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Corrosion, Concrete, Quality Control, and Paint Beads

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CONTENTS

FOREWORD

This RECORD contains 11 research papers that address topics in portland cement concrete, corrosion, quality control, and paint beads. They should be of interest to researchers and practicing engineers alike.

Spellman and Ford trace the historical development of curing materials and practices. They examined the use of a resin/varnish type and a chlorinated rubber type of compound for improving the durability of surface texture of pavements. New methods of evaluation of the compounds are described.

Lin, Walker, and Payne examined the effects of rapid freezing rates on the deterioration of concrete. They found an interaction between the degree of saturation and the freezing rate that affected the concrete durability.

Iddings et al. describe a field-testing method for measurement of the cement content of concrete using activation analysis with californium-252. The technique is rapid and can be used on samples large enough to be representative.

Dana and Peters describe a field test method to measure corrosion reaction rates of corrugated metal pipe. It was determined that metal pipes in low-resistivity soils exhibited a high corrosion rate. A linear polarization test method was developed and compared with conventional resistivity test methods. The use of bituminous coating was found to be effective in inhibiting pipe corrosion.

Noyce, Ostrowski, and Ritchie investigated 287 galvanized culverts through visual observation and electrical tests to define the degree of corrosiveness. They found that the factors contributing to culvert deterioration were dissimilar soil contacts, presence of organic soils, differentials in aeration and soil moisture, and the presence of sulfates and chlorides. Biological corrosion was observed in the form of sulfate-reducing bacteria.

Stratfull, Jurkovich, and Spellman report on the evaluation of corrosion investigation techniques. They found that 1 lb of chlorides per cubic yard of concrete at the level of the steel was associated with active corrosion and that an analysis to determine the amount greater than this may have no practical significance because the chloride content is already too great. They found there is active corrosion when the potential of the steel is greater than -0.35 volts CSE. Potential measurements made on 5 bridge decks where delaminations had been repaired showed a drop in corrosive potential of about one-half.

Willenbrock describes the experiences of the Pennsylvania Department of Transportation and Pennsylvania State University in the educational process to develop a program of implementing statistically oriented end-result specifications. Pennsylvania's future work plan indicates that by 1980 there will be full implementation of this technique.

Weed describes the process of developing a statistically based specification for the rolling straightedge used to measure surface irregularities. A quality level is selected using appropriate producer's and consumer's risk. Operating characteristic curves are presented that illustrate the applicability of the specification. The technique is discussed to aid those wishing to apply the technique to other measurement processes.

Benson and Ames examined the precision of standard tests such as sieve analysis, percentage of crushed particles, L.A. abrasion, sand equivalent, cleanness value, durability index, and R-value by conducting an interlaboratory correlation program. The amount of error between operators and between laboratories is described and causes are discussed. Recommendations for improving test precision are given.

DaForno evaluated the 2 most common types of floating beads, narrow and broad gradation. The narrow-gradation floating beads gave good reflectivity under dry conditions but showed poor reflectivity under light rainfall. Through the use of wet and dry night photographs, DaForno shows that the broad-gradation floating beads perform best under all conditions.

Girard, Murray, and Rucker tested cold-applied Missouri standard dispersion resin-varnish and chlorinated rubber-alkyd paints, high-heat paints, and floating beads to study the effectiveness of Missouri's traffic delineation system on both concrete and asphalt concrete pavements. They used a high-beam point-count night visibility rating method to evaluate the floating bead-paint system. Their investigation produced a delineation system that provides economic savings over the system previously used.

DEVELOPMENT OF CONCRETE CURING PRODUCTS AND PRACTICES

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The historical development of curing materials and practices in the California Division of Highways since 1967 is discussed. Greater demand on the performance of pavement texture because of earlier use, greater volumes of high-speed traffic, and increased emphasis on skid resistance created a need for better quality curing. The cost and effectiveness of compounds formulated play the major role in their adoption. Chlorinated rubber compounds, while more expensive than other types investigated, had some superior properties that justify their use on structures. Availability of raw materials and air pollution laws have required some adjustments. Laboratory tests used to compare performance are not very satisfactory. Alternatives to the standard mortar pan test method were explored, and a field test to measure spread rate was developed.

•THE beneficial effects of good curing are well known (1-6, 8, 9). Although compressive strength has often been a criterion for comparing curing methods, other important factors are involved. For example, craze cracking, plastic shrinkage, and full-depth transverse or map cracking are undesirable. These conditions may result from delays in application of curing procedures but do not necessarily reduce compressive strength to any significant degree. Some types of cure provide temperature control (sometimes by cooling and sometimes by heating) to minimize adverse effects. Surface abrasion resistance is a particularly important factor for floors and pavements, yet compressive strength tests of cores will not usually reflect surface conditions.

As the need for better surface textures increased, the whole operation of curing and texturing was critically examined to determine areas of possible improvement. Factors such as increased traffic, higher speeds (resulting in the need for greater initial and longer lasting skid resistance), earlier use of pavement after construction (and even during construction), and an awareness of lack of texture durability prompted a search for improvements in construction and in materials used to form the texture of the pavement. Upgrading of curing appeared to be a way to help overcome some of the problems caused by loss of surface texture.

Liquid types of curing compounds for concrete pavement have been used for a long time, perhaps born from the desire to get away from the messy, more complicated and expensive "water cure" recognized by early concrete technologists as necessary for best results. While liquid-type curing membranes are not as effective as wet-type curing, they are in many cases adequate, much lower in cost to use, and more convenient. Maintaining wet mats is both messy and inconvenient. Water supplies must be developed and inspection maintained during the whole curing period. Removal and replacement of mats for sawing operations on pavements require a considerable amount of extra labor. One significant plus factor, however, is the cooling effect of a wet cure, which is important on bridge decks. A small outdoor "laboratory-type" test program confirmed the relative effectiveness of the more common types of curing

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used. Curing temperatures and compressive strengths were compared for concrete cured with a liquid compound, a reinforced plastic sheet, and wet mats; the results are given in Table 1.

Curing studies have been numerous and it is difficult to say when the idea of a liquid seal first came about. In the late 1950s asphaltic emulsions were in common use, and the ASTM committee concerned was busy developing a performance specification for "nonbituminous liquid compounds for curing concrete" (2). The 1944 specification for liquid materials was developed for bituminous-type products, and other types could not successfully be tested under this specification. In his report to the ASTM committee, Proudley stated, "The method of test is of special importance and should be simple, so that almost any laboratory can perform it satisfactorily, and comprehensive, so that it will simulate a practical range of field conditions, and finally, the test should be reproducible so that a well-equipped laboratory with reasonable care can agree with the findings of other acceptable laboratories and can repeat the tests on the same material ... with only a minor tolerance for test errors."

Although some of these goals have been partially achieved after more than 20 years of effort, it seems fair to conclude, based on recent correspondence of the present ASTM subcommittee, that there is yet a lot to do. Development of a generally acceptable test has been painfully slow.

Until about 1968, the so-called wax or wax/resin type of curing compound, pigmented to give it ability to reflect heat and thereby lower concrete curing temperature, was most often used for curing pavements. Considering its cost and effectiveness, it was regarded as adequate. As the demand for deeper and more durable texture increased, it became evident that some upgrading of the product would be desirable. Typical properties that were considered deficiencies were the tendency of the wax to crystallize during temperature changes, high pigment settling rates, the fact that the cured "film" did not impart any strength or toughness to the surface at early ages (tracked), and viscosity characteristics that allowed it to sag or run off the peaks of the deeper textures that resulted from switching from burlap drags to brooming.

An early concern developed about use of new and different curing compounds and increased rates of application. The increased application rate and tougher films suggested a reduction in early skid resistance that could create a slick pavement. In one case, the pavement, after receiving an application of a new chlorinated rubber product, appeared to be slick because of its sheen and light reflectance. Skid tests made on one project approximately 2 weeks after application showed that initial skid resistance was indeed reduced but, because a heavier, deeper texture (brooming) had been used, the initial values did not create any skid hazard. If a heavy application of the new material were made to a pavement having initially a borderline skid resistance, it could conceivably cause a skid hazard. The newer materials have now been in use for several years, with no problem with skid resistance reported. The new curing compounds may be expected to prolong better skid resistance because of the toughness of the compounds and better cures. Laboratory abrasion testing indicated that the new materials could

Table 1. Comparison of curing methods, temperatures, and strength.

Type of Cure	14-Day, Percent Relative Core Strength	Maximum Temperature[a] During First 24 Hours (deg F)
Wet mats (7 days, then air-cured)	100	103
No cure (allowed to air-dry)	56	98
Chlorinated rubber type of membrane	77	116
Reinforced plastic sheet (7 days, then air-cured)	75	110

[a]½ in. below surface; maximum air temperature on first day was 98 F, and ambient temperature during curing period ranged from 58 F to 105 F.

be expected to reduce surface wear. Field testing for skid resistance over a 7-year period, however, showed no significant differences among curing compounds for prolonging the period of good skid resistance.

Basically, curing compounds specified were required to meet the performance specifications defined in AASHTO Specification M-148. A review of tests on samples from jobs showed a large variation in performance. No doubt some of the wide variations were the result of test method deficiencies, inasmuch as a number of highway departments had expressed dissatisfaction with the test. Thus, as part of a broader plan, work was initiated to find ways of improving the test procedure. New approaches were explored and the existing test method was modified in an attempt to improve it also. As testing was moved out to the field, it was found that, even there, problems developed because of inability to measure spread rates accurately. Various procedures were developed in an attempt to improve field measurements.

CURING COMPOUNDS

A search for alternatives to the wax/resin type of compound led to two general types: a resin/varnish type and a chlorinated rubber type. A commercial chlorinated rubber compound was being marketed for curing concrete to improve durability of surface textures, in recognition of a national problem. The commercial product conformed basically to Federal Specification TT-C-00800. Its relatively high cost, however, limited its use to structures where, because of physical conditions, a better grade material could be justified. The chlorinated rubber product had good moisture-retention properties and, equally important to bridge construction, had good drying and scuff-resistance qualities, which made it particularly desirable for deck and box girder construction. Typically there is much construction activity on decks and inside box girders as soon as the concrete is hard enough to walk on without damage.

By specification, damaged sealant must be repaired immediately. The chlorinated rubber type of seal provided a fairly tough membrane that could take a reasonable amount of "traffic" without damage. Another factor that justifies higher curing sealant cost for structures is the fact that almost all of the curing a structure will get during its lifetime is the formal curing it gets the first few days after construction. Because some parts of a bridge are up in the air instead of in contact with the ground, the concrete not only tends to dry out more but also is deprived of some additional moisture that could be supplied at ground levels.

By applying coatings technology acquired through paint formulation activities, modified chlorinated rubber compounds were developed that not only retained the desirable physical properties but also were lower in cost than commercially available products. Even so, the cost of this type of curing compound is still relatively high, and it is not generally used for curing pavements. Various formulations of chlorinated rubber compounds were developed to meet other specific needs. For example, a somewhat thixotropic version was developed for use on median barriers and other vertical surfaces. Both clear and gray formulations were developed for use where a white was not wanted for aesthetic reasons. As air pollution rules came into effect, the solvent systems had to be altered to comply.

The solvency and relative evaporation rates of solvents had to be considered since those properties affect the continuity of film, pigment suspension, and sag and flow characteristics of the curing compounds. Faster evaporating solvents were required for formulations designed for use on vertical surfaces.

Curing compounds with a petroleum hydrocarbon resin base, being lower in cost, were developed for use primarily on pavements. Although they may not have properties equal to the chlorinated rubber compounds, they are adequate for the purpose. Generally speaking, a good curing seal that lasts at least 2 or 3 weeks is probably all that is necessary because pavements, being in contact with the ground and subject to wetting during rainy periods, continue to cure indefinitely.

In addition to being tough enough to resist some tire traffic from joint saws and the profilograph, the compounds had to be able to remain on the ridges left by broom texturing without running down into the low areas.

Several formulations were devised and lab-tested. Alternatives to the resin type were also investigated. For example, a compound using a limed tall oil base was developed that equaled the resin type. Although a specification for this product was used as an alternate to the resin type, it was never made commercially, possibly because of some significant differential in manufacturing cost. An acrylic-base product also was formulated. Literally dozens of formulations were made and tested, with each succeeding one that was adopted having slightly better properties than the former. The number discarded, however, is formidable.

One difficulty in formulating products such as curing compounds, whether supplied by composition or performance specifications, is the changes that can occur in raw materials available. Although earlier resins supplied were fairly color-stable, later supplies turned quite yellow after a few days of exposure to the sun. Of course, the specifications can include restrictions on yellowing, but a few "colored" pavements resulted in some adverse comments before being corrected.

Later, materials shortages required many other changes in composition. Formulations involving solvent substitutions required extensive testing because solvent release is a major factor in the formation of impermeable films.

Many combinations of titanium dioxide with various extender pigments and antisettling agents have been tested in an effort to conserve limited supplies of TiO_2 while retaining good reflectance and pigment suspension.

Choice of additives is not indicated in our specifications because the selection is dependent on process variables such as pigment dispersal equipment, which differs among curing compound manufacturers. This policy has occasionally caused problems. As an example, after a new specification was issued, a factory sample from one of our curing compound suppliers failed to meet requirements for water retention. The sample complied with all other chemical and physical requirements. Upon investigation, we learned that bentone had been used as an antisettling agent. That addition produced a porous film through which an excessive amount of water vapor was transmitted.

Table 2 gives the curing compound formulations developed at the Transportation Laboratory that are included in the 1975 California Standard Specifications.

Lab Testing and Evaluation

Dissatisfaction with the ASTM and AASHTO test methods for measuring moisture loss has been voiced by nearly everyone having to rely on them for product control. Despite the many refinements made over the years, reproducibility is poor and comparison of results between laboratories is most difficult. Particularly troublesome has been the effect of time of application of the compound to the portland cement mortar, the effect of surface texture, and the difficulty in securing a seal between the mortar and the pan forms. All this could be avoided, however, if the general principle of the test were changed; i.e., the compounds should be tested on mortar that is representative of conditions existing in actual use. This argument, of course, has great merit. It would, for example, show up products that might react adversely with the highly alkaline mortar. It would also, if texturing is properly carried out, measure the ability of the compound to remain and protect the higher points and not run down into the valleys.

One test condition related to equipment requirements is the 32 ± 2 percent relative humidity (at 100 ± 2 F) specified in AASHTO T-155. Temperature can be readily controlled, but relative humidity is another matter and, to our knowledge, no equipment is commercially available at reasonable cost that can consistently comply with these requirements. Various studies have been made to isolate the relative effects of test variables, and about the only conclusion that can be made is that there are a lot of them. However, as with some other tests, a specific operator can get a "feel" for the test and, by careful control, use it successfully. By replicate testing and experience, for example, a defect such as failure of the seal between the mortar and the pan will be easily recognized. Operator skill in applying the membrane must be at a high level because the amount of sealant sprayed and the method of spraying some materials are critical.

To overcome some of the difficulties and cost of conventional AASHTO testing, other means of evaluation were explored. For example, liquid membranes were applied to filter paper, which in turn was used to cap a jar containing water. The loss of water through the coated paper was determined by weighing the jars, which were stored in an oven at 137 F. Unfortunately, the test results did not always correlate with moisture losses determined by the AASHTO method. This method, nevertheless, has better repeatability and may be useful in comparing curing products. Some typical test results are given in Table 3.

Other paper or filter-type base materials are still being considered, however, and means of obtaining a uniformly reproducible film are being explored.

Another approach investigated was the drawing down of a film of predetermined thickness on a transparent base. After drying, the films were examined with a microscope. Although the method was not considered suitable for a control test, we did find that wax-base or wax/resin material usually failed the moisture retention test if the dried film appeared sandy or gritty.

Other measures of compound efficiency explored were flexural strength of coated concrete specimens and abrasion resistance. These tests, while a measure of what the curing compound does, are difficult and time-consuming to perform and are, therefore, considered unsuitable for routine acceptance testing.

Field Testing

Field testing is generally limited to sampling and determining rate of application. Usually the application rate was checked by counting the number of barrels of compound used over some measured distance and calculating the coverage in square feet per gallon (as specified). This procedure, of course, does not tell us anything about uniformity of coverage, nor does it reflect the amount of compound lost in the wind or overspray. Despite the fact that spray rigs are required to have "shields", there are often conditions under which shields are not effective. Some protect the spray only if the wind is from the front or rear. Side winds can carry much of the compound away. To improve the accuracy of measuring, a procedure was developed to measure the compound at different points on the pavement surface. Absorbent pads were made that were preweighed, then placed at various points in front of the spray rig. Immediately after the sprayer passed, the pads were folded (wet sides together) to prevent loss of weight through evaporation, and then reweighed. Knowing the area of the pad and the gain in weight, coverage could be readily calculated. Some typical results are given in Table 4. These data show some rather nonuniform application rates on actual jobs and point up the inadequacy of a specification calling for a single application rate.

Later, the test method (California 535) called for "Pampers" as the absorbing medium. The method required field-weighing of the pads, which were recovered immediately after application of the compound and sealed in small plastic bags.

Although the pad-weighing method was shown to be operational, an even simpler test procedure was desired. The use of a wet film thickness gauge used in paint inspection was developed. For this procedure, wet films are collected on a metal or glass base and a specially made gauge is used to measure the thickness (Figure 1). Rigid paint-can lids were substituted for the thicker metal and glass plates, which eliminated cleaning of plates.

Some measurements on the dried film were also made. Theoretically, if the percentage of solids of the compound is known, the dried film thickness should be directly related to wet film thickness. In the case of one field test, the relationship was not good because the compound was not adequately stirred before use and the actual solids content was low compared to what was specified. Since this was measurable, the test might be used as a gross check on the solvent/solids ratio.

Some skill is required to handle the gauge because slight tipping or failure to hold the base flat to prevent flow will cause erroneous results. A magnet fastened to the

Table 2. California curing compound formulations.

Specification Number	Nature of Compound	Intended Uses
742-80-71	Pigmented petroleum hydrocarbon resin	General use, chiefly pavements
741-80-100	Pigmented chlorinated rubber base	Bridge decks and horizontal surfaces where abrasion resistance during construction is required
741-80-101	White or gray pigmented chlorinated rubber base	Vertical surfaces, median barriers where nonsagging pigmented finish is desired
722-80-102	Clear chlorinated rubber base	Colored concrete, exposed aggregate concrete, or where natural color is to be retained

Table 3. Comparison of test results from mortar and water vapor transmission methods.

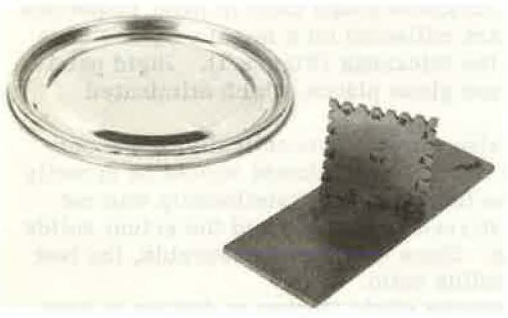
	Water Loss at 24 Hours (grams)	
Sample No.	AASHTO Mortar Method	Water Vapor Transmission Through Filter Paper
TK-26, gray	10	5.2
TK-26-1	6	3.4
70-006	41	15.2
70-007	21	12.2

Table 4. Curing compound spread-rate determination.

	Spread Rate (ft²/gal)	
Test	Determined by Volume Method	Deposited on Test Pads
Laboratory	132	129
	132	130
	150	145
	150	152
	200	195
	200	220
	100	96
Laboratory, 8 × 18-ft slab	200[a]	160
		200
		185
		195
		210
Field, 24-ft pavement, first day	123[b]	310
		310
		242
		165
		235
		235
		225
Field, 24-ft pavement, second day	128	165
	98	170
	134	195
	120	175
	—	165

[a]Average for whole slab. [b]Average for day's paving.

Figure 1. Equipment used for wet film thickness determination.



end of a pole proved to be a simple yet effective means of collecting the plates when placed more than an arm's length in from the edge of the pavement. Such placement is, of course, necessary to check transverse spread rates.

CONCLUSIONS

1. Curing compounds used to cure pavements and structures have been formulated to have better overall properties (i.e., moisture retention, viscosity, sag, and uniformity) than those available in the past. Chlorinated rubber curing compounds were designed for use on decks and other surfaces where scuff resistance is important and for use on vertical surfaces where sagging is detrimental. Resin-type compounds of lower cost were developed for pavements where long-lasting curing seals are not necessary.

2. A moisture vapor permeability test using filter paper as a substrate could be used in lieu of AASHTO T-155 for comparing water retention characteristics of curing compounds. The AASHTO method should be used as a referee procedure until an acceptance limit can be established for each type of material tested.

3. Field tests for determining spread rates of curing compounds at the time of application have been developed. The use of these spread rate methods is effective in encouraging contractors to apply uniform coatings of adequate thickness.

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The authors wish to acknowledge the work performed by the many engineers and technicians who took part in this study over the years. Since this report includes results from various substudies, the list of participants is large; however, our special thanks go to the late Herbert Rooney for his guidance in formulation, to Bill Neal, Carl Sundquist, Lee Wilson, Ben Squires, John Boss, Gary Mann, and Phil Young for their assistance in performing lab and field tests, and to Faye Penrose for her patient assistance in preparation of the report.

