts

The number of spans containing popouts did not increase substantially. It was observed, however, that the occurrence of popouts was largely confined to the Northern Virginia area. This is explained by the occurrence of lightweight chert in the locally available coarse aggregate.

PCA-BPR SAMPLE VERSUS VHRC STUDY

In 1963, a special study was made of 17 randomly selected bridges then under construction to study the influences of finishing and other construction procedures on the properties of the freshly mixed and hardened concrete and those on the performance of the decks. On each of these 17 structures, one span was studied in detail during construction. The 17 bridges contained 78 spans. Two of the structures were much larger than the others, containing eight and 13 spans respectively. The remaining 15 bridges, containing 60 spans, were more representative of the typical bridge constructed in Virginia. The detailed evaluation of these decks has been discussed in a VHRC report. Here a comparison will be made of the changes in condition of the two samples during the period since the initial surveys (Table 5).

The agreement is unusually good. Several of the categories (popouts, surface spalling, and rusting) are in exact agreement. Joint spalling shows the largest disparity, and scaling and cracking show slight differences. In both samples the relative order of severity for the six different types of cracking is the same, and the individual values are in good agreement.

Factors Influencing Performance

The variety of conditions and the long time periods for construction and performance of the structures making up the Virginia sample do not lend themselves to a detailed cause-effect analysis. Several factors known to affect deck performance are worthy of mention as a general backdrop against which to view the results. The following four will be discussed: (a) specification requirements for concrete used in decks, (b) required cover over upper reinforcing steel, (c) policy on de-icing, and (d) climatological characteristics.

The degree to which compliance with the requirements for the first two factors was obtained is obviously an important but nondocumentable factor. Many problems can be traced to failure to fulfill completely the requirements specified for concrete quality and cover. Assuming a relatively constant degree of quality control, the requirements specified furnish a basis for comparison, particularly with performance in the other states included in the nationwide survey.

Concrete Quality—Significant specification requirements for concrete are given in Table 6. It can be seen that, during the time period of construction of the sample bridges (1940 to 1961), the requirements were essentially the same. The most significant change was the introduction of air entrainment in 1954. The oldest group of bridges (1940 to 1947) in the survey may have contained concrete of a slightly higher water-cement ratio than those covered by the later specifications. The number of bridges in the oldest class was small compared with the total. Significant upgrading of the requirements did not occur until 1966. Prior to that time the concrete specified would have had borderline resistance to weathering when judged in light of subsequently developed knowledge.

Cover—Insufficient cover of the upper reinforcing steel has been widely identified as a primary cause of surface spalling. Research in Kansas (2) showed a very high tendency for deterioration at covers of 1.5 in. or less and very little deterioration for covers of 2 in. or more. Spellman and Stratfull (12) have developed an empirical relationship to relate the variation of chloride content in concrete to the depth below the surface. A minimum of 2 in. of clear cover is widely recommended by various agencies prominent in concrete technology (8).

During the period of construction of the bridges included in the sample, the requirements for location of the uppermost steel was 2 in. to the center of the uppermost bar. This would be approximately $1^{11}/_{16}$ in. of clear cover. This cover requirement is adequate with respect to protection from spalling but is close to the breaking point of the data developed in Kansas (2). The requirement was increased in 1966 to provide $2^{1}/_{4}$ in. to the center of the uppermost bar, which is approximately $1^{15}/_{16}$ in. of clear cover. This is very close to the 2-in. value now judged to be necessary.

De-icing Policy—Prior to World War II there was little use of de-icing chemicals. Use was limited to very heavily traveled routes, with the remainder of the roads being cleared by plowing. The bare pavement policy developed in the mid-1940s, essentially after World War II, and resulted in the widespread use of chemicals for de-icing. For concrete pavement surfaces, calcium chloride, rather than sodium chloride, was required until about 1960. Since then, either sodium or calcium chloride has been used on concrete pavements. As in other areas, Virginia witnessed a fivefold increase in the use of chloride during the period 1953 to 1970.

Environmental—It is well recognized that the environment to which concrete is exposed greatly influences its performance. It is apparent that this performance is affected by temperature variations, particularly in the vicinity of the freezing point and simultaneously with the presence of moisture. The combination and quanitification of these two characteristics into a meaningful measure applicable for concrete are complex, and numerous techniques have been used. There is still much to be done as reflected by the fact that the area of assessing and quantifying the environmental characteristics to which concrete is exposed in practice was given highest priority in a recent summary of research needs (7).