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EFFECTS OF COOLING RATES ON THE DURABILITY OF CONCRETE

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Past research generally assumed that rapid cooling rates cause faster deterioration of concrete that is susceptible to damage from freezing and thawing. The objective of this project was to investigate the effect of varying freezing rates on an otherwise standard ASTM test. Eighty-one concrete specimens were fabricated with an aggregate capable of causing deterioration under freezing-and-thawing conditions. The aggregates were placed in the concrete at 3 different degrees of saturation. Three rates of cooling were used: 4.4 F/hour (2.45 C/hour); 6.6 F/hour (3.67 C/hour); and 13.3 F/hour (7.39 C/hour). Modifications to freezing and thawing equipment are described, and possible explanations of the results obtained are presented. If the aggregate was not initially saturated when placed in the concrete, slower freezing rates produced demonstrably faster rates of deterioration. It is theorized that a slower rate of cooling enables more water to migrate to the surroundings of the coarse aggregate. Therefore, during the thawing phase, more water is available for the coarse aggregate to become increasingly saturated. Rate of cooling seemed not to affect the rate of deterioration of concrete containing aggregates placed in the concrete already in the saturated state.

•AS one of the most abundant construction materials in the world, concrete by the thousands of tons is used to build a variety of structures exposed to natural weathering forces. Methods of testing the resistance of concrete materials to these weathering forces have been studied since at least the late 1800s. In the temperate zones, alternate freezing and thawing are considered among the most destructive of the natural weathering conditions.

Many organizations have worked with various methods of evaluating concrete in the laboratory for the purpose of predicting durability in the field, especially since the 1940s. ASTM, beginning in 1952 and 1953, originally described 4 procedures (essentially, slow and fast procedures for freezing in water and thawing in water and slow and fast procedures for freezing in air and thawing in water—C290-52T, C291-52T, C292-52T, and C310-53T). In a cooperative program involving 13 laboratories, an attempt was made to compare these tests (1). Aside from showing that the freezing-in-water tests caused concrete to deteriorate more quickly than those that freeze in air, the tests demonstrated that the various laboratories had difficulty in obtaining consistent results. All methods provided means of distinguishing between known good concrete and bad, with the rapid freezing-in-water procedure being the quickest and most consistent. The slow freezing-in-water tests gave essentially the same results but required more time. In general, both the slow and fast tests use about the same rate of cooling; the fast test has more cycles in a 24-hour period.

ASTM currently describes 3 procedures, 2 of which are in C666-73, "Standard Method of Test for Resistance of Concrete to Rapid Freezing and Thawing" and rep-

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resent a combination of the old C290 and C291, the fast freezing-in-water and fast freezing-in-air procedures. A third method, C671-72T, "Tentative Method of Test for Critical Dilation of Concrete Specimens Subjected to Freezing", evolved from suggestions made by Powers (2, p. 1150) and later developed by the California Department of Highways (3) and Larson and Cady (4). This test restricts the rate of cooling to 5 ± 1 F (2.775 ± 0.556 C). This latter test, however, is not in common use. Tests using the fastest rates of cooling are similar to that used by the Corps of Engineers (2, p. 1148), which has cooling rates up to 36 F/hour (20 C/hour).

Except for C671, criticism of freezing and thawing procedures in general has often centered around the fact that the rates of cooling used in the laboratory tests are much greater than those found in the field, thus reducing the possibility of good correlation between laboratory results and field performance. T. F. Willis (5, p. 1140) summarized this view by stating:

> The fastest rate of cooling to which pavements in this country are subjected under service conditions is 6 F (3.336 C) per hour. If rate of cooling is a factor in the deterioration of concrete exposed to frost action, the acceleration achieved by these test procedures is somewhat analogous to accelerating a test of a glass shelf, intended as a support for a light flower pot, by hitting it with a sledge hammer.

This paper compares laboratory test results of an air-entrained concrete (containing an aggregate generally recognized as "poor" and placed in the concrete at various levels of saturation) exposed to different rates of cooling in alternate cycles of freezing and thawing.

RATE OF COOLING

Properly made air-entrained concrete containing "good" aggregate will withstand many alternate cycles (up to 1,000 in some tests of laboratory freezing and thawing) without exhibiting significant deterioration. An exception to this is concrete tested by procedures where extremely rapid freezing is obtained by circulating brine at -20 F (-28.9 C) around a specimen in a rubber boot filled with water. However, most laboratory procedures use 2 or more hours to lower the temperature of the specimen from 40 F (4.4 C) to 0 F (-17.8 C), with cooling rates between 6 F (3.33 C) and 20 F (11.11 C) per hour.

It is thought that the more rapid the cooling rate, the more the specimen suffers thermal shock, which possibly masks the effects of aggregate susceptibility. As indicated by Powers' hypothesis (6), the faster the cooling, the greater the velocity of movement of unfrozen water and the greater the stresses induced. In addition, the rate of concrete contraction is more rapid than the water expansion. If the specimen is in a dry condition, the rate of cooling should have little effect on durability. If a specimen is saturated, a maximum stress condition will occur.

When thawing water is circulated, a second shock occurs. The rapid rate of thawing used with most laboratory methods [0 F (-17.8 C) to 40 F (4.4 C) in ½ to 1 hour in most tests] will cause rapid movement of unfrozen water, but the concrete will be expanding and the water contracting, thus eliminating some of the stress encountered in rapid cooling (6).

Because the rate of thawing is less likely to be significant, and because of the limitations of equipment available for this study, only the rate of cooling is considered here.

OBJECTIVE

The objective of this project was to extend knowledge of concrete durability by investigating the effect of different freezing rates. To achieve this objective, a nondurable aggregate was subjected to 1 of 3 moisture conditions: (a) air-dried; (b) 24-hour water soaking; and (c) 24-hour vacuum saturation at 1.5 cm mercury. The aggregates were incorporated in concrete specimens 3 × 3 × 16 in. (7.62 × 7.62 × 40.64 cm), fabricated with a carefully controlled air content, and subjected to alternate cycles of freezing and thawing between 40 F (4.4 C) and 0 F (-17.8 C). Three average rates of cooling were used: 4.4 F/hour (2.45 C/hour), 6.6 F/hour (3.67 C/hour), and 13.3 F/hour (7.39 C/hour).

SCOPE AND DESCRIPTION OF WORK

Eighty-one specimens were used to investigate the effects of various rates of cooling on the frost resistance of concrete made with "poor" aggregates placed in the concrete at different levels of saturation. For each of the 3 rates of cooling, 9 specimens were made with aggregates saturated at the 3 different levels of moisture. For each mix design, 3 specimens were made on different dates. All concrete was air-entrained, with air contents averaging 6 percent. The concrete was designed for a slump of 3 in. (7.62 cm) and a compressive strength of 3,500 psi (24 132 kPa).

Procedures

The 81 specimens were exposed to alternate cycles of freezing and thawing after 13 days of curing in lime-saturated water. ASTM C 666-71 was followed for fast freezing and thawing in water, using the Logan apparatus developed by Cordon (7). The specimens were measured for length, weight, and dynamic modulus before beginning the test and every 6 to 10 cycles thereafter. Also, length-change measurements were made at intervals of approximately 2 F (1.112 C) during the initial freezing and thawing cycle. Testing was continued until the specimen had lost 40 percent of its original dynamic modulus or had undergone 100 freezing cycles.

Materials

The "poor" aggregate was the float material (at specific gravity of 2.55) from a heavy media plant in the midwest using a glacial gravel source. Although heterogeneous from a mineralogical standpoint, the amount of weathered chert was such that a 100-cycle durability factor of about 5 was obtained when the aggregate was used in vacuum-saturated condition. It is from the same material source described in full as aggregate H in NCHRP Report 12 (8).

A single-source type III cement was used throughout the study, and Vinsol resin added at the mixer was used to control the air content.

A local fine aggregate of crushed limestone sand was used in all specimens. The sand has a fineness modulus of 2.75, an absorption of 1.07 percent, and a bulk specific gravity of 2.60.

Equipment Modification

The freeze-thaw equipment in the Virginia Tech concrete laboratory was not designed to be regulated for various rates of cooling. To vary rates of cooling for this study, a micro-adjusting valve was installed in the system, as shown in Figure 1. When a slow rate of cooling was desired, the valve was regulated to a small opening, allowing a small amount of Freon liquid to be pumped into the evaporator. On the other hand, if a rapid rate of freezing was used, the valve was opened wider, allowing a larger amount of Freon liquid to be pumped into the evaporator.

RESULTS AND DISCUSSION

Complete results are given in Tables 1, 2, and 3, with the averages shown representing the values discussed. These tables also serve to demonstrate the kind of variability that existed in the test results as well as to provide information difficult to present in graphical form. Mix V refers to specimens made with vacuum-saturated aggregate, mix S to specimens with aggregate soaked for 24 hours, and mix A to specimens made with aggregate that was air-dry (except for a brief wetting period) when put into the concrete.

Statistical Analysis

Simple statistical analysis (analysis of variance) substantiates what is apparent merely from viewing the data. For mix V (aggregates vacuum-saturated), the analyses of the test results affirm the hypothesis that the concretes, when subjected to alternate cycles of freezing and thawing, have the same durability under various rates of cooling. Concerning concretes made with soaked and air-dried aggregates, the analyses reject this hypothesis; thus, it is concluded that these concretes, when subjected to alternate cycles of freezing and thawing, are affected by the cooling rate. Figure 2 shows this conclusion.

Significance and Hypothesis of Failure

At first glance, the surprising result is that slower cooling rates apparently produce loss of strength in the concrete more quickly than the fastest rate, especially where the aggregate is not vacuum-saturated. Actually, the concrete containing the vacuum-saturated aggregate was so low in durability that no difference was displayed.

Considering that the literature implies that faster cooling rates should produce more rapid destruction, the results from the other 2 mixes are indeed interesting. It is easy to surmise that the time needed for aggregate in the S and A mixes to become critically saturated is an important part of the answer to the question of why the slower rate of cooling results in lower concrete durability.

Time required for critical saturation of an aggregate is discussed by Verbeck and Landgren (9), who point out that, after a significant drying period, some aggregates may need only a few weeks of wetting to become critically saturated. In these tests, all specimens are placed in water for 2 weeks before placing into test, a test that keeps the specimens wet. The results indicate, however, that even after this test period the aggregates in the S and A mixes are not critically saturated. Since failure occurs at a maximum of about 60 cycles, only 10 to 14 more days are required for failure and presumed critical saturation. It would appear that the freezing process greatly accelerates the saturation process.

If freezing accelerates the saturation process, what are the mechanisms involved? The Verbeck and Landgren paper (9) and the works by Powers (2, 6) discuss many aspects of this. It is not believed that the limited data in this paper refute statements in those works. On the other hand, those articles do not present explanations that are completely applicable to the test conditions to which the S and A mixes in this study were exposed.

It has been recalled to one of the authors that Stanton Walker (then Director of Engineering at the National Ready-Mixed Concrete Association) argued on the floor of some forum that the fact that the slow freezing of turkeys is detrimental to the quality of the meat had nothing whatever to do with the freezing and thawing of concrete. We are inclined to agree with that statement but not with the implication that fast freezing is more detrimental than slow with regard to concrete.

Nerenst (10) in 1960 indicated that the boundary layer between aggregate and paste is a locus for conditions favorable for the formation of ice until a certain degree of hydration of the surrounding paste is reached. He also hypothesized that, as paste with high amounts of water surrounds coarse aggregate with low porosity, the heat

Figure 1. Freezing and thawing system.

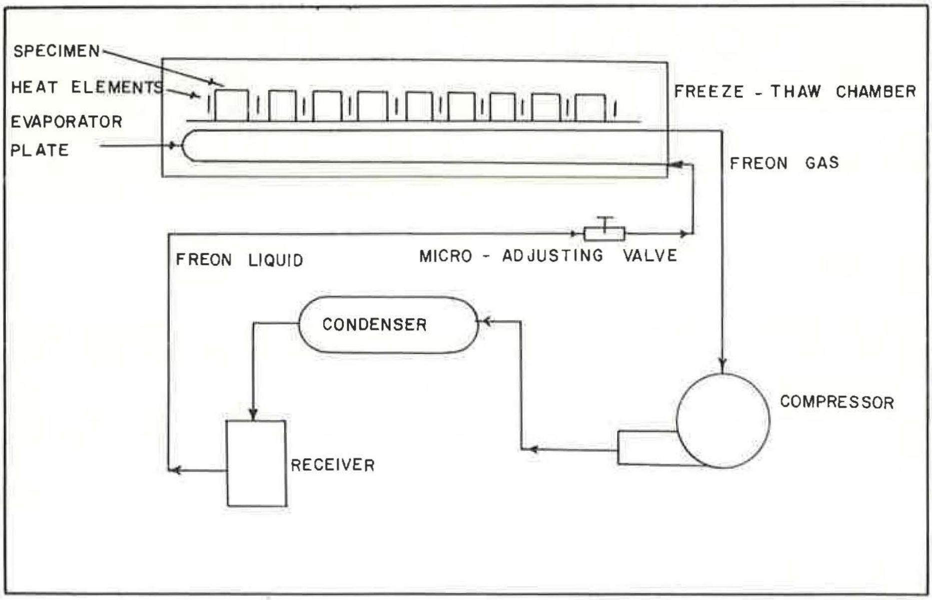


Table 1. Cooling rates versus concrete durability of mix V.

		Relative Dynamic Modulus (percent)								
		90		80		70		60		
Cooling Rate	No.	C	L	C	L	C	L	C	L	DF100
F	1	1.74	17.4	3.48	35.0	5.19	62.0	8.0	89.0	4.80
	2	3.18	26.0	4.92	27.0	6.39	34.0	7.5	45.0	4.50
	3	1.37	19.0	2.74	39.0	4.57	54.0	5.5	72.0	3.30
	4	1.35	26.0	2.70	36.0	3.35	49.0	4.0	55.0	2.40
	5	1.51	8.0	3.02	13.0	4.00	17.0	5.0	21.0	3.00
	6	1.00	17.0	2.00	33.0	3.00	42.0	4.0	50.0	2.40
	7	1.40	13.0	2.80	25.0	6.00	59.0	8.1	86.0	4.86
	8	1.86	19.0	3.72	33.0	6.00	53.0	7.5	76.0	4.50
	9	1.27	21.0	2.54	42.0	3.81	55.0	5.0	74.0	3.00
	Average	1.63	15.6	3.10	31.4	4.70	47.2	6.1	63.1	3.64
M	1	3.30	24.0	5.00	31.0	6.46	40.5	8.0	49.0	4.80
	2	1.56	13.0	4.67	34.0	5.80	47.0	7.0	60.0	4.20
	3	2.16	23.3	4.16	40.0	5.24	44.8	6.3	50.0	3.80
	4	3.58	29.2	6.00	44.0	7.20	56.0	8.2	68.0	4.90
	5	2.38	17.8	4.53	32.8	6.18	42.0	7.5	52.0	4.50
	6	2.35	21.2	4.54	40.0	6.36	55.0	8.0	68.0	4.80
	7	1.33	14.8	2.66	29.6	4.00	46.0	5.0	68.0	3.00
	8	1.10	17.0	2.00	34.0	3.10	42.0	4.0	50.0	2.40
	9	0.98	12.0	1.92	20.5	2.84	35.3	4.2	51.0	2.50
	Average	2.10	19.1	3.94	34.0	5.20	45.4	6.5	57.3	3.88
S	1	1.62	9.7	3.24	19.4	4.86	29.2	6.3	42.0	3.80
	2	1.41	16.1	2.82	32.2	4.23	48.2	5.9	70.0	3.50
	3	1.37	12.3	2.74	24.6	4.11	37.0	5.7	50.0	3.40
	4	1.69	16.9	3.38	33.8	5.07	50.7	6.5	64.0	3.40
	5	1.90	15.2	3.80	30.4	5.70	45.6	8.0	68.0	4.80
	6	1.62	16.2	3.24	32.4	4.86	49.0	6.5	64.0	3.90
	7	1.11	28.8	2.22	57.6	3.33	85.6	4.5	117.0	2.70
	8	0.02	10.1	1.84	20.2	2.76	30.3	4.0	50.0	2.40
	9	1.35	13.6	2.70	32.2	4.05	48.4	5.5	60.0	3.30
	Average	1.44	15.4	2.89	31.4	4.33	47.1	5.9	65.0	3.52

Note: C = cycle, L = cumulative length change. F = 13.3 F/hour, M = 6.6 F/hour, S = 4.4 F/hour (1 F/hour = 0.556 C/hour).

Table 2. Cooling rates versus concrete durability of mix S.

Cooling Rate	No.	Relative Dynamic Modulus (percent) 90 C	90 L	80 C	80 L	70 C	70 L	60 C	60 L	DF100
F	1	11.4	14.5	23.6	30.0	32.0	38.0	44.0	61.0	26.4
	2	11.1	27.0	27.0	44.0	35.3	58.4	44.0	69.0	26.4
	3	11.2	26.0	22.0	21.0	32.0	27.0	44.0	42.0	26.4
	4	6.0	22.0	9.5	39.0	12.5	53.0	16.0	76.0	9.6
	5	7.0	26.0	12.5	31.0	18.2	44.0	33.0	68.0	19.8
	6	10.1	24.0	20.5	29.0	39.0	58.0	50.0	86.0	30.0
	7	7.5	19.0	13.1	36.0	21.0	64.0	29.0	91.0	17.4
	8	5.0	13.0	14.6	35.5	26.0	58.0	32.0	79.0	19.2
	9	12.5	22.0	27.6	50.0	33.0	62.0	45.0	93.0	27.0
	Average	9.1	21.5	18.9	35.1	27.7	51.4	37.4	73.9	22.5
M	1	7.0	16.0	16.0	27.0	21.0	39.0	25.0	49.0	15.0
	2	5.0	20.0	13.0	30.0	24.8	49.0	26.0	55.0	15.6
	3	9.0	13.0	24.0	40.0	28.4	43.0	34.0	68.0	20.4
	4	5.5	12.0	13.0	17.0	17.0	28.0	20.0	45.0	12.2
	5	6.0	15.0	25.4	25.0	30.0	33.0	44.0	60.0	26.4
	6	7.7	29.0	26.0	47.0	29.0	54.0	34.0	66.0	20.4
	7	9.6	16.0	15.5	22.0	22.0	35.0	33.0	58.0	19.8
	8	8.2	18.4	12.0	28.3	21.2	38.3	29.0	54.6	17.4
	9	6.0	16.2	13.0	31.1	24.3	43.4	32.7	60.0	19.6
	Average	7.1	17.3	17.5	29.7	24.6	40.9	30.8	57.3	18.5
S	1	6.0	16.0	11.5	33.0	14.5	46.0	17.0	58.0	10.2
	2	6.0	11.0	10.0	20.0	14.0	42.0	18.5	65.0	11.1
	3	10.0	23.0	20.1	34.0	24.0	40.0	27.0	50.0	16.2
	4	3.5	16.0	7.0	31.0	11.0	48.0	14.5	64.0	8.7
	5	8.6	13.5	16.0	21.0	21.0	37.0	23.6	45.0	14.2
	6	8.6	11.5	15.5	20.0	21.0	25.0	25.0	34.0	15.0
	7	4.0	19.0	8.0	33.0	11.0	40.0	14.0	50.0	8.4
	8	4.5	14.0	9.0	26.0	12.0	36.0	16.0	45.0	9.6
	9	7.8	10.0	16.0	24.0	21.0	34.0	24.0	43.0	14.4
	Average	6.6	14.9	12.6	26.9	16.6	38.7	20.0	50.4	12.0

Note: C = cycle, L = cumulative length change. F = 13.3 F/hour, M = 6.6 F/hour, S = 4.4 F/hour (1 F/hour = 0.556 C/hour).

Table 3. Cooling rates versus concrete durability of mix A.

Cooling Rate	No.	Relative Dynamic Modulus (percent) 90 C	90 L	80 C	80 L	70 C	70 L	60 C	60 L	DF100
F	1	5.0	13.0	10.0	26.0	26.0	39.0	47.0	67.0	28.2
	2	11.0	11.5	20.0	15.0	33.5	28.0	46.0	46.0	27.6
	3	10.0	21.0	43.4	35.0	58.0	38.0	72.0	40.0	43.2
	4	31.0	17.0	55.0	34.0	66.0	47.0	79.0	57.0	57.4
	5	32.0	17.0	55.0	52.0	67.0	78.0	79.0	127.0	57.4
	6	24.0	29.0	39.0	39.0	59.0	72.0	67.0	88.0	40.2
	7	8.0	10.0	25.3	25.3	42.0	44.0	56.0	58.0	33.6
	8	10.0	9.0	42.0	42.0	54.5	28.0	62.0	29.0	37.2
	9	8.2	18.0	23.2	23.2	35.4	49.0	50.0	90.0	30.0
	Average	15.5	17.1	34.8	30.6	49.0	47.0	62.0	66.8	39.4
M	1	13.0	7.6	16.0	19.0	32.7	51.0	48.0	83.0	28.8
	2	6.7	10.8	24.8	31.6	36.0	48.0	40.0	50.0	24.0
	3	14.0	8.5	29.5	23.0	49.0	37.0	57.0	47.0	34.2
	4	15.0	18.0	30.0	36.0	44.0	51.0	54.0	60.0	32.4
	5	16.0	11.7	28.0	19.0	36.0	26.0	52.0	48.0	31.2
	6	8.9	18.8	17.8	37.6	27.0	51.0	42.0	70.0	25.2
	7	8.1	11.1	29.0	37.0	41.0	55.0	50.0	85.0	30.0
	8	34.0	6.0	50.0	21.0	62.0	30.0	76.0	40.0	45.6
	9	22.0	6.0	37.6	21.0	42.8	34.0	54.0	48.0	32.4
	Average	15.3	10.9	29.2	27.2	41.2	42.6	52.6	59.0	31.5
S	1	15.1	16.0	28.7	23.0	44.0	42.0	46.5	45.0	27.9
	2	25.0	16.0	43.0	42.0	50.0	67.0	60.0	76.0	36.0
	3	12.0	5.0	22.0	5.0	28.0	8.0	8.0	11.0	20.7
	4	9.0	16.0	22.0	40.0	30.0	51.0	37.0	65.0	22.2
	5	8.4	6.0	18.7	32.0	30.0	42.0	37.5	60.0	22.5
	6	24.0	17.0	38.0	40.0	46.0	60.0	50.0	80.0	30.0
	7	7.0	23.0	17.0	32.0	27.0	51.0	35.0	66.0	21.0
	8	11.5	21.0	32.0	38.0	42.3	52.0	50.0	70.0	30.0
	9	26.0	25.0	45.8	48.0	52.0	53.0	55.0	55.0	33.0
	Average	15.3	16.1	29.7	33.3	38.8	47.3	45.1	57.5	27.0

Note: C = cycle, L = cumulative length change. F = 13.3 F/hour, M = 6.6 F/hour, S = 4.4 F/hour (1 F/hour = 0.556 C/hour).

Figure 2. Durability factors versus cooling rates for mix V, mix S, and mix A.

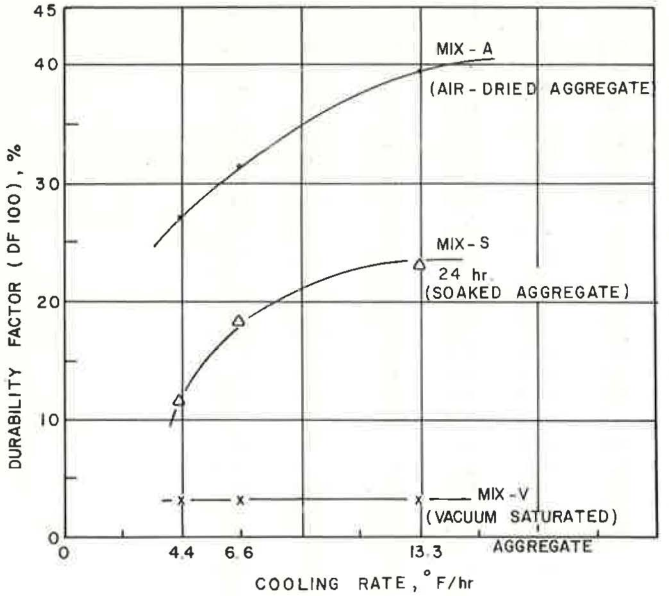
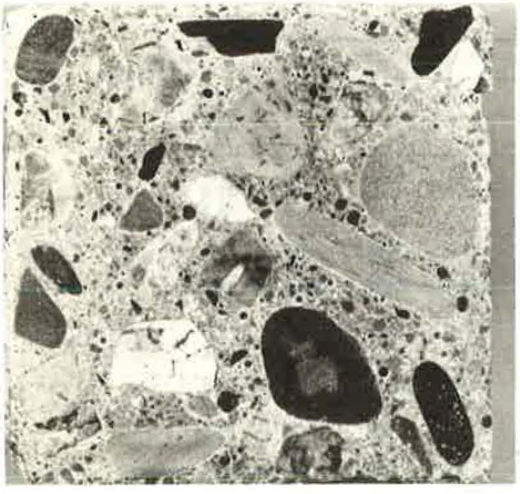


Figure 3. Section of concrete specimen after reaching 60 percent relative dynamic modulus of its original value.



released by the fusion of ice may delay the penetration of frost in the paste in comparison with the penetration of freezing temperatures into the aggregate. Hence, coarse aggregate may serve as centers of low temperature that extract water from the unfrozen paste in all directions.

Experiments by Khakimov (11) relating to soils demonstrated that during frost action the part played by water migration toward the freezing front decreases with increasing rate of cooling. It appears that during the freezing of concrete a slower rate of cooling allows more water to migrate to the surroundings of the coarse aggregate; therefore, during the thawing phase, more water is available with which the coarse aggregate may become increasingly saturated. Put another way, the faster the rate of cooling, the less water that moves to the boundary between aggregate and paste; thus it takes the aggregate much longer to obtain a critical saturation, resulting in greater concrete durability.

Figure 3 shows a section of the concrete specimen that lost 40 percent of its original dynamic modulus.

SUMMARY

From the foregoing discussion, the following statements might be made:

1. For concretes made with vacuum-saturated aggregates that are extremely poor in frost resistance, the slow rate of cooling tends to result in durability factors that are about equal to the low durability factors produced by the fast rate of cooling.
2. For concretes made with soaked or air-dried aggregates, the faster rate of cooling tends to result in greater concrete durability.
3. The slower the cooling rate, the more quickly the aggregates become critically saturated.

ACKNOWLEDGMENTS

This paper is drawn from a portion of the work performed by Chung-Hsing Lin while pursuing the PhD degree at Virginia Polytechnic Institute and State University. Research was done by Dr. Lin under the general supervision of Richard D. Walker and W. W. Payne. Dr. Walker was mainly responsible for the preparation of the manuscript for this particular article. Robert D. Krebs assisted with editorial review. The research was supported by the Civil Engineering Department of Virginia Polytechnic Institute and State University.

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DISCUSSION

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The authors have made an interesting study of a fundamental parameter in freeze-thaw durability of concrete. The importance of rate of cooling has been emphasized in most of the reported research, and yet little experimental data have been presented to demonstrate the role of this factor quantitatively.

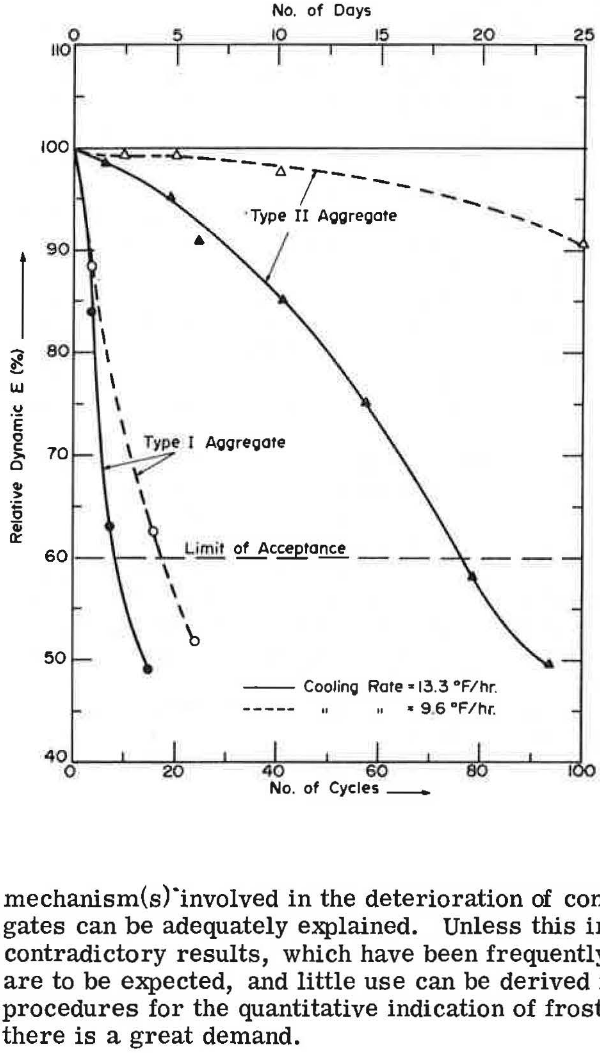
Although the limited data presented by the authors preclude a generalized conclusion, it has been shown that under a certain testing condition with a particular type of aggregate, contrary to the generally accepted view, slower rates of cooling could be more damaging to concrete than fast rates.

To demonstrate the importance of the testing conditions and the influence of various characteristics of aggregates, the results of tests performed by the writer are shown in Figure 4. In these tests aggregates were soaked in water for 24 hours prior to mixing (corresponding to mix S of the authors), and 24 hours after casting the concrete the specimens were placed in lime-saturated water for 13 days prior to commencement of the freeze-thaw tests. The freeze-thaw test procedure was a modified version of the ASTM C 666-71 test. The freeze-thaw cycle consisted of lowering the specimen temperature in humid air from 40 F to the minimum cooling temperature (0 F in fast cooling and 11 F in slow cooling) in 3 hours. The rates of cooling thus produced were 13.3 F/hour or 9.6 F/hour respectively. The limitation of the apparatus was such that the length of the cooling period could not be altered. At the end of the cooling period the specimen temperature was raised from the minimum temperature to 40 F gradually in 1 hour by means of humid air, and then water at a temperature of 40 F was sprayed on the specimen for 2 hours. In this manner the thermal shock usually experienced by the concrete at the end of the cooling period was avoided. Two types of aggregates were used. Type 1, which had a poor service record, was a mixture of sandstone and shale with a specific gravity of 2.46 and absorption value of 3.52 percent. Type 2 aggregate was a weathered flint gravel with the corresponding values of 2.54 and 2.05 percent respectively.

Figure 4 shows that increasing the rate of cooling from 9.6 F/hour to 13.3 F/hour reduced the durability factor from 10.8 to 4.8 for type 1 aggregate and from 92 to 46.8 for type 2 aggregate.

In conclusion, these results clearly illustrate the point that there are other significant factors, besides the rate of cooling, that affect the freeze-thaw behavior of aggregates in concrete. In one situation these factors could add up and produce a satisfactory performance for an aggregate, and in another situation they could cancel each other and result in an entirely unsatisfactory performance. The disparity in results reported by the authors and those mentioned here is most likely due to the empirical nature of the testing techniques used. Further fundamental research is needed to determine the influence of different parameters, individually and collectively, before the

Figure 4. Effect of rate of freezing on the freeze-thaw performance of air-entrained concrete with different types of aggregates.



mechanism(s) involved in the deterioration of concrete due to frost-susceptible aggregates can be adequately explained. Unless this important step is taken, these types of contradictory results, which have been frequently reported during the past 30 years, are to be expected, and little use can be derived from these results or these testing procedures for the quantitative indication of frost-susceptible aggregates, for which there is a great demand.

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AUTHORS' CLOSURE

The authors sincerely thank Famili for his pertinent and timely discussion of such an important phenomenon in the freeze-thaw durability of concrete.

In a study by Pence (13) there are recorded data indicating that for some aggregate particles there is a contribution to the freeze-thaw damage of concrete at temperatures below 11 F. Additional recorded, but unpublished, experimental data concerning this phenomenon are available from the same research. To make a valid comparison of different rates of cooling, it is believed that upper and lower limits of the cooling cycles should be the same for all rates involved.

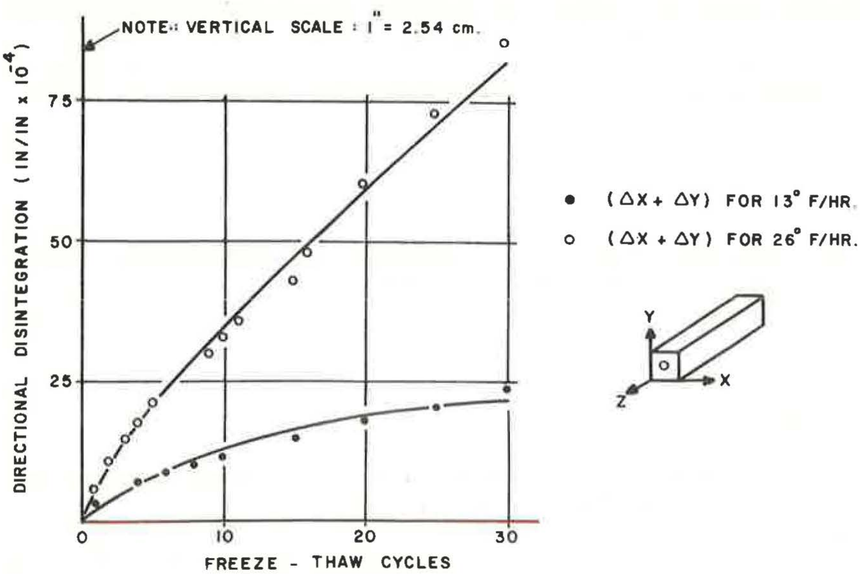
In the authors' research, the concrete specimens were cycled in water from 40 F to 0 F and back to 40 F. In the discusser's research, the concrete specimens were cycled in humid air and then sprayed with water (at 40 F) for 2 hours at the end of the heating cycle. This difference produced an endless supply of water for the concrete in one case and a limited supply of water in the other case. This additional available source of water in the authors' work was forced into the concrete by the freezing and thawing action. In the discusser's work, it is conjectured that the phenomenon involved was mainly the movement of water from one point to another, with a limited source of water. These entirely different experimental techniques probably produced different results and may represent an area of further research.

After the original research paper was submitted to the Transportation Research Board, the results on research regarding the effects of compressive stress fields on the deep-seated (aggregate-generated) type of concrete deterioration (freeze-thaw damage) were obtained. Graphs from the results of that project illustrate the effects of the rate of cooling on these aggregates in concrete when vacuum-saturated and cooled at 2 different rates (14, 15).

Approximately the same materials and concrete mixture were used in this project as described in the main paper. The same automatic freezer was used for both projects. These members (beams) were 3 × 3 × 14 in. A concentric compressive stress of 1,000 psi was applied in the longitudinal (14-in.) direction of all members. The heat-transfer medium was water-saturated kerosene, and the members were cycled in a closed system, i.e., cooled and heated in water-saturated kerosene. The temperature in the concrete was cycled from 40 F to 0 F and back to 40 F. Cooling rates were 13 F/hour and 26 F/hour, and the heating rates were the same for both conditions. Deterioration was measured by permanent strain or change in length.

Because the members were restrained from elongating in the Z-direction (Figure 5), the permanent change in length was measured in the X- and Y-directions and called "directional disintegration". In Figure 5, the symbol Δ was used to indicate a change

Figure 5. Directional disintegration for 2 cooling rates versus freeze-thaw cycles.



in length in any direction. The numerical value (in inches per inch) for each data point was obtained from 3 beams and was the average value over 36 inches.

The results are shown in Figure 5. At the end of 30 cycles, the directional disintegration at the faster cooling rate (26 F/hour) was approximately 350 percent greater than the directional disintegration at the cooling rate of 13 F/hour. Also, the slope of the curves delineates the rate of the disintegration of the concrete.

There is the possibility that the increase in the rate of deterioration of this concrete could be affected by 2 temperature differentials during the cooling cycles. The freezing front (freezing line) moves from the surface inward; consequently, the faster the cooling rate, the greater the temperature differential in the concrete. This could result in detrimental tensile stresses at the surface. If the temperature differential of the 26 F/hour cooling rate were a contributing variable, the microcracks and the macrocracks would have first appeared on the surface. Physical observation and experimental measurements indicated that the microcracks propagated from the inside to the surface, and not the reverse.

These results are in agreement with Powers' hypothesis (6) in regard to the effect of the rate of cooling on critically saturated particles or aggregates. In this case, the hydraulic pressure generated is a function of the rate of cooling.

The paper illustrates the effects of the rate of cooling on the movement (transportation) of water through the concrete and the rate at which the particles become critically saturated. There appears to be no conflict between the results of the 2 experiments.

From the results in the paper, Figure 2, there is an indication that the transverse fundamental frequency method (ASTM C 215-60) may not be the most sensitive method of measuring disintegration of concrete due to these particles when they are critically saturated.

It is possible that for this type of concrete damage a slow cooling rate produces a more rapid movement of water into the concrete and that, once the point of critical saturation is reached, then the higher cooling rate creates the greater damage. This seems to be true in laboratory research and could indicate a field physical phenomenon.

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DETERMINATION OF CEMENT IN CONCRETE BY ACTIVATION ANALYSIS WITH CALIFORNIUM-252

Frank A. Iddings, Ara Arman, Calvin E. Pepper, Winton G. Aubert, and John R. Landry, Louisiana State University

Results from neutron activation analysis of in-place and plastic concrete samples are presented. Data were obtained by equipment suitable for and operated under field conditions. The system described for determination of cement content of in-place concrete includes a 35-microgram Cf-252 source, portable activation/shield assembly with remote operating cable, and commercially available detector and electronics. An analysis of in-place concrete is accomplished in 22 minutes. Results for plastic concrete were obtained with a system designed for soil-cement mixtures. Using a 140-microgram Cf-252 source, an analysis could be completed in 9 minutes with an accuracy of ±5 percent of the amount of cement for normal cement contents. A system for analysis of samples of plastic concrete, cores, and soil-cement is described that can be moved to field sites in a trailer. Most existing methods for determination of cement content of concrete suffer because they are too slow, use too small a sample to be representative, and must be done in a laboratory. The only other field measurement technique being studied utilizes low-energy photon scatter. This technique uses only a thin layer of the available sample and fails to achieve necessary accuracy when aggregate varies in size distribution or heavy element content. Neutron activation analysis offers a rapid, simple, field-operational procedure for measurement of cement content. Besides these advantages, activation analysis allows the use of large, representative samples and offers considerable freedom from interferences.

•EXPOSURE of a sample of concrete to neutrons produces measurable quantities of a number of radioactive isotopes. By controlling the neutron energy spectrum, time of activation (neutron bombardment), decay time, measurement time, and instrument settings, certain of the radioactive products representing cement content can be emphasized. For a fixed set of conditions, radioactivity and composition are directly related. The strict relationship between radioactivity at the end of neutron activation and composition is

$$A = \frac{(\text{weight of element})\,(6.02 \times 10^{23})\,abc\,\{1 - \exp[-0.693\,(t/T)]\}}{\text{atomic weight of element}}$$

where

A = activity in disintegrations per second;
a = abundance of the reacting isotope of the element;
b = bombarding neutron flux in neutrons per square centimeter per second;

Publication of this paper sponsored by Committee on Instrumentation Principles and Applications.

c = cross section (or probability of reaction) in square centimeters;
t = activation time; and
T = half-life of radioisotope produced.

This relationship is simplified for fixed experimental parameters such as sample size and geometry as well as those mentioned above:

$$\text{Counts} = K\ (\text{percent cement})$$

where Counts = disintegrations measured by instrumentation and K = a constant for the fixed conditions selected. Such a relationship permits formation of a graph relating counts and cement content. Standard samples treated exactly like samples for the fixed experimental parameters generate the graph for a specific set of components.

EXPERIMENTAL PARAMETERS

Table 1 gives the radioactive materials produced in appreciable quantities by short-duration neutron bombardment. Of these observed radionuclides, ^{49}Ca represents cement content better than any other. In areas using siliceous aggregate and sand, the ^{49}Ca is indicative of only the cement content. By instrumental discrimination against gamma energies below 2.5 MeV, only ^{49}Ca and ^{24}Na produce counts. By using short neutron bombardment, decay, and counting times, ^{49}Ca activity greatly exceeds ^{24}Na activity. With large neutron sources, the typical analysis schedule includes a 5-minute neutron bombardment or activation, 1-minute decay for transfer of the sample to counting instrumentation, and a 5-minute counting or measurement period. For small neutron sources or small samples, the schedule may lengthen to 10 : 1 : 10 minutes for activation : decay : count.

The counting follows the procedures and instrumentation established for soil-cement mixtures (2, 3, 4). A 12.7 × 12.7-cm NaI(Tl) crystal detects the gamma radiation. Thermal insulation, shock mounting, and neutron shielding protect the crystal for field use. The large crystal gives the sensitivity necessary for detection of the 3.08-MeV gamma radiation from the small quantity of ^{49}Ca produced. Smaller crystals can be used, but they require substantially larger neutron sources along with longer activation and counting times.

The associated electronic instrumentation consists of a tube base with high-voltage divider network fitted to the detector and connected to a single-channel-analyzer scaler system by a single coaxial cable. The scaler system provides high voltage for the detector operation and permits selection of the gamma energies to be included in the measurement. The discriminator of the single-channel analyzer rejects gamma energies below those of the Ca-49. A convenient prepackaged scaler system is the Eberline Instrument Co. model MS-1. With a little electronic modification to obtain better temperature stability, the MS-1 system operates adequately for laboratory and field use since it can accept either 110 VAC or 12 VDC (auto battery) power. The model MS-1 includes solid-state electronics, large LED display, and oscillator timing system in a compact, lightweight package.

The most compact, high-output, and constant-yield neutron source available for activation of the concrete samples in the field is Cf-252. The Cf-252 decays by alpha emission and spontaneous fission. The fission produces a broad energy spectrum of neutrons. This broad energy spectrum of neutrons permits deep penetration into the sample. Figure 1 shows the thickness of a sample producing useful information on cement content. Figure 2 shows the increase in analytical signal with sample area at constant thickness. These data mean that the analytical information comes from several kilograms of sample material. Such a sample has a good chance of being representative of the bulk of material.

The inner construction of the activation/shield assembly for analysis of in-place

Table 1. Radioactive isotopes produced in concrete by short-duration neutron bombardment.

Radionuclide	Half-life	Gamma Energy (MeV)
^{28}Al	2.3 minutes	1.78
^{49}Ca	8.8 minutes	3.09, 4.05
^{42}K	12.4 hours	1.52
^{27}Mg	10.0 minutes	0.84, 1.02
^{56}Mn	2.58 hours	0.84, 1.81, 2.13
^{24}Na	14.8 hours	1.37, 2.75

Figure 1. Thickness of a concrete sample that produces useful information on cement content.

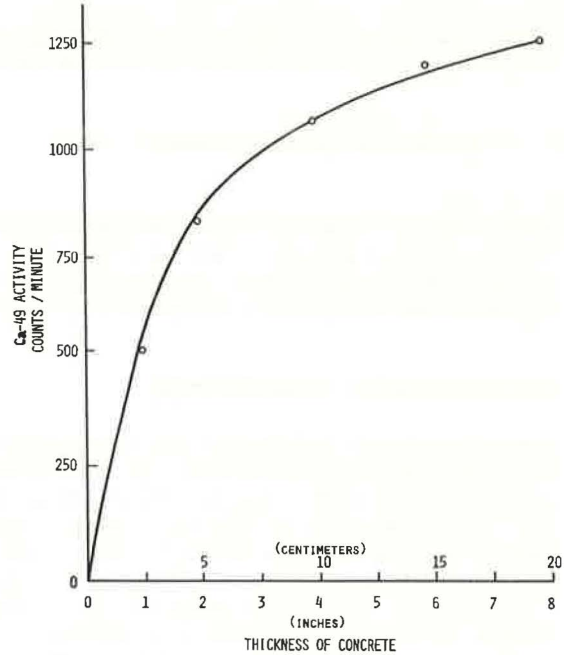


Figure 2. Increase in cement content signal with sample size for concrete slabs 12.5 cm thick.