There is a wide variation of effective freezing and thawing cycles across Virginia: as few as 23 in the southeast and as many as 91 in the northeast. With respect to freeze-thaw cycles, 65 to 70 are typical in the western areas of the state. These represent frequencies as high as are found in the United States with the exception of the Rocky Mountain areas (14).

Attempts to correlate performance characteristics with single calculated parameters such as freezing or weathering indexes were unsuccessful. Larson and Malloy (4) likewise found that a correlation of macroclimate with performance had not been successful and that an approach directed toward study of microclimate should be pursued.

RELIABILITY AND REPRODUCIBILITY OF THE SURVEYS

The many factors affecting deck performance and the variability, even within a given span, raise questions on the reliability and reproducibility of these and similar surveys. Although no direct proofs are possible, several indications from the experiences in Virginia lend support to the reliability.

It should be borne in mind that the random survey (13) did not include the same number of bridges from each geographic area.

The sample turns out to be a relatively good distribution, however, although there were relatively few in two areas that have comparatively mild exposures. The same is true, however, for one area that has a severe exposure. Thus the sample to some extent represents the median exposure.

The concept on which the PCA-BPR random survey was based was that the condition of the bridges within a given state would be estimated from surveys of a randomly selected source and that randomly drawn samples from the eight participating states would reflect the condition of bridges over the nation. The sample size was determined by statistical concepts with the intent that the limit of error would be ±8 percent, with a probability of not exceeding this limit fixed at two standard deviations; that is, 95 percent of the results would fall within the ±8 percent limit.

At approximately the same time that the random survey was conducted in Virginia, VHRC was conducting a less detailed survey of all of the decks in the four western districts. This survey, made in 1964, was a part of the study of potentially carbonate aggregates (11). Some results from a single district are of interest and are given in Table 7.

Table 5. Distribution of span defects in PCA-BPR study and in finishing study.

Defect	PCA-BPR Spans (percent)	Finishing Study	
		Spans (percent)	Structures
Covered	17	23	2
Uncovered	83	77	15
No scaling	5	18	3
Scaling	95	82	12
No cracking	25	10	1
Cracking	75	90	14
Transverse	59	63	12
Longitudinal	15	30	7
Diagonal	4	8	3
Pattern	23	38	8
"D"	0	0	0
Random	51	55	13
No rusting	100	100	15
Rusting	0	0	0
No surface spalling	90	91	13
Surface spalling	10	9	2
No joint spalling	97	71	14
Joint spalling	3	29	1
No popouts	82	82	11
Popouts	18	18	4

Table 6. Concrete requirements, 1938 to 1970.

Year	Cement Content (sacks/ cu yd)	Water-Cement Ratio (gal/ sack)	Air Content (percent)	Slump (in.)	Maximum Agg. Size (in.)	28-Day Strength (psi)	Maximum Aggregate Loss									
							Los Angeles Abrasion of Coarse Agg. (percent)		Sulfate Soundness				Freeze-Thaw			
									Coarse Agg.		Fine Agg.		Coarse Agg.		Fine Agg.	
							100 Rev.	100 Rev.	Percent	Cycles	Percent	Cycles	Percent	Cycles	Percent	Cycle
1938	$6\frac{1}{4}$	6	–	2 to 5	1	3,000	10	40					10	15		
1947	$6\frac{1}{4}$	6	–[a]	2 to 5	1	3,000	9	35	8	5	8	5	5	15	5	15
1954	$6\frac{1}{4}$	$5\frac{1}{2}$	3 to 6[a]	0 to 5	1	3,000	9	35	8	5	8	5	5	15	5	15
1958	$6\frac{1}{4}$	$5\frac{1}{2}$	3 to 6	0 to 5	1	3,000	9	35	8	5	8	5	5	15	8	15
1966	$6\frac{3}{4}$	$5\frac{1}{4}$	$6\frac{1}{2} \pm 1\frac{1}{2}$	2 to 4	1	4,000	9	40	12	5	12	5	5	20	5	20
1970	$7\frac{1}{4}$	$5\frac{1}{4}$	$6\frac{1}{2} \pm 1\frac{1}{2}$	2 to 4	1	4,000	9	40	12	5	18	5	5	20	8	20

[a]Air entrainment was first used in pavements in 1948. It was used experimentally in several bridge decks prior to being incorporated into specifications.

Table 7. VHRC and random survey results on comparable spans.

Defect	Random Survey (1961)	VHRC Survey of One District (1964)
Deck resurfaced	40	38
Moderate or heavy scaling	18	22
Cracking	33	40
Spalling	0	0

The general agreement between surveys of three different groups of bridges given in Tables 5 and 7 and the concurrence reflected in the reasonable progression of defects recorded from two separate surveys of the same group suggest that the observational techniques and method of selecting the sample size used in the PCA-BPR survey do result in essential agreement and are reliable and reproducible.

DISCUSSION OF VIRGINIA'S DECK PERFORMANCE

When compared with the performance of decks in the other states included in the random survey and with published reports from other areas, the performance of the decks included in the Virginia sample is comparatively good. It is true that 51 percent of the spans have been resurfaced within 20 years of their original construction, but there are indications that many of these resurfacings had been placed more as a matter of convenience than for correction of below-par performance. Others were placed with the hope of delaying potential problems. The rate of resurfacing has been decreasing in recent years.

Although Virginia is located in a temperate climate, the weathering conditions are in some respects severe, particularly with respect to cycles of freezing and thawing. The late adoption of air entrainment is reflected in the high incidence of scaling. For those bridges constructed with air-entrained concrete, the concrete did not contain amounts consistent with the amounts subsequently shown to be required for durability of structural concrete.

The absence of widespread spalling and serious cracking is fortunate, and, although reasons cannot be definitely stated, the use of $1^{11}/_{16}$-in. clear cover and a high proportion of simply supported spans are likely major contributors. Seventy-five percent of the uncovered spans in the original sample were simply supported.

As given in Table 6, the specification requirements for deck concrete have been progressively upgraded since about 1965. This upgrading has been concomitant with upgrading of construction techniques including the requirement of machine screeding and a general emphasis on the demands for diligence in inspection and controls. Intensive training and certification programs for producer and department personnel involved in concrete construction have also provided benefits. Although no quantitative data are available for assessing the improvement in deck performance, it is generally conceded that, despite the increased demands being placed on concrete in decks and the increased use of de-icing chemicals, the performance has improved since the survey was made. The initial PCA-BPR survey and this updating provide a basis, along with the VHRC's special study, from which to assess quantitatively from future surveys the benefits of improvements in materials and construction procedures and increased attention to the details of good practices.

CONCLUSIONS

Based on the observations and data developed, the conclusions given below appear warranted. It should be emphasized that the behavior reflected by the results is that obtained under specifications and techniques used during the period of construction (1940 to 1961) rather than those currently in use.

1. Listed in order of decreasing frequency, the defects observed during the 1970 resurvey were scaling, cracking, popouts, spalling, and rusting. This is the same order as reported for the initial survey in 1961.
2. The percentage of spans affected by each of the defects increased between surveys. Considering all defects except those in the lowest severity category of each classification, scaling increased from 13 to 31 percent, cracking from 3 to 13 percent, and spalling from 0 to 6 percent. Changes in the other defects were insignificant. These changes are believed to be reasonable in view of the environmental factors and the specification requirements, subsequently upgraded, that existed at the time of construction.
3. The types of cracking observed in 1970 in decreasing order of frequency were transverse, random, pattern, longitudinal, and diagonal. No "D" cracking has been found in either of the Virginia surveys or in the nationwide survey.

4. The order of frequency of defects from the sample in the random survey as well as from the smaller sample included in the VHRC study of finishing methods on bridges exposed for about the same period as those in the random survey was approximately the same, and the percentages of spans affected by each type of defect agreed closely. The percentages of spans showing the various types of cracking also agreed closely.

5. Because details concerning the construction of decks included in the random survey are not available, correlations of the materials and construction characteristics with performance is not possible. The close agreement between the performance of the decks in the random survey and those in the VHRC finishing study, for which detailed observations were made during construction, indicates that causative relations developed from that study can be extrapolated to the performance of the larger sample.

6. Agreement among the results from surveys conducted at different times and on different samples validates the survey methodology and sampling plan developed for the original survey.

7. Action taken in 1966 to upgrade specifications for bridge deck concrete (Table 6) included upgraded requirements for entrained air content and water-cement ratio, both of which should greatly reduce the incidence of the most prevalent defect observed in the survey, namely, scaling.

8. Although not documentable, the comparatively low frequency of surface spalling when compared with performance in the other states included in the survey is probably related to the provision of $1^{11}/_{16}$-in. clear cover reinforcement. Adequate cover, per se, is not a complete solution to the problem of surface spalling inasmuch as the quality of concrete is also important, but there is no evidence to support the belief that the concrete quality was significantly better in Virginia than in other states.

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APPENDIX

DEFINITIONS OF DETERIORATION TYPES

The following definitions and the random survey data sheet (Fig. 10) are those used in the PCA-BPR studies (1, 3, 5, 6, 13, 14).