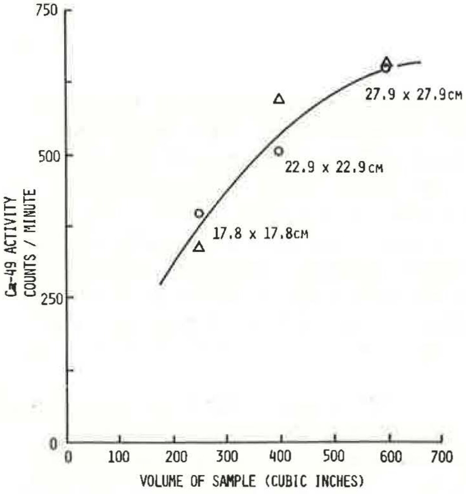


Figure 3. Cross-sectional diagram of activation/shield assembly for analysis of in-place concrete. (WEP = 60 percent water and 40 percent resin.)

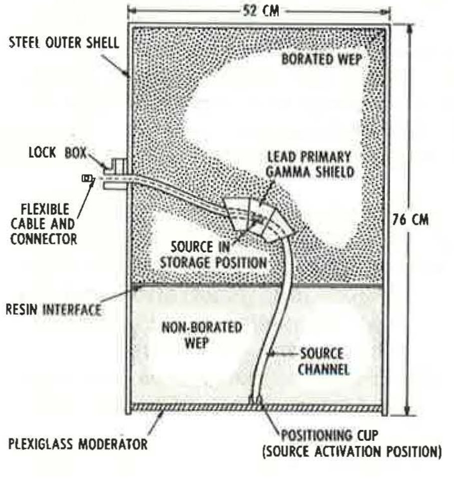
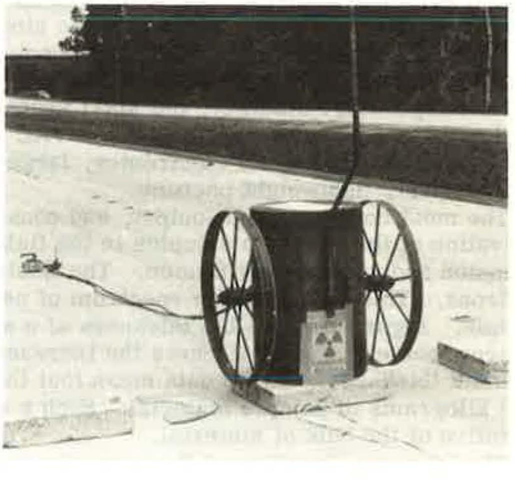


Figure 4. Activation/shield assembly in activation position on a concrete calibration slab; remote crank-out and cable are visible behind the assembly.



concrete is shown in Figure 3. This assembly rolls to the site of the analysis and stands on end for activation of the surface of the concrete. The activation with neutrons begins when the Cf-252 source (about 35 μg) moves from storage position to a point 1.25 in. (3.2 cm) from the end of the assembly. The source moves when the operator turns a crank attached to the source by a flexible cable (such as that used in isotope radiography). The crank and cable arrangement removes the operator to a safe, low-radiation exposure position remote from the activation position. Figure 4 shows the activation/shield assembly in an activation position on top of a calibration slab of concrete. Calibration slabs are 20 × 20 × 5 in. (56 × 56 × 12.5 cm) in size. The remote crank-out and cable are visible behind the assembly. Figure 5 shows the construction of the detector system containing the 3 × 3-in. (12.7 × 12.7-cm) NaI(Tℓ) crystal.

The source size, 35 μg of Cf-252, arose from the need to have an easily portable system combined with a need for rapid analysis. The present system can be moved and operated by 1 man, although a 2-man crew is desirable. Any larger source of Cf-252 would require a shield too heavy for easy use.

The activation/shield assembly for plastic concrete must house a source large enough for rapid results. A compromise between speed and source size (cost and shielding) resulted in the assembly shown in Figure 6. This assembly holds a 150-μg Cf-252 source. As noted, this assembly also uses a moving source. The source moves to an activation position below the sample on a wheel when the operator turns a crank on the side of the shield. Samples sit on the top of the unit inside a series of "donuts" to accommodate samples ranging from 2-liter cylindrical cardboard cartons to those contained in large polyethylene buckets. These larger samples are over 20 cm in diameter and are about as large as can be conveniently handled. Although this assembly can be moved on rollers, its portability will be confined to a small trailer.

With proper "donut" adapters, the activation/shield assembly can also be used for activation of soil-cement samples and standard core samples. Adapters must be used to keep radiation intensity at the operator position at a safe level. The detector system for all these samples is the same as that used on soil-cement samples, as shown in Figure 7.

RESULTS

Operation of the in-place concrete analysis system in the lab using carefully prepared standard slabs produced the results shown in Figure 8. The slabs cover the range from 4 to 7 bags of cement per cubic yard of concrete. Their physical size is 20 × 20 × 5 in. (56 × 56 × about 12.5 cm), with some variation in thickness. Designations of A, B, and C groups of samples mean separately mixed batches of each composition. Corrections for variation in thickness (see Figure 1) are applied to the results. Similar results were obtained in the field on a new section of Interstate 10 in which both the calibration slabs and highway surface were examined.

Results on plastic concrete include only laboratory measurements. Figure 2 includes data taken on plastic concrete samples of varying size. Figure 9 shows the results obtained on cylindrical samples using a 140-μg Cf-252 source in the soil-cement activation/shield assembly (Figure 10). The standard deviation (1 σ or 68 percent confidence) for each set of 5 different samples represents a variation of less than 5 percent of the amount of cement measured, i.e., 10.0 ± 0.5 percent cement. The same samples (sealed in polyethylene bags) activated using a 29-μg Cf-252 source gave the results in Figure 11. Even with an increase in activation time, precision and sensitivity are lost using the smaller neutron source.

CONCLUSIONS

For siliceous aggregate and sand samples of concrete, cement content is rapidly and accurately measured by neutron activation analysis. Both in-place and plastic samples

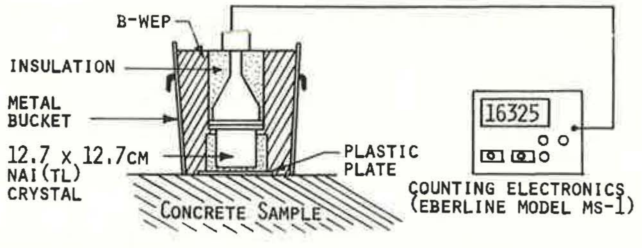


Figure 5. Diagram of detector system used for determination of cement content of in-place concrete.

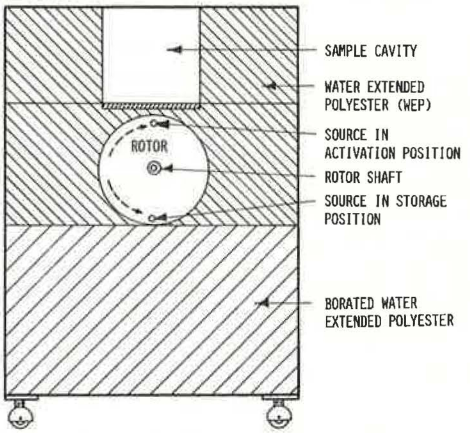


Figure 6. Activation/shield assembly for use with plastic concrete samples.

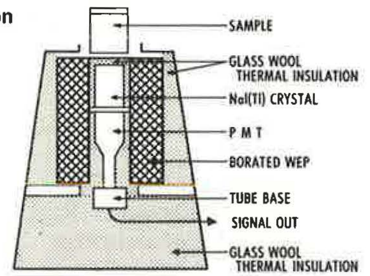


Figure 7. Detector system for determination of cement content in plastic cement, soil-cement mixtures, and standard core samples.

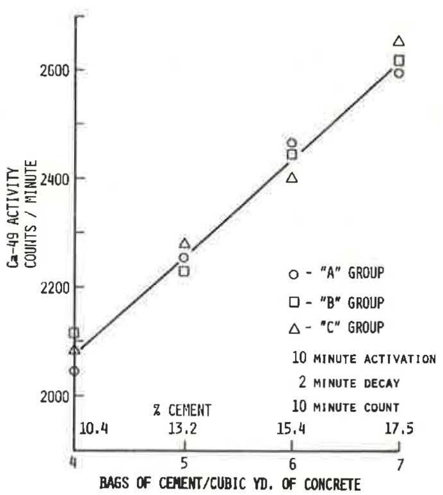


Figure 8. Laboratory analysis of in-place concrete slabs.

Figure 9. Determination of cement content in plastic concrete samples using soil-cement field system.

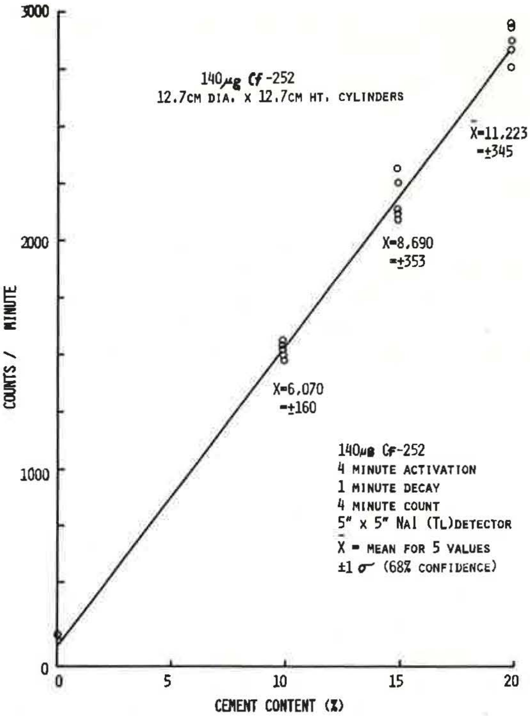


Figure 10. Cross-sectional diagram of field activation/shield assembly for soil-cement samples.

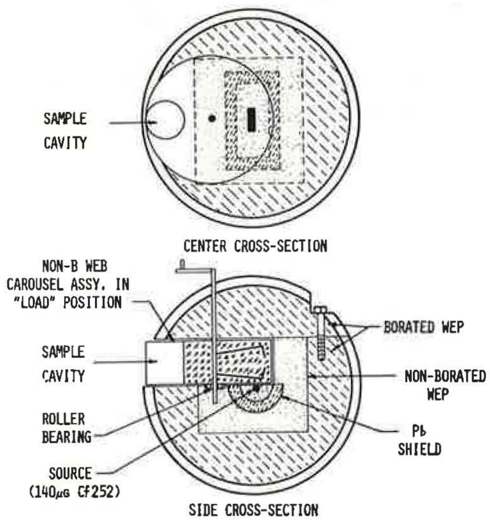
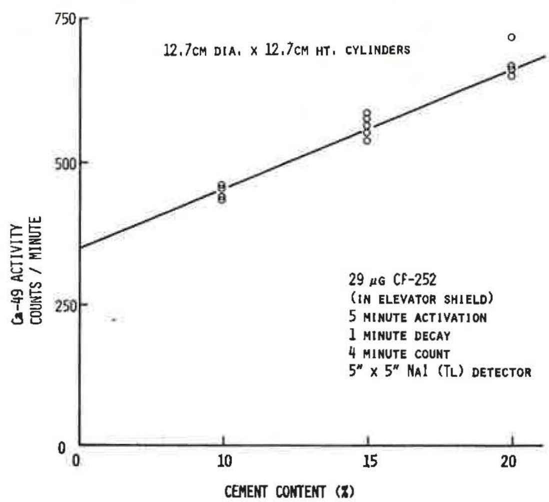


Figure 11. Determination of cement content in concrete samples using a small Cf-252 source in a laboratory activation system.



can be analyzed in field as well as in laboratory environments. Commercially available electronic systems and sources are adequate for use. Activation/shield assemblies must be fabricated by the user or specialty companies since they are not yet commercially available.

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Department of Highways or Federal Highway Administration.

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CORROSION OF HIGHWAY STRUCTURES

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A research study was undertaken with the objectives of evaluating existing corrosion design procedures for corrugated metal pipe and determining if the development of new corrosion test methods for service life designs was indicated and feasible. As a result of the research, a field test method for corrugated metal pipe installations was developed to measure corrosion reaction rates. By utilizing the new test method to measure corrosion rates, it was determined that metal pipes in high-resistivity soils (rho greater than 3000 ohm-cm) had low corrosion reaction rates. The rate of corrosion in medium-resistivity soils (rho above 2100 ohm-cm) was high initially but decreased greatly after a few years. However, in low-resistivity soils (rho less than 1000 ohm-cm) the corrosion reaction continued at a high rate. Also, a linear polarization test method was developed and compared with the conventional resistivity test method with satisfactory results. The study included corrosion protection of corroded steel pilings utilizing magnesium anodes. Research showed that corroded corrugated metal pipes could not be protected as easily because of the large surface area and soil resistance. The use of bituminous-coated corrugated metal pipe was found to be effective for inhibiting corrosion in most cases tested in Arizona.

•A STUDY was undertaken to define the problems of corrosion and its effects on corrugated metal pipe. The primary objective of the research was to evaluate present corrosion design methods and alternatives. In this paper corrosion is defined as the electrochemical reaction between buried metals and soils. The corrosion research study included the following topics:

1. Field corrosion test method (tests on buried metal structures by a voltage measurement method);
2. Laboratory test methods (testing of corrosive soil materials for pH and electrical resistivity and testing of corrosive soil materials by electrical polarization techniques); and
3. Corrosion protection methods (protection of buried highway structures by magnesium anodes and protection of corrugated metal pipe by bituminous coatings).

DEVELOPMENT OF A FIELD CORROSION TESTING METHOD

It was found that reactions between corrosive areas on corrugated metal pipe could be recorded by measuring voltage gradients existing along the length of the buried metal structure. The voltage gradients developed on the surface of the pipe were measured with a high-impedance voltmeter and a constant-voltage reference cell. The most widely used corrosion test cell for this purpose is the copper sulfate reference cell.

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By using the reference cell, it was possible to measure voltages on corrugated metal pipe at various points to study corrosion. The reference cell has a constant internal voltage, which allows voltage changes existing on the metal surface to be measured.

When a voltage difference is measured between 2 areas on a buried metal structure, active corrosion has been detected with a relative positive and negative region. The relative positive region is anodic and the relative negative region is cathodic. When a large voltage gradient is detected, it is a direct indicator of corrosion. It produces high corrosion current and accelerated deterioration of the metal.

EQUIPMENT AND PROCEDURE FOR FIELD CORROSION VOLTAGE TEST

The test procedure for measuring corrosion on corrugated metal pipe requires only a few minutes to perform. The negative lead from a high-impedance voltmeter is attached to the metal pipe, and the positive lead is connected to the reference cell. A series of readings are taken as the reference cell is moved at approximately 1-m intervals across the road surface and along the length of the pipe (Figure 1). After the voltage data are taken along the length of the pipe, the arithmetic mean of the voltage data and the statistical variance about the mean are calculated to determine the corrosion rate. Field testing showed that a rapidly corroding pipe usually has a variance of data from 5×10^{-3} to 10×10^{-3}. It was determined that the variance was a representative measurement of corrosion activity (Figure 2).

Buried metal pipes were tested in many locations and conditions throughout Arizona using the new test. After testing a pipe, it was uncovered and the visual appearance was then compared with the voltage test data. The voltage variance test correlated well with visible corrosion on the surface. Several of the culverts exhibited extreme corrosion, and in some cases perforation of the culvert was observed. The voltage variance was highest on extremely corroded pipes and lowest on uncorroded pipes. On uncorroded pipes, the voltage change was 0 to 50 millivolts, and the variance was below 2×10^{-3}. On extremely corroded metal pipes, the voltage change was between 0 and 300 millivolts, and the variance was above 5×10^{-3}. The results indicated that there was a good correlation between predicted corrosion as determined by the voltage survey method and visual inspection of the corrosion. After the test was fully developed to accurately measure underground corrosion reactions, it provided a good comparison for the evaluation of laboratory corrosion tests.

LABORATORY TEST METHODS

Resistivity and pH Test Methods

For electrochemical corrosion to occur, the electrolyte that is in contact with the anode and cathode surface must conduct electricity. The rate of corrosion caused by an electrolyte depends on a number of factors. Two important factors affecting the corrosion rate of corrugated metal pipe are pH and resistivity. The pH is a measure of acidity or alkalinity of the electrolyte. A neutral pH is the least corrosive, while an excessive concentration of either acidity or alkalinity accelerates the rate of corrosion. Moist soil that is in contact with buried corrugated metal pipe acts as an electrolyte characterized by a particular pH and resistivity that directly affect corrosion reactions.

The resistivity (rho) of a substance determines the amount of electrical current that passes through a given volume of material. Resistivity is a natural characteristic of a material, independent of dimensions. It is not the resistance (R) but is related by the equation, R = rho × L/A (ohms). For example, a copper wire has a constant resistivity, but the resistance of the wire increases as it is made longer or as the cross-sectional area is made smaller.

By rewriting the equation for resistivity we have rho = R × A/L. In the equation the

Figure 1. Corrosion voltage testing equipment.

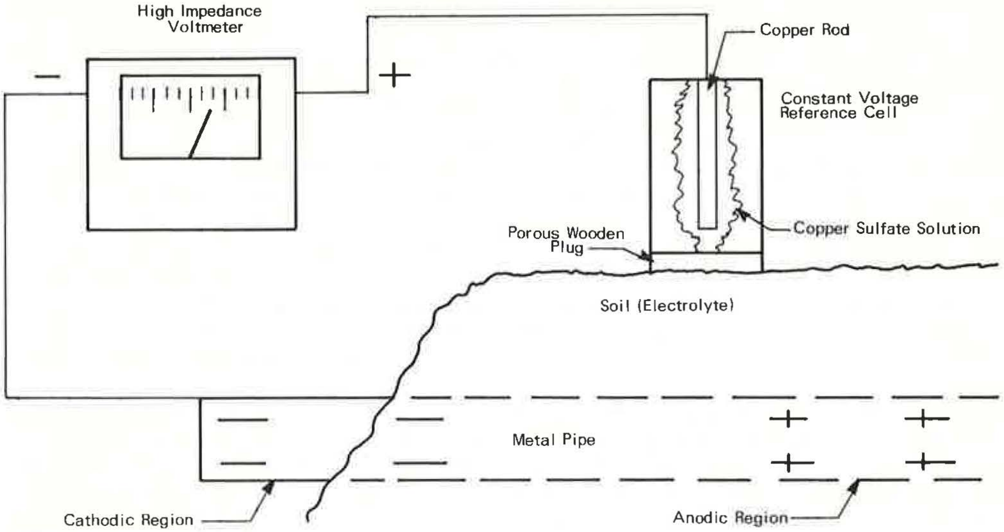
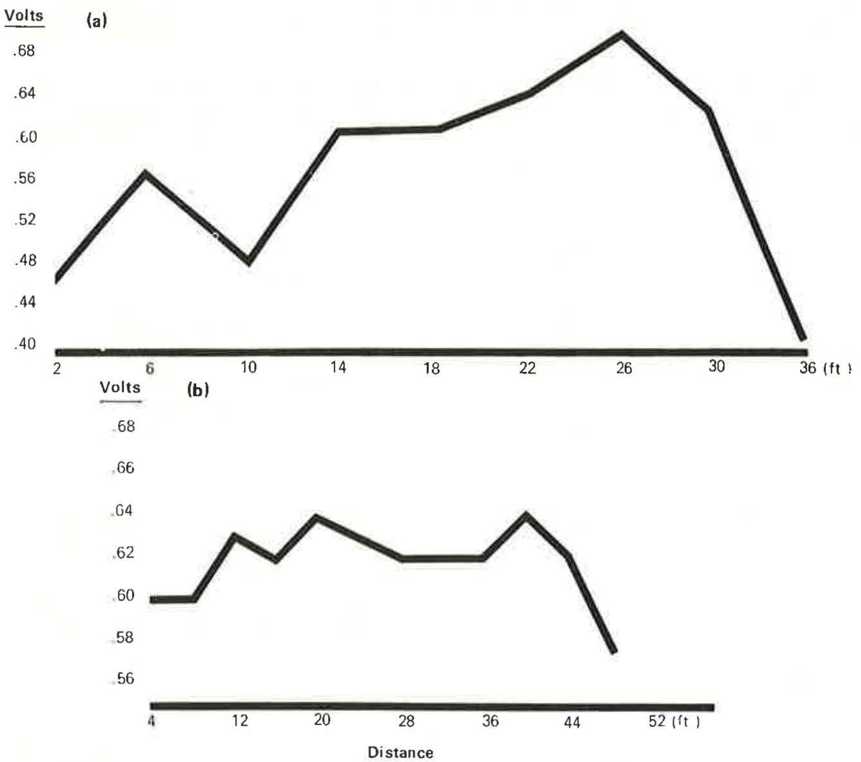


Figure 2. Corrosion test for corrugated metal pipe.



LINE GRAPH OF VOLTAGE FLUCTUATIONS TAKEN ACROSS PIPE CULVERT expression of area/length is derived from the testing cell dimensions. It is called the effective cell constant and has dimensions in centimeters. The effective cell constant is used as a fixed multiplication factor for a testing apparatus such as a soil box. For example, the reading (ohms) of the ac ohmmeter is multiplied by the effective cell constant of the soil box (cm) to determine the value of resistivity (ohm-cm).

As part of the corrosion study, buried corrugated metal pipes were tested in the field for corrosion by the voltage test method and soil samples were taken from the backfill material around each pipe. The soil samples were then tested for resistivity and pH in the laboratory (1). To determine whether significant relationships might exist between soil parameters and field tests, data were analyzed by graphical means and by numerical calculations on a digital computer. A rough correlation was developed between the pH-resistivity test and the voltage test method results from the field. For a wide range of resistivity values, the corrugated metal pipe corrosion data could not be correlated accurately. When the resistivity data were classified into categories of low, medium, and high values, more significant relationships were developed.

When the corrosion rate was plotted against time in the soil, an interesting relationship was found for medium- and high-resistivity soils. By analyzing the corrosion voltage test data, it was determined that nearly all the galvanized steel pipes demonstrated high reaction rates for the first few years. However, for medium- and high-resistivity soils (rho greater than 2100 ohm-cm) the rate of reaction decreased rapidly after 5 years (Figure 3).

When the corrosion data were plotted for low-resistivity soils, no definite relationship could be found. Sometimes the initial high reaction rate was found to continue, and in other instances the initial reaction rate decreased greatly. A variable such as moisture content may affect the rate of corrosion for soil of the low-resistivity classification.

It is interesting to note that the findings of the corrosion research study from the voltage test parallel the findings of Schwerdtfeger (2). Results from the research demonstrated low corrosion rates for non-saline soils with resistivities greater than 2100 ohm-cm. Similarly, Schwerdtfeger reports:

> In higher resistivity soils greater than 500 ohm-cm, there appear to be no regular variations between maximum pit depth and soil resistivity. For soils with resistivities over 2000 ohm-cm, and assuming the absence of stray currents or contact with more noble metals, the extrapolated data indicate that perforation of 8 inch diameter (0.322 inch wall thickness) steel pipe in 30 years is rather unlikely. However, the data definitely showed the need for protective measures, such as coatings, cathodic protection, or both, on wrought materials exposed to soils with resistivities less than 2000 ohm-cm, and even in some soils of higher resistivity, all depending on the hazard involved should a perforation occur....

Concerning low-resistivity soils, the Schwerdtfeger report states, "For periods of exposure up to 5 years, the maximum pit depths are on the average deeper in soils with resistivities up to 500 ohm-cm than in soils with higher resistivities" (Figure 4). Schwerdtfeger found that "perforation of a pipe wall ranging in thickness from 0.172 inch to 0.322 inch is predicted in almost all soils having resistivities less than 1000 ohm-cm." In Arizona perforated corrugated metal pipes were found in soils with resistivities of 20 to 1400 ohm-cm.

The voltage survey test showed that corrosion rates were higher in medium- and high-resistivity soils immediately after burial of corrugated metal pipes than 5 years later. Schwerdtfeger states, "For periods longer than 5 years, the rate of maximum penetration decreases as the soil resistivity increases." From these findings a similarity in both studies can be seen, even though different testing methods were incorporated. Results from this research study indicated that when a metal pipe is first buried there is a high rate of corrosion in most moist soils. In high-resistivity soils, the rate decreases after a few years, whereas low-resistivity soils continue corroding at a high rate.

Figure 3. Corrosion reaction of corrugated metal pipe in non-saline soils.

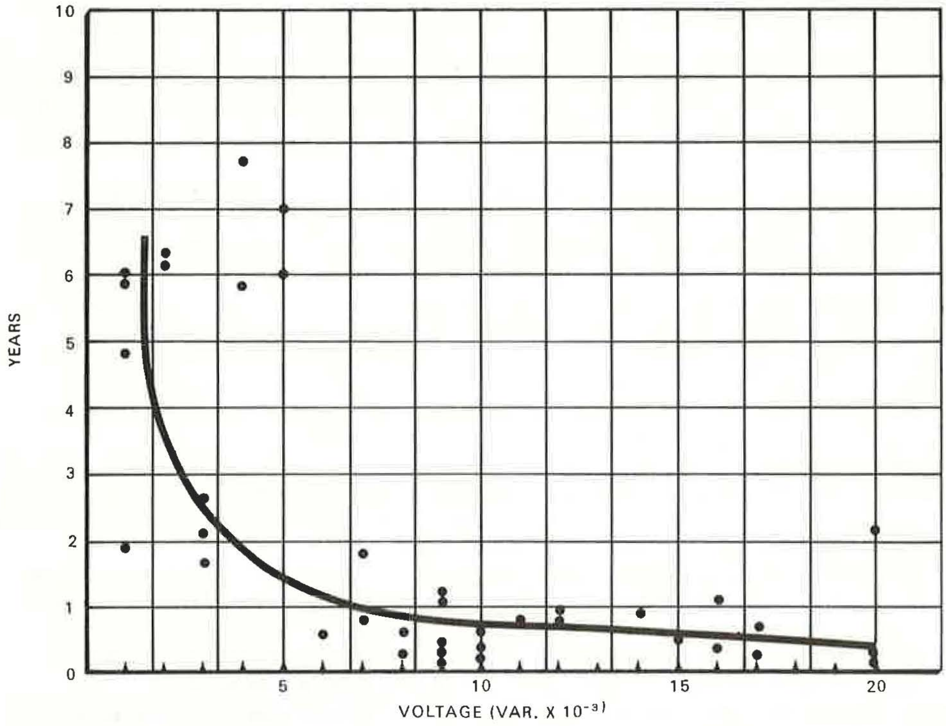
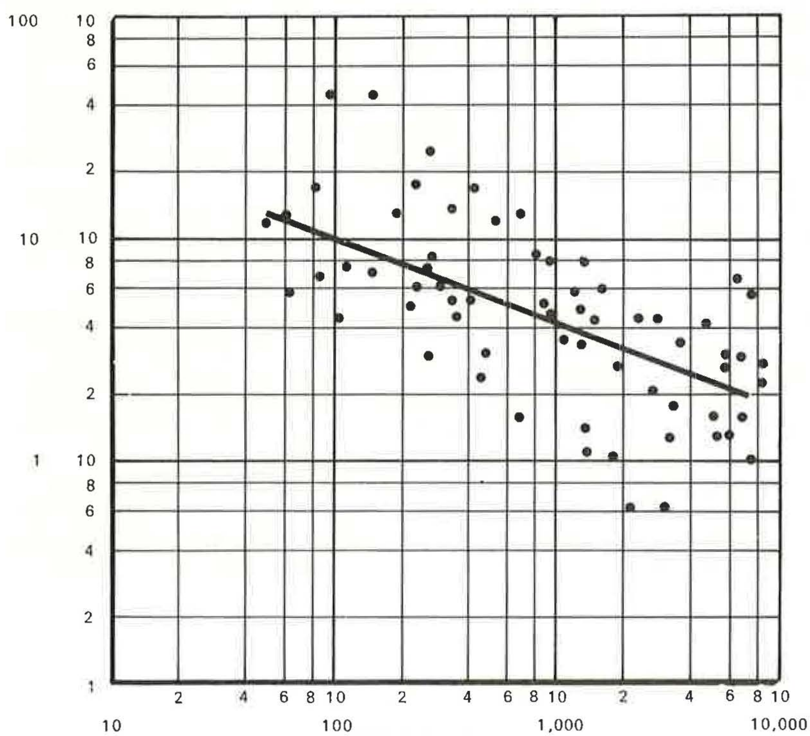


Figure 4. Rates of penetration based on maximum pit depth (National Bureau of Standards).



Polarization Test Method

Polarization is a method that tests corrosion reactions directly and has been widely used by corrosion engineers to measure instantaneous corrosion rates. The voltage of a metal surface in an electrolyte is a function of the corrosion reaction. When external energy in the form of direct current is applied to the metal in the electrolyte corrodent, a change in the corrosion rate is brought about and is reflected by a voltage change on the surface.

The metal surface becomes polarized when a voltage change occurs and reflects a corrosion behavior change. Voltage polarization is the change of voltage in either the positive or negative direction due to changing conditions of the corrosion system. The corrosion surface voltage is measured with a constant-voltage reference cell and is termed the corrosion potential. Normally the corrosion potential remains constant unless some change in the reaction occurs either externally or internally. For example, if an external direct current is applied to a rapidly corroding metal, the corrosion potential will not change greatly because the internal reaction is too intense. In the opposite case, when the corrosion rate is not rapid, the corrosion potential at the metal surface changes greatly for an applied external current.

The corrosion potential of the metal in the electrolyte can be measured by a calomel reference electrode. By using current and potential measurements, the corrosion rate can be quickly evaluated. In this manner, electrical measurements permit instantaneous testing of the actual metal in its corrosive environment.

With polarization testing methods, either the anodic or cathodic corrosion behavior of a corroding metal can be determined rapidly and accurately. As corrosion occurs on a metal surface, the current density and corrosion potential can be directly measured with polarization testing techniques. Once the value of corrosion current is calculated, the rate of corrosion of a particular reaction can be determined.

A constant-voltage reference cell is used to measure the corrosion potential in conjunction with a high-impedance voltmeter. The reference cell most widely used in the laboratory is the calomel electrode (Figure 5). The auxiliary electrodes must be platinum to remain inert and not interfere with the corrosion reaction. Also, platinum has repeatable and reproducible electrochemical properties. Two auxiliary electrodes were found to be preferable, one on each side of the test specimen to allow an even current distribution. The spacing of the electrodes was fixed and equal in relation to the metal plate (Figure 6).

In order to have the most corrosive soil conditions for the test, distilled water was added to the soil to reach saturation conditions. Approximately 100 to 200 milliliters of water was required for 400 grams of soil. Care should be taken to prevent a surplus water layer from forming above the soil level in the beaker.

After the metal sample is placed in the moist soil and the voltages have stabilized, a small positive direct current is transmitted through the auxiliary electrodes. The current supply must be well regulated, producing a constant current that will not vary under changing load conditions. When a small current is transmitted to the metal specimen, the voltage is changed to another value slightly above the corrosion potential and then remains constant. Once the value is recorded, an incremental current is added and another voltage value is reached by the system. This procedure continues until the voltage has changed about 10 millivolts above the corrosion potential. After the points have been plotted, a line is drawn on linear graph paper and the slope of the line is determined (Figure 7). By using techniques of polarization, the corrosion rate was determined from average slopes calculated from several tests. Measurements were obtained for small voltage changes with an incremental current every 30 seconds.

The equation for the corrosion current is $I = K/R$, where R is the polarization slope. The average value for the constant K is 26 millivolts. Once the value of I is found, Faraday's Law is used to determine the corrosion rate (3).

Thus:

$$\text{Rate} = 0.13\ (I)\ (e/d)$$

Figure 5. Apparatus for polarization measurements.

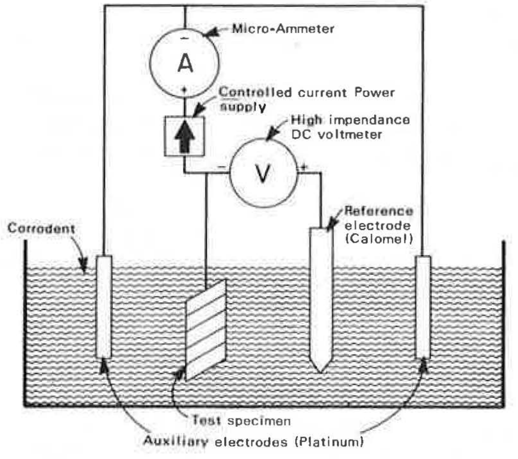


Figure 6. Fabricated polarization testing cell.

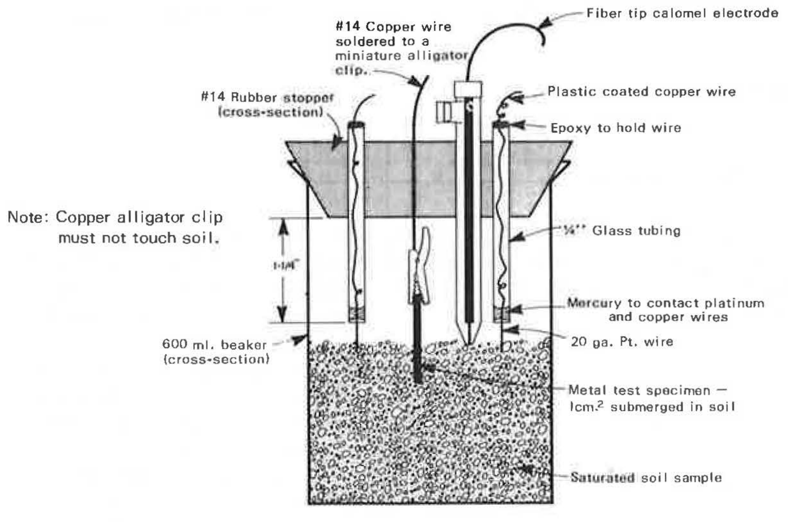
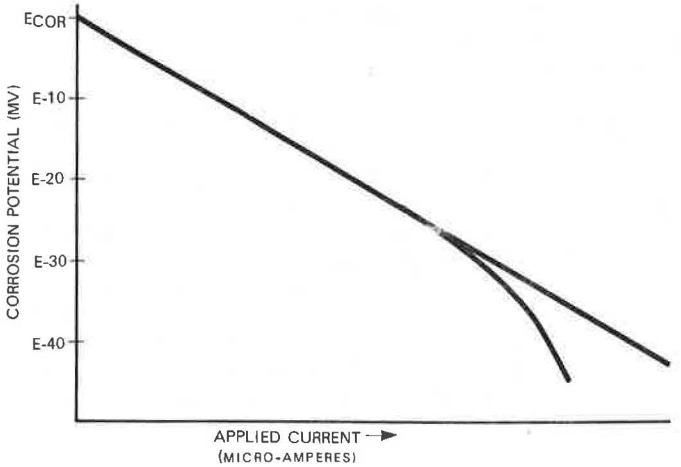


Figure 7. Linear polarization curve.



where e is the equivalent weight and d is the density.

When the thickness of the metal structure is known, the service life can be calculated based on the predicted corrosion rate (4).

After the voltage test was made to determine the corrosion of the corrugated metal pipe, a polarization test was made on the soil to see if a relationship existed between the tests. A good correlation was found between the corrosion predicted by the polarization method and actual corrosion in the field (Figure 8). A satisfactory coefficient of correlation and standard deviation were found for the tests (Figure 9). It would appear that a useful new direct test method was developed from the research study. The time required for the polarization test is approximately 15 minutes. The electrical test equipment needed is costly, but commercial test equipment is available. The complete test equipment costs approximately $1,000 to $2,000.

To determine the rate of corrosion for corrugated metal pipe from the polarization test results, the polarization slope is found for the sample. It was found that the corrosion rate of corrugated metal pipe was less than the predicted value by the polarization test. Since the test was made in saturated soil, it stands to reason that the predicted lifetime would be short unless the pipe soil was always in a saturated condition. To compensate for the difference in moisture conditions of the field and laboratory conditions, the test results may need to be adjusted to meet expected field conditions based on field-testing results. However, more research is needed to develop an accurate design for service life of corrugated metal pipe predicted by the polarization test.

CORROSION PROTECTION METHOD

Cathodic Corrosion Protection by Magnesium Anodes

Because galvanic corrosion is electrochemical in nature, it seems reasonable that there should be an electrochemical method for preventing corrosion. Corrosion takes place at anodic areas where current leaves the structure. If the entire structure is made a cathode it should be free of corrosion. Through the use of an anode to change the voltage, corrosion can be prevented.

To protect underground structures, the anodes are buried a distance from the structure. For even current distribution, the anodes are separated and located so the current will flow to all parts of the structure and cause it to be totally cathodic. Copper wires are connected between the structure and the anodes to conduct current.

Under most conditions that usually occur in low-resistivity soils, protective anodes made of magnesium can be used. These anodes drive current through the resistance of the soil to protect the structure. In this manner, the anodes, moist soil, and structure form a large galvanic protection circuit that operates continuously until the anodes have completely deteriorated.

If the soil between the anode and the structure has a high resistance, the sacrificial magnesium anodes do not supply enough current to adequately protect the buried structure. In this case, an impressed current power supply must be applied to overcome the soil resistance. The anodes used with the direct current power supply system are usually inexpensive graphite.

The current required to protect a structure depends on many factors. In general, at least 22 milliamperes per square meter of surface area are required. To determine the proper current, tests should be made on the structure. Anodes in most cases can be used to provide the necessary current if properly designed.

Magnesium anodes were installed on bridge pilings, and tests were made to determine if cathodic protection could be used on highway installations and how effective the protection would be against corrosion. Corroded steel bridge pilings in northern Arizona were protected with magnesium anodes. The pilings were wired together and connected to the magnesium anodes, which were spaced about 3 m apart. The bare steel pilings were adequately protected because of the voltage difference between magnesium and iron. Adequate current output was generated a few weeks following installation,

Figure 8. Pipe corrosion reaction versus polarization test.

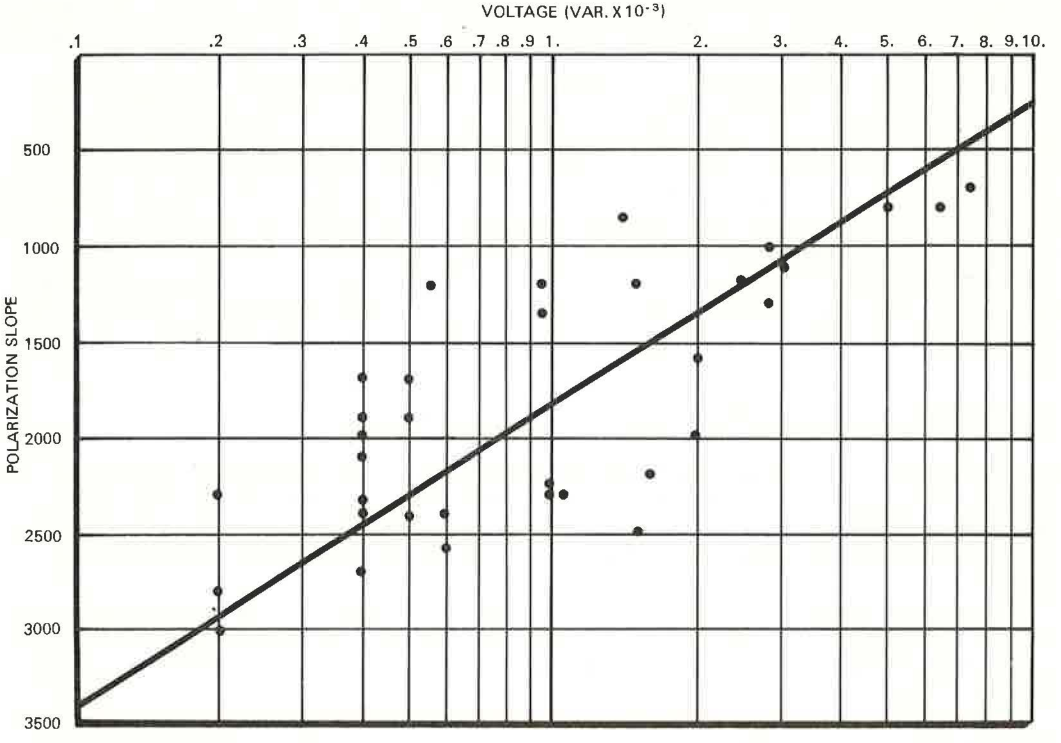
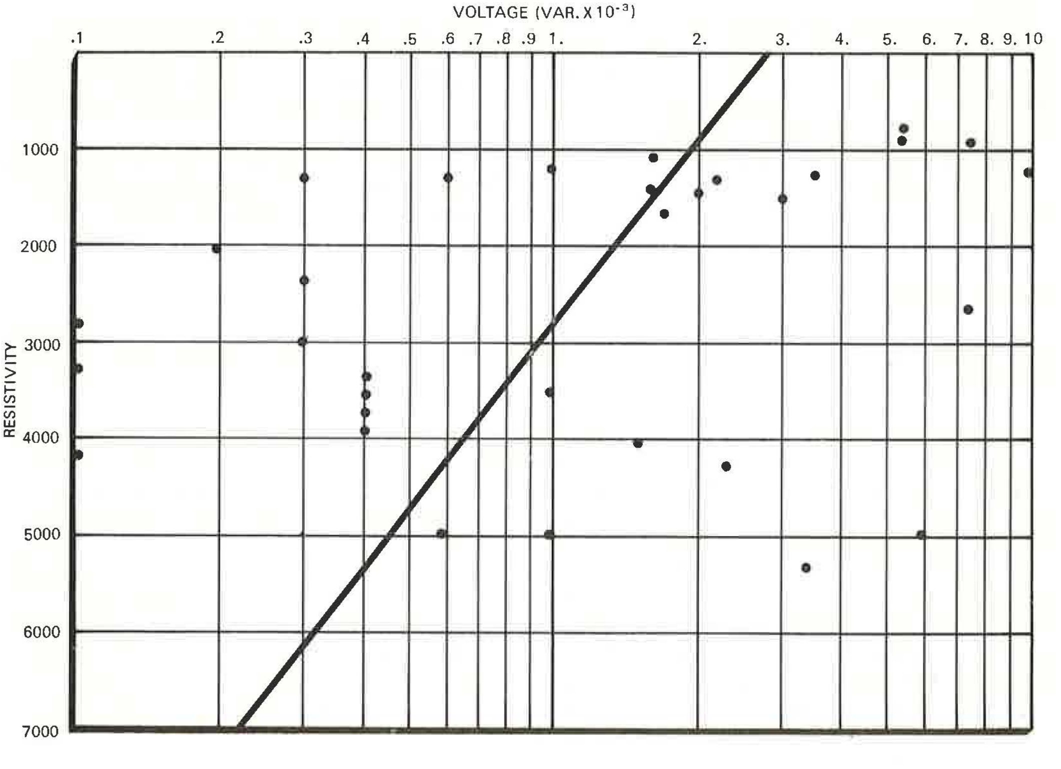


Figure 9. Corrosion reaction versus resistivity test.



and effective protection of the piling was achieved.