Scaling

Light—loss of surface mortar up to ¼ in. in depth and exposure of surface of coarse aggregate.

Medium—loss of surface mortar ¼ to ½ in. in depth with some loss of mortar between coarse aggregate.

Heavy—loss of surface mortar and mortar surrounding aggregate particles ½ to 1 in. in depth such that aggregate is clearly exposed and stands out from the concrete.

Severe—loss of coarse aggregate particles as well as surface mortar and mortar surrounding aggregate, greater than 1 in. in depth.

Cracking

Transverse—Reasonably straight cracks perpendicular to centerline of roadway; vary in width, length, and spacing; frequently occur over primary slab reinforcement; sometimes extend completely through slab; may be visible before bridge is open to traffic or at some later date. (On some skewed bridges, the transverse slab steel is placed at an angle other than 90 deg to the roadway centerline; cracks parallel to this steel are also defined as transverse cracks.)

Longitudinal—fairly straight cracks, roughly parallel to centerline of roadway; vary in width, length, and spacing; sometimes extend completely through slab; may be visible before bridge is open to traffic or at some later date.

Diagonal—roughly parallel cracks forming an angle other than 90 deg with the centerline of the roadway, except as noted under transverse; usually shallow in depth; vary in width, length, and spacing; may be found immediately after completion of construction or may not appear until after bridge is open to traffic.

Pattern or map—interconnected cracks forming networks of any size and usually similar geometrically to those seen on dried mud flats; may vary in width from fine and barely visible to well defined and open; may develop early in the life of the concrete or at some later date.

"D"—usually defined by dark-colored deposits and generally located near joints and edges.

Random—cracks meandering irregularly on surface of slab, having no particular form, and not fitting other classifications.

Surface Spalling

Small—roughly circular or oval depression generally not more than 1 in. deep or more than about 6 in. in any dimension; caused by separation and removal of a portion of the surface concrete, revealing a roughly horizontal or slightly inclined fracture; generally a portion of the rim is vertical.

Large—may be a roughly circular or oval depression generally 1 in. or more in depth and 6 in. or more in any dimension; caused by separation and removal of a portion of the surface concrete, revealing a roughly horizontal or inclined fracture; generally a portion of rim is vertical (in some cases the spall may be an elongated depression over a reinforcing bar).

Hollow area—an area of concrete that, when struck with a hammer or steel rod, gives off a hollow sound, indicating the existence of a nearly horizontal fracture below the surface.

Other Defects

Joint spall—elongated depression along expansion, contraction, or construction joint.

Figure 10. Sample random survey data sheet.

STATE ILLINOIS COUNTY COOK HIGHWAY NOS I 294 BRIDGE NO. 72

YEAR BUILT 1958 DECK: UNCOVERED ✓ COVERED ___ TYPE OF COVER ___

IS DETAILED CONSTRUCTION DATA AVAILABLE ? YES

WHAT TYPE OF DECK REPAIR OR RECONSTRUCTION HAS BEEN DONE ? NONE

SPAN NO I HAS BEEN SELECTED AS THE N (S) E W END OF BRIDGE.

SPAN NO.		1	2	3	4	5	6	REMARKS
LENGTH, FT.		45	78	78	45			
SPAN TYPE		←— SS-IB-CN —→						
SCALING	1				3			
	2				X			
	3			15				
	4			X				
CRACKING	1	L	L	L	L			
	2		L					
	3							
	4			M	L			
	5							
	6			H				
RUSTING				R				
SURFACE SPALLS	1				2			
	2		1		3			
JOINT SPALLS	1		2					
	2			3				
	3							
POPOUTS		M	F	F	F			

COMMENTS: DECK SPECIFIED TO BE OF AIR-ENTRAINED CONCRETE

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Popouts—conical fragments that break out of the surface of concrete, leaving holes that may vary in size from that of a dime to as much as a foot in diameter (generally, a shattered aggregate particle will be found at the bottom of the hole, with part of the particle still adhering to the small end of the popout cone).

Pitting—loss of thin coats of surface mortar directly over coarse aggregate particles without apparent damage to the aggregate particles; pits usually not over $1/8$ in. in depth (this is in contrast to popouts).

Mudballs—small holes in the surface left by dissolution of clay balls or soft shale particles.

Other Definitions

Bleeding channels—essentially vertical, localized open channels caused by heavy bleeding.

Water-gain voids—voids formed during the bleeding period along the underside of aggregate particles or reinforcing steel; initially filled with bleeding water that subsequently evaporated or was absorbed.

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