Cathodic protection of corroded corrugated metal pipe was also tested. Magnesium anodes were tested because in most locations power was not available near pipe culvert installations.

Test results indicated that the protection was not adequate to prevent corrosion. One possible explanation for the lack of protection of pipe culverts is that the large surface area of the pipe requires a sizable protection current to flow evenly over the entire pipe. In addition, to provide protection the anode conductivity through the soil must be sufficient to permit a large current to flow to the pipe. Even in low-resistivity soils, the current output to the galvanized pipe from the anodes was insufficient because the voltage generated by the zinc galvanizing opposes the driving voltage of the magnesium anodes. However, the use of high-output magnesium anodes may increase the driving potential enough to solve the problem. Further research is needed to develop improved methods for protection of corroded corrugated metal pipe.

Protection of Corrugated Metal Pipe by Bituminous Coatings

In saline areas of Arizona, where the soil usually has a low resistivity, corrugated metal culvert pipes are bituminous-coated to prevent corrosion. Test sites were selected in these regions for corrosion research studies to compare the effectiveness of bituminous-coated galvanized metal pipes with that of uncoated galvanized metal pipes.

The culvert metal pipes were located in a variety of soils and moisture conditions. Culvert metal pipes at these test sites have been installed for 5 to 25 years in corrosive soil conditions. The range of soil resistivity was 50 to 1300 ohm-cm, and the pH was 7.4 to 8.4.

The bituminous coatings on these pipes were visually inspected and usually were found in satisfactory condition. The asphalt coating in most cases demonstrated excellent adhesion to the metal surface, with no peeling or cracking beneath the soil. When exposed to the air, the asphalt surface coating was usually cracking and peeling from the metal surface because of weathering.

Voltage survey tests and polarization tests were performed to evaluate the corrosion activity on bituminous-coated metal pipes. These test results showed that (a) the voltage on bituminous-coated pipes was significantly lower on coated pipes in most cases and (b) the average corrosion current of coated pipes was a fraction of the current density on uncoated metal pipes.

The results of the study showed that bituminous-coated galvanized culvert pipes had a greatly reduced corrosion rate. In nearly all cases tested, bituminous coatings extended the predicted service life of corrugated metal pipe by at least 15 years.

CONCLUSIONS

As a result of the corrosion research study (5), the following new findings and testing methods were developed:

1. Corrosion behavior on buried corrugated metal pipe can be evaluated by measuring voltages along the length of the pipe and then computing the statistical variance of voltage readings. A high corrosion reaction rate has a variance of voltage data greater than 5×10^{-3} Low reaction rates have a variance below this value, and therefore the pipe is not damaged by corrosion.
2. The corrosion voltage tests made on galvanized pipe show that most buried pipes have high reaction rates for the first few years in any soil. In medium- and high-resistivity soils (resistivity greater than 2100 ohm-cm), the rate of reaction decreased rapidly after the first few years. In low-resistivity soils (resistivity below 2100 ohm-cm), the corrosion rate usually continues at a high rate even after several years.
3. The linear polarization method was used for laboratory soil testing to directly

measure corrosion rates of metal samples in corrosive soils. The estimated corrosion rates compared well with the actual observed corrosion on buried corrugated metal pipes in the field.

4. A comparison of the polarization and resistivity testing methods was made. The polarization method had a better correlation to field corrosion than the resistivity test but is more complicated to perform.

5. In low-resistivity soils, magnesium anodes were used to inhibit corrosion on bare steel pilings. Tests were made to try to protect galvanized corrugated metal pipe with magnesium anodes. These attempts were unsuccessful because the large surface area of the corrugated metal pipes requires a large protection current.

6. The use of bituminous-coated corrugated metal pipe is effective for preventing corrosion in saline soils. In most cases where bituminous-coated galvanized pipes were tested in Arizona, the coating was very effective for prevention of corrosion. No measurable corrosion was found on bituminous-coated galvanized pipes in Arizona. In most cases, the bituminous coating extended the predicted service life of corrugated metal pipe by at least 15 years.

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CORROSION OF GALVANIZED METAL CULVERTS

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Corrosion is a major factor in the life expectancy of metal culvert structures. Corrosion resistance, therefore, is of special interest to the user of galvanized culverts. A thorough investigation was conducted on 287 galvanized culverts located on the 56-mile (90-km) route of I-75 in the Upper Peninsula of Michigan. The service life of the culverts inspected ranged from 10 to 14 years. Visual observations and electrical tests were made at the culvert site to define the degree of corrosiveness. Water and soil were sampled and chemically analyzed to determine their relationship to or influence on deterioration. Significant corrosive attack was noted in 39 culverts. Test results revealed that major deterioration was occurring from the exterior (soil side) of the culvert. Exposure conditions found to cause excessive deterioration in uncoated galvanized culverts are dissimilar soil contacts, presence of organic soils, and differentials in aeration and soil moisture. Sulfates and chlorides were contributing factors to the excessive culvert deterioration as was biological corrosion in the form of sulfate-reducing bacteria.

•AN investigation of the corrosion performance of galvanized metal culverts was initiated by the Michigan Department of State Highways and Transportation in June of 1972 upon discovery of a severely corroded culvert that resulted in a collapsed roadway shoulder. A preliminary inspection along the I-75 route suggested that the deterioration might be widespread; therefore, an investigation was needed to define the scope and magnitude of the problem in Mackinac and Chippewa Counties where galvanized metal culverts were installed and to identify environmental factors influencing culvert durability.

Michigan has culvert installations using many different materials for drainage structures—galvanized steel, concrete, plastic, vitrified clay, and aluminum are all in service. Selection of culvert material is dictated by specific drainage conditions, design requirements, and economic considerations. Along Interstate 75 in Mackinac and Chippewa Counties, corrugated galvanized metal culverts were installed. The lightweight, securely banded, corrugated steel culverts met the necessary design criteria for the soft soil conditions prevailing in that area.

Durability (or service life expectancy) should be a prime consideration in the design of any underground structure. There is a general tendency of designers to look at fluid and strength requirements for drainage structures without establishing service life. The years of service a culvert gives are an important consideration for these essential installations along a heavily traveled highway. Frequent culvert maintenance and periodic replacement create high maintenance expenditures.

Metallic corrosion is a major factor in the life expectancy of metal culvert structures. Corrosion resistance, therefore, is of special interest to the users of galvanized culverts.

In the testing program initiated in this study to evaluate corrosion performance, 287 installations of galvanized culverts in various exposures were reviewed. Analysis

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of the data collected established a relation with aggressive parameters unfavorable to economical culvert life and provided a basis for a more knowledgeable approach to galvanized culvert corrosion.

CORROSION

Corrosion is the deterioration and loss of a metal due to electrochemical attack. The electrical energy needed for a corrosion reaction to occur is supplied from a galvanic cell.

In a basic galvanic corrosion cell (Figure 1) there must exist a potential difference between 2 points that are in electrical contact and immersed in an electrolyte. The electrolyte in underground corrosion refers to the soil moisture or liquid in contact with the metal and includes any other chemicals contained therein. Any 2 areas on a metal surface, known as a cathode and anode, that have a difference in potential (volts) and are within an electrolyte constitute the necessary components for a flow of current. When these conditions exist, current flows from the anode through the electrolyte to the cathode area and then through the metal to complete the circuit. The electrically charged atoms, known as ferrous ions, break away from the anode area. It is here that corrosion (loss of metal ions) occurs. The cathode is the area to which the current flows through the electrolyte and where hydrogen ions from the water are deposited. Any number of corrosion cells may operate on the same piece of metal as a network, each with its own anode and cathode.

The potential difference between the anode and cathode that drives the corrosion reaction can come from various sources. Almost any chemical or physical difference between the anode and cathode areas will support a corrosion cell, whether the difference is in the electrolyte or the metal.

All corrosion cells are associated with a flow of electricity. They operate according to Ohm's Law ($I = E/R$); that is, the amount of current flowing, and hence corrosion, decreases as the resistance of the circuit increases. The amount increases as the potential difference between the anode and cathode increases. Therefore, the rate of corrosion is dependent on the resistance of the electrolyte, which regulates the amount of current flow. Wherever this current leaves the metal and flows through the electrolyte, corrosion will occur.

Various types of corrosion cells are recognized (1). Those associated with culvert corrosion include (a) galvanic composition cells, (b) electrolysis or stray current, (c) biochemical corrosion cells, (d) oxygen concentration cells, and (e) salt concentration cells. The galvanic composition cell is established when dissimilar metals are in mutual contact in an electrolyte. The metal higher in the electromotive series acts as the anode from which current will flow and metal loss will occur. Dissimilarities in composition along the surface of a metal can also create a galvanic cell that supports corrosion. Electrolysis or stray current from improperly grounded dc motors or generators can also cause extensive and rapid corrosion. Biochemical corrosion is attributed to the direct or indirect attack of microscopic bacteria. These bacteria act as another environmental factor that accelerates the process of corrosion. The oxygen-concentration type of corrosion is set up when a metal within an electrolyte has 2 areas on its surface with different concentrations of oxygen. Oxygen content is influenced by many factors, such as the oxygen content of the water, the rate of diffusion, and the permeability of the corrosion products on the metal. High corrosion rates have been observed in oxygen concentration cells. The oxygen-deficient areas become the anodes, and corrosion occurs at these locations. The salt concentration cell is formed when a metal is in contact with an electrolyte in which the salt concentration varies. The area of the metal in the more concentrated solutions becomes the anode and corrodes.

PHYSIOGRAPHY

Study Area

The study was conducted on 56 miles (90 km) of I-75 in Mackinac and Chippewa Counties, as shown in Figure 2. All of the culvert sites inspected are located within the I-75 right-of-way in these 2 counties.

Surface Geology

This area lies in the Great Plains Region, with surface details formed during the Pleistocene Epoch. Most of the study area was covered by the main and post stages of Glacial Lake Algonquin and has a relief of less than 150 ft.

Lake bed deposits associated with Glacial Lake Algonquin are the most extensively mapped geomorphic feature in the study area, forming the surface for 92 percent of the study area. Predominantly sand, silt, and clay, these flat-lying deposits have a high water table and sluggish groundwater movement. Many of these lake-bed deposits are very poorly drained low areas, forming large swamps and wetlands. These swamps, located within the lake-bed deposits, make up 67 percent of the area traversed by I-75 in Mackinac County and 22 percent in Chippewa County.

Bedrock Geology

The study area lies on the northern rim of the Michigan Basin, where the edges of 12 different bedrock formations subcrop between St. Ignace and Sault Ste. Marie (Figure 3). Bedrock is found at or near the surface throughout most of the Mackinac portion of the study. Outcropping occurs primarily in the southern part of the county near St. Ignace, the M-123 interchange, Carp River, and the M-134 interchange. The overlying drift is much thicker in the Chippewa County portion of the study area, and no bedrock outcrops are encountered.

Of the 5 rock formations traversed in Mackinac County only the Salina formation is of particular interest. At or near the surface between M-123 and M-134, this formation is composed of limestone and dolomite interbedded with thin beds of salt, anhydrite, and gypsum. These evaporites greatly influence the soils in this area. The highest concentrations of sulfates and chlorides in the soils were found between M-123 and M-134. The bedrock underlying Chippewa County has a much thicker glacial drift mantle and does not directly influence the soil to the extent that it does in Mackinac County.

Soils

Mackinac and Chippewa Counties lie in the Podzol Soil Region of Michigan. The character of major soil associations existing along the I-75 route is considerably different in each county (Figure 4).

The Mackinac portion of the study area has swampy, poorly drained soils with a high organic content throughout most of its length. Between the Mackinac Bridge and Castle Rock Road the limestone bedrock outcrops or is very near the surface. Any thin discontinuous mantle of soil that does exist in this area is primarily well-drained sands and sandy loams. From Castle Rock Road to the M-134 interchange the soils are generally very poorly drained mucks, peats, sands, and loams. Imperfectly drained to well-drained clays and loams are the third and largest group of soils encountered in the study. These soils are mapped in most of the remaining area from the M-134 interchange through Sault Ste. Marie.

Figure 1. Corrosion cell.

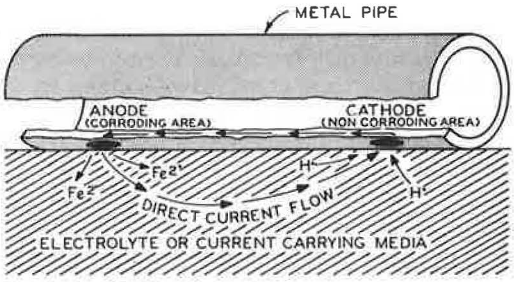


Figure 2. Location of the study area.

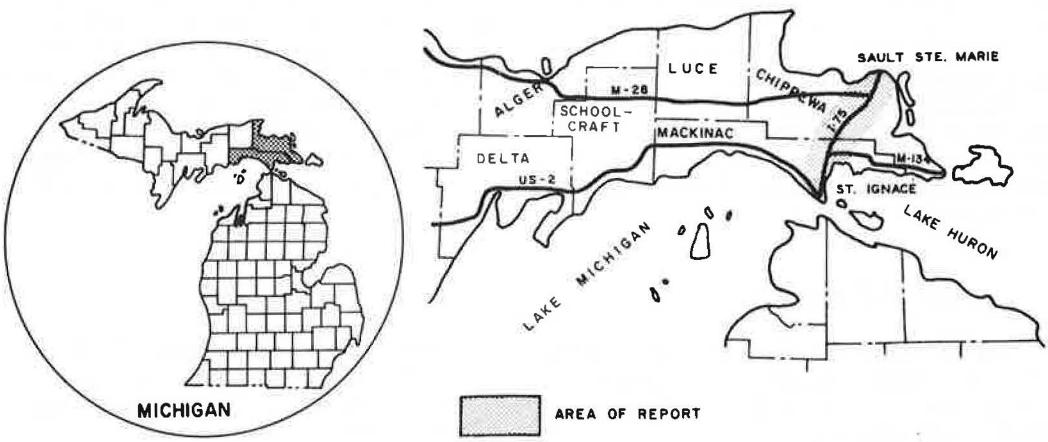
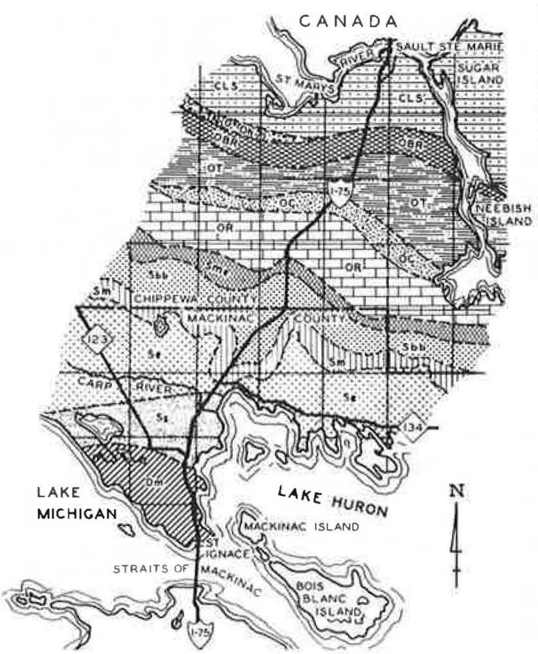


Figure 3. Major rock types.



BED ROCK GEOLOGY		
Dm	Mackinac Breccia	**Limestone and Limestone** Breccia, Chert, Shale, and Gypsum Blocks
Ss	**Salina Formation**	Shale, Interbedded with Limestone and Dolomite; Anhydrite; Gypsum; Rock Salt
Se	Engadine Dolomite	Massive, Hard Dolomite Interbedded with Limestone
Sm	Manistique Dolomite	Hard High Calcium Dolomite and Magnesian Limestone
Sbb	Burnt Bluff Limestone	Hard Limestone and Dolomitic Limestone. Some Beds of High Calcium Limestone
Smc	Cataract Group	Thick Bedded Dolomite, Gypsum, Ferrous, Calcareous, and Dolomitic Shale Members Near the Base of the System
OR	Richmond Group	Shale, Limestone, Argillaceous Limestone, and Interbedded Shale and Argillaceous Limestone
OC	Maysville Group	Bituminous and Dolomitic Shale
OT	Trenton Limestone	Limestone, Argillaceous Limestone and Dolomite
OBR	Black River Group	Limestone and Argillaceous Limestone
Oh	Hermansville Group	Sandstone and Dolomite
CLS	Lake Superior Sandstone Group	Sandstone

TESTING PROGRAM

The testing program consisted of a series of on-site determinations as well as laboratory analyses. Electrical resistivity and pH measurements were made at each culvert installation. The large number of sites inspected and the time constraints placed on the study by the urgency of the problem necessitated limiting the number of soil and water samples taken for chemical analysis. All tests made on the soil were performed on soils representative of that in which the culvert was lying; water samples tested were taken from the stream channel within the culvert.

Culvert Visual Examination and Rating

One aim of this study was to establish an inspection method that would provide a systematic procedure for defining the extent of the deterioration on a buried metal culvert. Each culvert was visually inspected and rated on a scale from 90 to 0 as adopted by the National Corrugated Steel Pipe Association (Figure 5). The top, side, and invert were carefully examined and assigned a rating. The lowest value was used to designate a measure of the culvert's service performance. Exterior culvert examination was limited to the top of the end section. Where physically possible, the entire length of the culvert interior was inspected and information recorded as to the general condition and estimated metal loss. Metal loss through corrosion was determined visually, aided with soundings made by a geologist's hammer.

Environmental Evaluation

Various physical and environmental conditions recorded at each site were watershed characteristics; culvert grades; stream rate of flow and direction; high water line; stream load in terms of abrasiveness, quantity, and sedimentation; soil series of the surrounding native soil; and the nature of the culvert bed material.

Electrical Resistivity

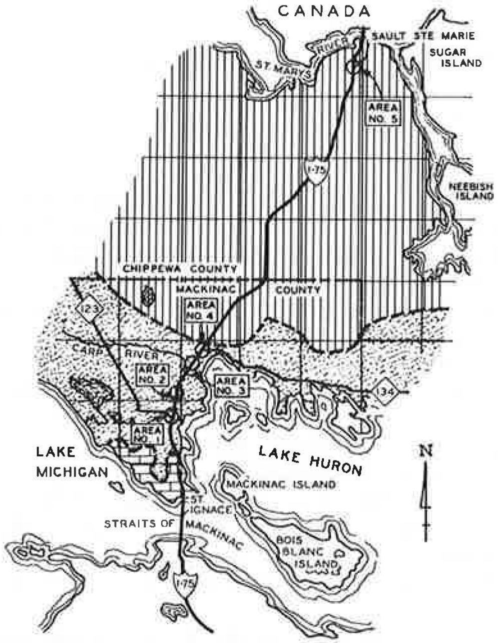
Electrical resistivity has universally been accepted as a rapid field testing method that indicates conduciveness to electrochemical corrosion. Electrical resistance is directly related to the quantity of dissolved salts in the soil and water; the higher the dissolved salt concentration the lower the resistivity. Several soil resistivity measurements using a Keck earth resistivity instrument were made at each culvert site to identify the area of lowest resistivity, which reflects the most corrosive condition. This aggressive area was selected for further testing and sampling.

A "minimum" soil resistivity was also obtained in the field using a soil resistance box and measuring the conductance with a Michi-Mho AC instrument in accordance with the apparatus and procedure developed by Beaton and Stratfull (2). This value of "minimum" resistivity provides a base level to which corrosion can be related.

Spontaneous Potential

An important consideration in any corrosion investigation is the detection of stray electrical currents within the earth. These currents may owe their origin to man-made mechanisms or naturally occurring phenomena and are known to cause rapid metal deterioration. Electrical currents flowing through the earth, regardless of their origin, are associated with potential gradients. A surface measurement of these gradients, called spontaneous potential, has been widely used in geophysical prospecting and is readily adaptable to corrosion analysis. Potentials were measured and a survey of mapped potential values was compiled for background data. Detection of large

Figure 4. General soil survey.



GENERALIZED SOIL CLASSIFICATIONS			
SYMBOL	GENERAL TEXTURAL CLASSIFICATION	GENERAL DRAINAGE CONDITIONS	MAJOR SOIL ASSOCIATION
	Well Drained	Limestone Bedrock Sands and Loams	Moran, St. Ignace, Johns Wood and Alpena
	Poorly Drained	Organic Deposits Sands and Loams	Muck, Peat, Roscommon, Saugatuck, Satago, Bruce and Angelica
	Imperfectly to Well Drained	Clays and Loams With Small Organic Deposits	Ontonagon, Bergland, Pickford, Muck and Peat

FIVE MOST SEVERLY CORROSIVE AREAS
AREA NUMBER 1 - 5 severely deteriorated culverts.
AREA NUMBER 2 - 6 severely deteriorated culverts.
AREA NUMBER 3 - 12 severely deteriorated culverts.
AREA NUMBER 4 - 3 severely deteriorated culverts.
AREA NUMBER 5 - 4 severely deteriorated culverts.

Figure 5. Rating scale for culvert inspection.

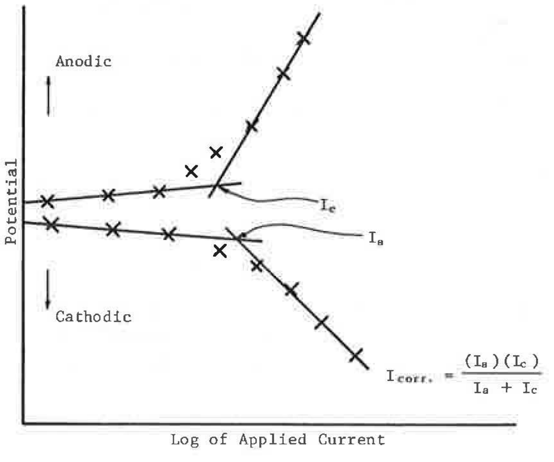
Visual Evaluation

	Rating	Comments
Top	______	______________________
Sides	______	______________________
Invert	______	______________________
Pipe exterior	______	______________________

VISUAL RATING SCALE:

- 90 Galvanizing intact
- 75 Galvanizing partly gone, some rust
- 50 Galvanizing gone, significant metal loss
- 25 Deep pitting, heavy metal loss, metal can be perforated with a sharp metal probe
- 0 Metal perforated

Figure 6. Polarization curves.



variances of mapped potentials would expose possible interference currents.

Polarization Test

Every electrochemical cell has an associated electric current that is directly related to the rate of corrosion. The polarization test is a technique used to measure the composite effective value of corrosion current, from which one can estimate corrosion occurring at the time data are obtained. Monitoring changes in electrical potential between the test specimen and soil induced by an impressed current allows a determination of the corrosion current.

By applying external direct currents both cathodically and anodically, the local galvanic action characteristic of the corrosion of buried metals is reduced to zero. A plot of the potentials measured versus the incrementally applied current determines where a change in slope or break occurs. This plot has important significance and is shown in Figure 6. The break in the curves where the two straight-line portions intersect represents the anodic and cathodic current (identified as I_a and I_c respectively) for the galvanic couples on the corroding metal. Pearson (3) from his studies derived an equation (Figure 6) that describes the corrosion current (I_{corr}), and with the use of Faraday's Law the weight of metal loss on a corroding metal surface can be calculated from this current.

The equipment used to measure the polarization voltages was a Sheppard's resistivity apparatus modified to perform according to the polarization circuitry developed by Lindberg (4). In order to more adequately represent the effects of all possible galvanic couples created on a large surface area of a culvert, this test method was conducted in the field.

Hydrogen Ion Concentration

Acidity or alkalinity is one parameter used to describe the aggressiveness of the natural environment. The pH investigations were directed toward determining the acidity and alkalinity of the culvert environment and correlating pH values with conditions that would be corrosive to metal culverts. To ensure representative measurements, all pH values for soil and water were obtained in the field at the culvert sites using portable pH electrometers.

Laboratory Measurements

Qualitative and quantitative chemical analyses were performed on water and soil samples to determine the presence or absence of specific ions. Water samples collected at every installation containing water were analyzed for their content of dissolved solids, chloride, calcium, magnesium, iron, sulfate, and hardness (as calcium carbonate). Total alkalinity was also measured and associated with dissolved solids and calcium in a common test called Langelier's Saturation Index (5).

Soil sample analysis included determination of the soluble quantities of chloride, sulfate, calcium, magnesium, and total iron concentrations.

Several metal samples were obtained from the severely corroded culverts for microscopic inspection. Each culvert sample was examined as to corrosion products and characteristic forms of corrosive action.

STUDY FINDINGS

This study involved the examination of 287 galvanized metal culverts. None of these culverts were bituminous-coated or paved. When inspected the culverts ranged in age from 10 to approximately 14 years. In terms of metal thickness the installations varied

Table 1. **Culvert condition from visual evaluation.**

Rating[a]	Number of Culverts
90	137
75	84
50	17
25	18
0	21

[a]National Corrugated Steel Pipe Association scale (see Figure 5).

from 16- to 8-gauge. Three basic classes were surveyed: circular corrugated metal pipes, corrugated metal arch pipes, and corrugated metal structural plate culverts. Each culvert structure was fabricated out of copper steel base metal galvanized by a hot-dip process with no less than 2 ounces of coating per square foot.

Records of tested stock reports on the installed galvanized culverts were reviewed to determine the possibility of a common materials factor related to the incidence of corrosion. It was found that a common denominator did not exist.

The field inspection revealed rather large differences in culvert performance in spite of the small differential in the range of culvert ages. All the culverts classified in Table 1 have been in service for an average of 11 years. As seen in the table, 39 culverts classed as 0 or 25 exhibited signs of heavy metal loss. Most of the severely corroded culverts, 30 out of 39, are contained within 5 geographic areas (Figure 4).

Metal samples were obtained from 15 perforated culverts for microscopic inspection. In 12 out of the 15 samples, the greater pit depth was found on the outside of the culvert sample. These observations coupled with the field inspections indicate that, although there is some corrosion activity taking place on the inside of the culverts, the major deterioration is occurring from the exterior, with the soil as the corrosive controlling factor.

Resistivity Results

Soil resistivity measurements taken at 277 culvert sites varied from a minimum of 684 ohm-cm to a maximum of 15 242 ohm-cm. The resistivity ranges and their relationship to corrosion are given in Table 2. At 188 culvert sites, which is 68 percent of total sites tested, the soil resistivities ranged from 684 to 4500 ohm-cm. This low range of resistivity offers little resistance to the flow of corrosion current and may be considered as a highly aggressive environment. It is significant to point out, however, that many culverts in excellent condition were found within this same range.

Water resistivity values obtained at 155 sites varied from a minimum of 311 ohm-cm to a maximum of 19 500 ohm-cm. As seen in Table 2, there were very few culvert sites containing water with resistivities over 4500 ohm-cm. Although this indicates high possibilities for corrosive action on the interior of the culvert, evidence of interior corrosion by runoff water was not established in this study through other tests.

It is apparent that the resistivity by itself does not ensure the occurrence of corrosion; it will, however, identify a potentially corrosive environment for metal culverts.

Electrolysis or Stray Current

An important form of underground corrosion recognized today is electrolysis or stray current corrosion. This corrosion results when direct current emanating from a source external to the buried pipe flows to the pipe and is eventually discharged from the pipe wall.

Examples of external sources of current include improperly grounded electric generators or equipment that can leak currents that travel through a low-resistance soil and support corrosion. Different lithologies in bedrock strata can also generate electrical currents. Potentials on the order of 1700 millivolts have been associated with ore deposits. A common source for stray cur-

Table 2. **Relationship of corrosion to resistivity.**

Resistivity Range (ohm-cm)	Corrosiveness	Number of Occurrences Soil	Water
0-2000	Severe	100	88
2000-4500	Heavy	88	47
4500-6000	Mild	29	6
6000	Little or none	60	14

rent corrosion is cathodically protected structures installed nearby.

A thorough investigation of the culvert's surrounding area with respect to any neighboring electric current sources was made in the study area. Spontaneous potential measurements were collected at each culvert site for analysis. A comparison of potentials taken at the corrosive sites against those in an area of no corrosion activity was used to distinguish any pipes exposed to interference currents. This analysis reflects no evidence of any culverts influenced by stray currents from an external source. Stray current corrosion is clearly not a factor in the culvert deterioration in this investigation.

pH Results

The pH was measured from May to August in a widely distributed topography consisting of pasture, tilled land, forests, and swamps. The soils classified in this region are of the Great Podsol Group, which is generally acidic. Th pH values of the soils tested at the culvert installations, however, were found to be near neutral. The average for all sites was 6.7, and it varied within a narrow band of 5.9 to 8.5. The surface water runoff at 154 culverts had a pH range of 5.7 to 8.6, with an average of 6.8. It is felt that the neutral soil and water pH measurements observed eliminate the possibility that culvert deterioration in this study area is caused by acidic corrosion.

Statistical Interpretations

Oftentimes a statistical examination will bring forth relationships that could not be seen otherwise. Data compiled on all culverts inspected were analyzed with statistical methods. A multiple linear regression was used to determine the relationship between the culvert's performance and corrosive factors in the culvert's environment. The culvert's performance was denoted by an assigned "rating" of 90 through 0. The variables measured at each culvert installation were then correlated against this visual "rating" that reflects the severity of corrosion.

Variables found to be the best indices of the culvert's service life (performance) are pH, resistivity, chloride, and sulfate. A multiple linear regression on these variables had a correlation of 0.41 with the culvert rating and a standard error of estimate of 31.30. This analysis accounted for only 16 percent of the data variation, suggesting that other factors not included in the regression analysis were largely responsible for the rapidly deteriorating culverts.

Further refinement of the data into special groupings drew attention to a definite relationship existing between deteriorated culverts and near-saturated soil conditions, aeration, and organic content of the soil. These soil parameters appear to play a significant role in the corrosion activity, particularly at the most severely deteriorated culvert sites.

Effect of Soil Factors on Corrosion

A relationship between corrosion and various soil types and characteristics, although difficult to establish, is necessary in the investigation of deteriorating buried metal structures. Potential differences developed at various areas on the surface of a buried metal are a principal factor in the corrosion process. Dissimilar soil types, drainage conditions, aeration, and the presence of organic material were associated with severely corroded culverts and can render a soil aggressive to metal structures.

Although most of the culverts were installed in trenches and backfilled with granular material from local sources, the culvert bed mat was usually the native soil. In many instances, some of the native soil (clays, loams, mucks) was included with backfill material during culvert installation. This condition of including native soils was found to occur primarily at the culvert end sections. The lack of homogeneity in the soils around the culvert can develop a potential difference that is capable of driving current in a corrosion cell. Visual inspections in this study revealed that 14 culverts that incurred heavy localized corrosion at the end sections were in contact with dissimilar soils.

Changes in the environment along the length or circumference of a culvert due to dissimilar soils can cause galvanic cell corrosion. When 2 areas on the surface of a metal culvert have dissimilarities and are joined in electrical contact in the presence of an electrolyte, electricity will flow from the anode to the cathode. In this study, many culverts that showed serious localized deterioration at their inverts were found to be installed on bed mats of clay backfilled with a granular material. Within this environment the portion of the culvert lying in the clay soil acts as an anode from which current is discharged and corrosion occurs.

Corrosion can also be formed by a variation in moisture of the soil around the culvert. A galvanic corrosion cell is created by the higher moisture content near the bottom of the culvert trench. Current flow in this case is from the bottom of the culvert through the soil to the top portion of the pipe. This type of corrosion occurs at the lower portion of the culvert.

Culverts lying in contact with poorly aerated native soils and well-aerated granular backfill are susceptible to corrosion through oxygen concentration cells. The culvert surface surrounded by poorly aerated soils acts as the anode. A cathode develops at that portion of the culvert with higher concentrations of oxygen such as well-aerated backfill or dissolved oxygen carried by flowing surface water. The difference in oxygen concentration induces a current flow from the anode to the cathode, causing metal ions to go into solution and corroding the anodic area of the culvert pipe.

Most of the severely corroded culverts were located within the 5 geographic areas shown in Figure 4. Mapping the native soil types shows that, of the 39 rapidly deteriorating culverts, 22 were in areas of very poorly drained mucks and peats. Poorly drained clays, loams, and sands were found at most of the remaining corrosive sites.

A characteristic of organic deposits and fine-grained soils with small pore spaces is sluggish groundwater movement. Low permeability restricts water movement and therefore limits oxygen diffusion within these soils. Mottled brown, gray, and black soils were encountered within the environment of many deteriorating culverts. This mottling effect is associated with the more poorly drained and less aerated soils. The natural drainage of a soil is an important factor in terms of corrosiveness because of its effect on moisture content, water movement properties, and aeration. A definite correlation between such poorly drained, poorly aerated soils and areas containing deteriorated culverts was established during this study.

Biological Corrosion

Another form of corrosion showing its effects on drainage culverts is biological corrosion. Severe corrosion of buried metal structures has been attributed to the activity of bacterial organisms in an oxygen-free environment.

The theory of the manner in which bacteria stimulate corrosion was proposed by von Wolzogen Kuhr and van der Vlugt (6). The process involves the consumption of hydrogen from the cathodic areas on the surface of steel by microbiological organisms. The main bacteria of concern is Spiro Desulfuricans, which utilizes hydrogen to break down sulfates into sulfides. By preventing a protective hydrogen film from forming (depolarization), current flow continues and hence allows corrosion to proceed unchecked.

Detection of bacterial corrosion is accomplished by noting the nature of the corrosion and characteristic corrosion products. The effect of bacterial corrosion is pitting of steel. In severe corrosive conditions, pits are concentrated close together and fused to produce large corroded areas. Von Wolzogen Kuhr and van der Vlugt (7) state that ferrous sulfide (FeS) was present in the highly corrosive anaerobic soils that they studied. A black, hard crust of FeS is commonly observed over the corroded area of the pipe. Upon removal of the loosely attached corrosion products, a bright metal surface is exposed. Generation of H_2S by treatment of the corrosion products with hydrochloric acid is a positive test for the presence of sulfide and is used to indicate

Table 3. Culvert rating versus corrosion current determined from polarization measurements.

Culvert Rating[a]	Corrosion Current (ma)	Metal Loss ($gram/ft^2/year$)
90	3.25	0.1050
75	4.54	0.0760
50	8.41	0.1787
25	6.09	0.1287
0	11.18	0.2713

[a]National Corrugated Steel Pipe Association scale.

the influence of Spiro Desulfuricans.

The conditions favorable for the growth of sulfate-reducing bacteria are (a) presence of moisture; (b) pH between 5.8 and 8.2; (c) total absence of air; (d) presence of organic matter; and (e) presence of sulfate. Sites with active anaerobic bacteria can be expected in flat, low-lying lands or swamps that maintain a high water table. Poorly drained, heavy-textured soils, such as clays and clay loams, are commonly involved. Peats and mucks rich in mineral and assimilable organic compounds are excellent breeding grounds for sulfate-reducing bacteria.

There is a basis for believing that bacterial corrosion plays an important role in some of the severely corroded culverts encountered in this investigation. Various factors recognized during this study lend support to this concept. Statistical data show that the majority of severely deteriorated culverts were found in anaerobic soils of high sulfates with organics available. In 15 out of 21 cases of severely perforated culverts, the hydrochloric acid test of the corrosion products gave evidence of sulfide. Corrosion products and forms of pitting examined under the microscope reflect those characteristics reported by other investigators as bacterial corrosion. Although the microorganism was not visually identified, its symptoms were detected.

Polarization Voltage Measurements

In that polarization measurements can give an indication of the magnitude of the corrosion activity on the surface of a buried culvert, this test method was intended to determine culvert metal loss in grams per square foot without the need of physically exposing the culvert for visual examination.

Time allowed only a preliminary trial of the polarization test in this study, and therefore the analysis is limited to the collection of measurements and plotting of polarization curves at 46 culvert sites. The corrosion current and calculated metal loss have been compared to the culvert ratings determined through visual inspection, and the results are given in Table 3. A correlation can be seen between the culvert rating and corrosion current. The mean value of the corrosion current, 3.25 ma, for culverts in good condition (90 rating) is relatively lower than the corrosion current, 11.18 ma, found at the perforated culverts (0 rating). A positive relation is not as evident when comparing metal loss per square foot with visual rating. This results from polarization data collected at culverts with large variations in surface areas.

Problems did arise from what appeared to be erroneous values. Several individual measurements obtained at various culvert sites could not realistically be assumed to occur for the duration of a year in view of the culvert's condition. The erratic values of the corrosion current measured at culverts with a 50 rating indicate this. Further definement of equipment and test procedures, therefore, is necessary before meaningful service life determinations can be derived.

CONCLUSIONS

1. Environmental exposures should be a prime consideration in the selection of culvert materials to ensure an economic service life.
2. An inspection of 287 galvanized metal culverts that have been installed for 10 to 14 years revealed 39 that are seriously corroded; 30 of these are confined to 5 limited areas.
3. The establishment of low resistivity determinations at a culvert site will not ensure the occurrence of corrosion; it will, however, identify a potentially corrosive environment.

4. Neutral soil and water pH measurements observed in the study area eliminate the possibility that the culvert deterioration is caused by acidic corrosion.

5. A spontaneous potential test procedure developed to detect stray electrical currents indicates that the deterioration of the buried culverts is not caused by interference currents.

6. The statistical evaluation of all soil and water parameters studied rendered pH, resistivity, chlorides, and sulfates as the best indices to the rate of corrosion. These variables had a correlation coefficient of 0.41 with the visual rating and only accounted for 16 percent of the data variation, suggesting that other factors not included in the regression analysis must be largely responsible for the rapidly deteriorating culverts.

7. Examination of the 5 delineated areas containing rapidly deteriorating culverts in this study reflects the importance of considering the following soil factors in determining the service performance of a metal drainage structure: uniformity of soil backfill, presence of organic materials, differential aeration, and soil moisture differentials.

8. Bacterial action is another factor that plays a role in underground corrosion. The presence of sulfate-reducing bacteria in anaerobic soils containing organic material increases the aggressiveness of the soil by preventing the formation of a protective hydrogen film on a buried corroding culvert.

9. The corrosion current determined from polarization measurements compared favorably with observed corrosion rates. Polarization measurements, as a field test, have merit in describing corrosion activity and can be improved by additional development of technique and equipment.

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CORROSION TESTING OF BRIDGE DECKS

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When the corrosive half-cell potentials on a bridge deck exceed about 10 percent or when corrosion-caused delamination exceeds about 1 percent of the deck area, a chloride analysis generally would not be required because the chloride content is already too great. For the average depth of reinforcing steel, the quantity of chloride apparently needed to cause corrosion was statistically related to the maximum amount at the 95 percent confidence limits of 1.0 lb/yd^3 (0.59 kg/m^3). The accuracy of chloride determinations was about equal whether the concrete was drilled or cored. Although the half-cell potential of -0.35 volts CSE is indicative of active corrosion, an equipotential contour map is the most reliable means for evaluating the corrosion activity of steel in concrete. After repair of concrete delaminations, the percentage of corrosive potentials decreased by about 50 percent. Repairing concrete delaminations does not prevent or necessarily control corrosion at other locations.

•SEVERAL reports of investigations relate the causes of or factors that influence the corrosion of steel in concrete bridges (1-12). Some also contain laboratory types of investigations of field structures (1, 3, 10, 13-15). For the most part, however, the published reports are not oriented to screening large numbers of structures for an operational type of evaluation of the corrosion condition.

As reported by Kliethermes (4), Hall (5), and others (6, 7), the corrosion of steel in concrete bridge decks is related to the use of deicing salts, and the problem was found in 46 states. In NCHRP Synthesis 4 (6), concern was expressed for 200,000 bridge decks in the United States. Because of the large number of structures that have been exposed to deicing salts, it is obvious that research techniques must be streamlined to determine operationally the methods of repair and/or preservation of the decks (2-6, 8, 10, 15). Except for special cases, there are simply too many bridges and too few people and dollars to spend a long period of time thoroughly evaluating each structure. Therefore, the intent of this investigation was to find a reasonable means to obtain the necessary technical information with a minimum of effort and resources.

In general, it has been demonstrated that the corrosion activity of steel in concrete can be nondestructively determined by half-cell potential measurements (7, 10); by evaluation of physical concrete distress through visual observation, sounding (10), or chaining (11); by ascertaining the chloride content associated with the corrosion of steel in concrete (1, 3, 7-10, 13); and by determining the concrete cover over the corroding steel (2, 6-10, 12, 13, 16).

With a few exceptions, concrete quality in California is good enough that corrosion is not of special significance. This is not to imply that concrete quality does not affect the time to corrosion of the steel. It is only pointed out that, once the bridge is built, it cannot be changed, and concrete that is structurally and physically sound has no real function in correcting, controlling, or preventing corrosion of steel in existing bridge decks. Therefore, this corrosion investigation was specifically oriented toward evaluating corrosion investigation techniques and did not include a systematic evaluation of concrete strength, absorption, air entrainment, etc., although these factors do have a significant effect on performance if seriously deficient.

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As a result, chloride analyses were made of the bridge deck concrete to evaluate quantity, effects of sampling method and number of samples, and relationship to concrete distress and to the 2 techniques employed for obtaining half-cell potentials of the steel. In addition, an evaluation was made on 5 structures to determine the effect of repairs on the change in the half-cell potentials of the steel.

BRIDGES INSPECTED

Twenty-two bridges were inspected in 1972 and 1973 to determine the operational feasibility of using various inspection techniques to evaluate the corrosion behavior of the steel in the decks. The overall results are given in Table 1.

As Table 1 shows, the data accumulation for the various bridges consisted of a chloride analysis (8, 15), the measurement of half-cell potentials (7), measurements of concrete delaminations (11), and the concrete cover. Where the cover is shown to the closest 0.01 in., it was measured by a pachometer; when the value is shown to be plus or minus, the indicated concrete cover was not measured but was that specified.

As will be recognized, when dealing with a random investigation of individual field structures, the data obtained do not always result in information that is suitable for an overall analysis. For example, the investigation was directed at evaluating inspection techniques for structures that had been exposed to deicing salts; however, one older bridge was found to have been constructed with calcium chloride added to the concrete. In addition, another bridge deck, selected as being in "good condition", was found to have an average of 5 lb of chloride per cubic yard at the level of steel (2.97 kg/m^3), and corrosion was not active and there was no evidence of past distress. The reason for the passive condition of this bridge deck was not investigated, although previous work (10) has shown that corrosion may be dormant for a period of time in salt-contaminated concrete under conditions of low moisture content.

In Table 1 the chloride content at the level of the steel is given in terms of the average, the maximum content analyzed in any sample, and the maximum content that would be indicated by calculating a statistical distribution and deriving the maximum quantity at the 95 percent confidence limits of the data. The 95 percent confidence limit was used to define a repeatable maximum limit of chloride content rather than to depend on a randomly obtained maximum value. In Table 1 there is fair agreement between the actual maximum chloride content found in a bridge deck and the maximum calculated from the 95 percent confidence limit of the data.

Table 1. Bridge condition variables.

Bridge	Chloride Content (lb/yd^3)[a] Average	Maximum in Sample	Maximum at 95 Percent Limit	Corrosive Potentials (percent)	Delamination (percent)	Years of Service	Average Concrete Cover (in.)
Weimar, Right	0.2	0.4	0.6	3	0.1	14	1.5±
Weimar, Left	0.2	2.0	0.8	5	0	14	2.43
New England Mills, Left	0.2	1.6	1.6	7	0	14	2.50
New England Mills, Right	0.5	1.6	1.6	4	0	14	2.71
Grapevine, Left	0.9	1.4	1.1	—	5	23	1.5±
Grapevine, Right	0.9	1.2	1.2	—	22	23	1.5±
Cressy	1.0	2.4	3.1	—	18	22	1.5±
Lebec Road	1.3	3.4	2.9	—	28	9	1.5±
Gray Creek	1.0	1.6	1.9	17	6.7	6	1.90
Canyon Creek	1.3	1.9	2.3	4	5.5	7	1.66
Ft. Tejon	2.0	3.6	3.2	—	40	23	1.5±
Pony Bar	1.9	3.1	3.4	35	30	6	1.70
Milagra Drive	3.0	6.4	6.4	6	11	7	1.30
101/116 Separation	2.7	4.9	5.6	2	7.6	17	1.43
Sawmill	2.2	3.5	3.7	10	0.4	8	1.78
Mt. Shasta, North	2.6	4.1	4.2	21	1.3	8	1.58
Sly Park	3.5	6.0	6.0	37	12	8	1.68
Lake Street	4.0	5.8	5.7	47	10	8	1.56
Long Valley	5.0	8.0	8.6	0	0	14	1.5±
Lebec	5.0	8.8	10.0	—	28	9	1.5±
Mt. Shasta, South	6.1	9.9	8.2	73	6.7	8	1.58

[a]At the depth of steel; 1 lb/yd^3 = 0.59 kg/m^3.

To obtain the chloride content at the level of the steel, the original data were plotted on semilog paper. The chloride content was plotted on the log scale ordinate against its associated depth below the surface of the deck on the linear abscissa. For the approximate 4-in. (10.2-cm) depth of the cores, the best fit of the points was made by a straight line, then the chloride content at the level of the steel was obtained for the determined average depth of cover.

In Table 1 the column "Corrosive Potentials (percent)" represents the percentage of potential measurements of the steel in the bridge deck relative to the saturated copper-copper sulfate half-cell (CSE) that are more negative than -0.35 volts (13, 14). Also, the column "Delamination (percent)" represents the percentage of the bridge deck having the concrete delaminated as determined by the chain drag (11). The "delamination" designation also included any surface area of the deck from which concrete had already spalled as a result of corrosion.

Chlorides Versus Potentials and Deck Delaminations

Data were grouped according to ranges of chloride content and then the averages for potentials and delaminations for the bridge decks were plotted against the associated chloride content, as shown in Figures 1, 2, and 3.

As shown by Figure 1, when the average chloride content was 0 to 1.0 lb/yd^3 (0.59 kg/m^3) or less, the average percentage of corrosive potentials was 4.8 percent of the total measurements, and the average percent of concrete delamination was 4.5 percent of the total deck area.

In Figure 2, where the maximum chloride content in any sample at the level of the steel was used as a criterion, it will be observed that, for chloride contents of less than 1.0 lb/yd^3 (0.59 kg/m^3), the percentage of corrosive potentials was found to average 4 percent, and the average area of delamination was 0.05 percent.

In Figure 3, which uses the maximum chloride content at the average level of the steel calculated to the 95 percent confidence limit, the average percentage of corrosive potentials was 4 percent, and the average area of delamination was 0.05 percent.

In Figure 3, which uses the maximum chloride content at the average level of the steel calculated to the 95 percent confidence limit, the average percentage of corrosive potentials was 4 percent, and the average area of delamination was 0.05 percent.

It is obvious from Figures 1 through 3 that a maximum chloride content at the average level of the steel is related to the incidence of active corrosion potentials, and the resulting concrete delamination confirms the amount previously associated with corrosion of the reinforcing steel (10).

Figures 2 and 3 may give a misleading impression that increasing chloride contents result in an increase in corrosive potentials and delaminations. Because the structures are those that have received deicing salts over a period of time, increasing salt content only reflects a gain of salt in the concrete with time. The continuing corrosion of the steel in concrete is time-dependent and not necessarily related to increasing salt content above some threshold level. For example, it is not required that the bridges in Table 1 have more than 3 lb of chloride per cubic yard (1.8 kg/m^3) in order to contain 26 percent corrosive potentials and 13 percent of the area delaminated. Once corrosion begins, it is time-dependent in that it becomes more extensive as time increases. In concrete, the primary function of the chloride ion is to destroy the passivity of the steel. Once this occurs, the actual corrosion rate of the steel is controlled by polarization and other effects as well as the continuance of the concrete as an electrolyte. An increase in salt content is not necessary to keep corrosion active.

For example, in Table 1, one bridge is listed with 8 years of service that contains a maximum of 9.9 lb of chloride per cubic yard (5.8 kg/m^3), and the relative area of corrosion-caused concrete delamination on the deck is 6.7 percent. Conversely, another bridge with 6 years of service has a maximum chloride content of 1.6 lb of chloride per cubic yard (0.94 kg/m^3) and also has 6.7 percent of its deck delaminated by corroding steel. From these and other data shown, it is obvious that, once the concrete becomes chloride-contaminated, the corrosion-caused distress is not controlled

Figure 1. Average chloride versus corrosive potentials and deck delaminations.

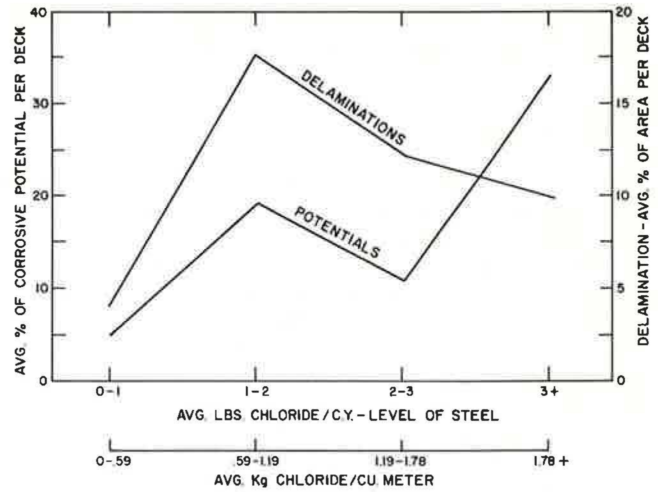


Figure 2. Maximum chloride versus corrosive potentials and deck delaminations.

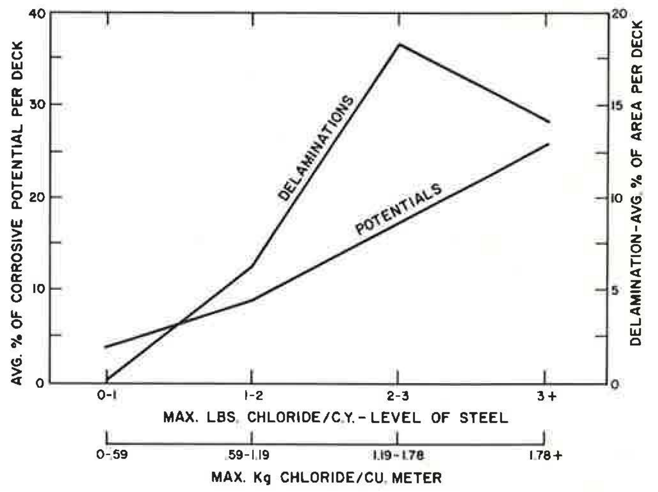
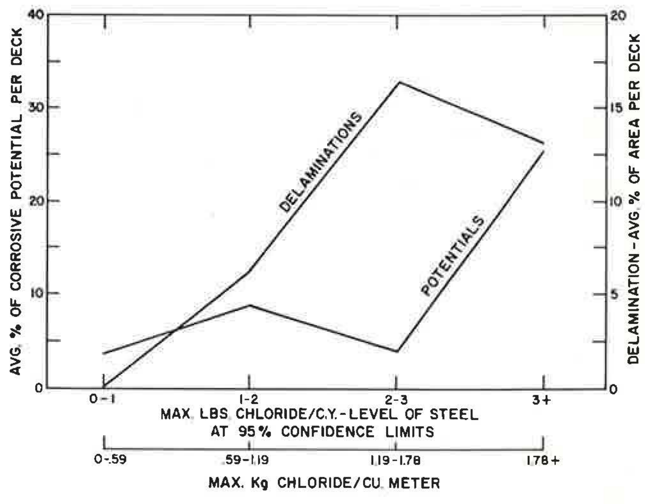


Figure 3. Maximum chloride versus corrosive potentials and deck delaminations at 95 percent confidence limits.



by chloride content beyond that needed to depassivate the steel but is controlled by other variables that specifically relate to the corrosion process and its effect.

Chloride Sampling

On 6 bridges, the chloride content of cores that were sliced and then pulverized in the laboratory was compared to that obtained by drilling the concrete and analyzing samples that were pulverized in situ. Results are given in Table 2.

At each bridge, cores 3 in. (7.6 cm) in diameter were obtained and sliced into 1-in.-thick discs that were then pulverized and analyzed. At each core location and approximately 1 in. (2.5 cm) from the edge of the core hole, 4 drill samples were made at 90-deg intervals about the circumference of the core hole. The drillings of the 2 different diameters of drills were first made to a 1-in. (2.5-cm) depth below the surface, then the pulverized concrete was removed with a small spoon. The hole was then air-blown to clean out the residue, and the process was repeated for the depth between 1 in. (2.5 cm) and 2 in. (5.1 cm) below the deck surface.

As shown in Table 2, the mean values for the chloride contents obtained either by drilling or coring do not appear to be consistently different statistically, as evidenced by the standard deviation and coefficients of variation.

In some cases, as shown in Table 2, the coefficients of variation for the results of the drilled samples are both greater and less than those for core samples. For an equal number of observations, the greater the coefficient of variation, the less the accuracy of the mean. Therefore, it seems that the major variable is an inconsistent distribution of chlorides through the structure rather than the diameter of the sampling apparatus, per se. This is further emphasized by the fact that the average of the coefficients of variation for the chloride analysis of the drill samples was about 33 percent whereas the average of the coefficients of variation for the cores was 36 percent. In a previous study (7) the coefficient of variation for cores was about 30 percent. From this, it is obvious that a sufficient number of samples (say, at least 6) should be obtained in order to evaluate the chloride content with a much higher level of confidence than can be obtained with a smaller number of samples.

POTENTIALS AND CONCRETE CONDITION

Half-cell potentials were measured and tabulated (Table 3) for 8 bridges having concrete delamination. In large delaminated areas, where 2 or more measurements were made, the potentials were tabulated for a maximum and minimum value. In small areas of delaminations, where only 1 potential measurement was made, it was listed as an "isolated" potential.

In general, where the length in area of delamination was about 3 ft (1.0 m) or more, at least 2 potential measurements were made. The average of the maximum potentials was -0.453 volts CSE, and the average of the minimum potential within the same delaminations was -0.334 volts CSE. At concrete delaminations that had a diameter (roughly) of about 1 ft (0.3 m) or less, the average of the single potential readings made in these locations was -0.385 volts CSE.

As shown in Table 3, in locations where the concrete was not delaminated, the average of all potential measurements was -0.180 volts CSE.

On a field structure, as compared to laboratory specimens, there is a greater possibility of error in defining corrosive and noncorrosive potentials. This is because of the polarizing effects between the anodes and cathodes on the large masses of steel in a field structure. This effect will reduce the actual open-circuit potentials of the anodes and simultaneously increase the potentials of the cathodes.

The maximum measured potential in any corrosion cell should be used as a criterion for analysis. In this regard, since concrete delamination on the 8 bridges surveyed was observed, when the average of the maximum values was -0.453 and the average of the isolated values was -0.385 volts CSE, it is obvious that these values signify active corrosion of the steel. Even though these values indicate the potential of corroding steel, it must be recognized that they represent an advanced stage of corrosion and are of a numerically greater potential than when corrosion was initiated.

The determination of noncorroding potentials of steel is not defined by the criterion that the concrete is not delaminated. This is because the corrosion of the steel may not as yet have progressed to the stage where it causes the concrete to fracture. This is illustrated in Table 3, where the average potential of the steel in nondelaminated concrete for one bridge (South Mt. Shasta) was -0.382 volts CSE whereas the average for all structures was -0.180. Although nondelaminated concrete cannot be used to establish clearly either an active or passive potential of the steel, it is obvious that the bridge deck concrete was observed to be sound when the average potential was -0.180 volts CSE.

For a high level of assurance that corrosion is active, a potential numerically greater than -0.350 volts CSE seems to be a reliable criterion. In certain cases, because of the probable effects of polarization, etc., measurement of potentials of steel will be in the range of -0.25 to -0.35 volts CSE and concrete distress may be evident.

POTENTIAL SURVEY METHODS AND DECK REPAIR

Half-cell potentials can serve 2 or more purposes, which can encompass (a) determination of the locations where steel is corroding; (b) classification of corrosion activity of steel according to the percentage of corrosive potentials; and (c) determining the effectiveness of a repair method.

To develop an economical method to classify the condition of bridges, potentials were made on a grid pattern of either 4 ft (1.2 m) or 2 ft (0.6 m) and on a random basis. The random selection method of obtaining potentials was based on obtaining the measurements in the curb area of lowest elevation at spacings of approximately 4 ft (1.2 m) longitudinally, and a minimum of 30 values was required.

A complete potential survey was made of 5 bridges before and after concrete repairs (Table 4). The repairs were made only at locations of delaminated concrete.

There appeared to be no significant difference in the percentage of corrosive potentials when a potential measurement spacing was either 4 ft (1.2 m) or 2 ft (0.6 m). However, when smaller intervals were used and results were plotted on equipotential contour maps, there was better definition of corroding areas.

With regard to evaluating the corrosion activity of the decks as determined by the percentage of corrosive potentials (percent of values numerically greater than -0.35 volts CSE), the overall average of corrosive potentials computed either by the complete or random survey for all structures was 19 percent. However, as will be noted in Table 4, the random survey did not detect any corrosive potentials on 2 structures (Canyon Creek and Sawmill) whereas the complete survey did detect corrosion activity. In one case (Sawmill), the amount of concrete delamination of the deck was 0.4 percent; in the other case, the bridge had 6.7 percent delaminated area. It is obvious that the random survey, although a rapid system for evaluating corrosion activity as compared to the time involved in obtaining a complete potential record, will not be perfect. Discrepancies can be minimized in the random type of survey by obtaining potential values in areas of delamination.

As previously mentioned, the percentage of corrosive potentials were determined for 5 bridge decks before and after repairs. From these data it is shown that, after repairs were made, there was a reduction in the percentage of corrosive potentials by an average of about 50 percent. Therefore, this type of repair is basically a mechanical repair that can initially reduce but not prevent or control additional corrosion of the steel.

DISCUSSION

The corrosion of steel in concrete is a dynamic process. There are continual replenishment of oxygen; conversion of iron to its final form of rust; polarization effects;

Table 2. Concrete sampling variations.

Bridge	Sample Type	Depth 0-1 in. n	$\overline{X}$	Std. Dev.	Coef. Var. (percent)	1-2 in. n	$\overline{X}$	Std. Dev.	Coef. Var. (percent)
Lebec	3-in. cores	6	9.74	3.11	31.9	6	4.54	2.50	55.1
	All drillings	24	8.00	1.98	24.8	24	5.42	1.74	32.1
	¾-in. drillings	4	10.2	3.53	34.6	4	6.00	1.18	19.7
	1-in. drillings	20	7.56	1.24	16.4	20	5.30	1.83	34.5
Cressy	3-in. cores	3	1.60	0.80	50.0	3	0.93	0.61	65.6
	¾-in. drillings	12	1.83	1.20	65.6	12	1.12	0.72	64.3
Lebec Road	3-in. cores	6	2.60	0.92	35.4	6	1.20	0.62	51.7
	¾-in. drillings	24	2.73	0.87	31.9	24	1.40	0.80	57.1
Ft. Tejon	3-in. cores	6	3.93	0.99	25.2	6	1.67	0.39	23.4
	¾-in. drillings	24	3.77	1.31	34.7	24	2.19	0.77	35.2
Grapevine, Right	3-in. cores	6	1.37	0.23	16.8	6	0.73	0.16	21.9
	¾-in. drillings	24	1.44	0.21	14.6	24	1.04	0.08	7.7
Grapevine, Left	3-in. cores	6	1.13	0.48	42.5	6	0.70	0.11	15.7
	All drillings	24	1.27	0.21	16.5	24	1.03	0.11	10.7
	¾-in. drillings	16	1.20	0.21	17.5	16	1.01	0.09	8.9
	1-in. drillings	8	1.40	0.15	10.7	8	1.07	0.15	14.0

Note: Chloride content in pounds per cubic yard; 1 lb/yd^3 = 0.59 kg/m^3.

Table 3. Potentials and delaminations.

Bridge	Delaminated Concrete: Maximum Potential (volts) n	Mean	Std. Dev.	Minimum Potential (volts) n	Mean	Std. Dev.	Isolated Potentials[a] (volts) n	Mean	Std. Dev.	Nondelaminated Concrete: All Potentials (volts) n	Mean	Std. Dev.	Percent Delaminated Concrete
Canyon Creek	6	0.390	0.049	6	0.247	0.106	4	0.278	0.064	1742	0.096	0.094	5.5
Gray Creek	26	0.462	0.059	26	0.343	0.068	80	0.356	0.100	376	0.180	0.010	6.7
South Mt. Shasta	10	0.523	0.076	10	0.429	0.091	97	0.466	0.066	391	0.382	0.084	6.7
Lake Street	25	0.496	0.036	25	0.383	0.044	107	0.388	0.070	377	0.227	0.093	10.0
Pony Bar	22	0.381	0.074	22	0.247	0.090	86	0.314	0.074	198	0.024	0.104	30.0
North Mt. Shasta	2	0.50	0.20	2	0.41	0.15	48	0.420	0.095	609	0.263	0.095	1.3
Sawmill	2	0.43	0.08	2	0.36	0.06	42	0.383	0.044	592	0.231	0.100	0.4
Weimar	1	0.34	—	1	0.23	—	10	0.29	0.062	392	0.181	0.071	0.1
Weighted average	94	0.453	0.060	94	0.334	0.072	474	0.385	0.075	4677	0.180	0.086	

[a]Isolated potentials signify that only one measurement was made in localized area.

Table 4. Potential measurements.

Bridge	Before Repair: Full Survey Spacing[a] (ft)	n[b]	Percent Corrosive[c]	Random Survey n	Percent Corrosive	After Repair: Full Survey Spacing (ft)	n	Percent Corrosive
Gray Creek	4	454	26	56	14	4	453	16
South Mt. Shasta	4	459	73	99	82	4	460	37
North Mt. Shasta	4	630	21	148	32	4	630	6
Lake Street	4	477	47	49	45	4	477	22
Canyon Creek	4	478	4	—	—	4	482	2
Canyon Creek	2	1802	3	99	0	—	—	—
Sawmill	4	607	10	30	0	—	—	—
Milagra	2	1041	7	180	5	—	—	—
Pony Bar	4	342	35	49	41	—	—	—
Long Valley	4	142	0	30	0	—	—	—
Weimar, Right	4	380	3	90	9	—	—	—
Weimar, Right	2	1424	4	—	—	—	—	—
Weimar, Left	4	389	5	90	9	—	—	—
New England Mills, Right	4	466	4	90	3	—	—	—
New England Mills, Left	4	465	7	90	12	—	—	—

[a]Minimum center-to-center spacing; 4 ft = 1.2 m, 2 ft = 0.6 m.
[b]Number of observations.
[c]Percent of potentials numerically greater than -0.35 volts.

variations in the half-cell potential due to oxygen, chloride, and hydrogen ion concentrations; and variations in the moisture content of the concrete that affect its resistivity and ability to act as an electrolyte.

If these facts are ignored, then the interpretation of the influence of particular variables, such as half-cell potential values and chloride content of the concrete, can lead to erroneous conclusions. For example, when the chloride-ion content of the concrete is greater than, say, 1.0 lb/yd^3 (0.59 kg/m^3), there is no reason to believe that there always is an automatic and irrevocable start of the corrosion process. A corrosion threshold of amount of chloride is only a point in the concentration where corrosion can begin. This is emphasized by previous work (9, 10) where it was demonstrated that corrosion activity was nil when the specific electrical resistance was greater than 60,000 ohm-cm in salt-contaminated concrete (10). However, it was also demonstrated that when nondistressed salt-contaminated concrete was painted, there were electrical potential (18) and visual (10) indications that corrosion was accelerated. In addition, evaluations of electrical potential measurements have indicated that there is corrosion activity of steel in concrete when corroding bridge decks are overlaid with concrete or waterproof membrane (4, 5).

When suitable data are available, the economics of bridge deck repair or corrosion prevention with procedures such as epoxy injection to bond the delamination (19, 20), concrete removal and replacement (4, 5, 6, 8, 11), cathodic protection (19), or the use of waterproof membranes (4, 5, 6, 10) can be determined.

SUMMARY AND CONCLUSIONS

Chloride, Potentials, and Delaminations

The quantity of chlorides in concrete associated with the incidence of active corrosion of the steel is about 1 lb/yd^3 (0.59 kg/m^3) of concrete.

It was observed that the maximum quantity of chloride at the 95 percent confidence limits found at the average level of the steel was the best indication of salt content in a structure that is causing corrosion of the steel.

Except for isolated cases, the data indicate that the chloride content need not be determined if more than about 1 percent of the surface area of the bridge deck concrete is delaminated or if more than about 10 percent of the total potential measurements are numerically greater than -0.35 volts to the saturated copper-copper sulfate half-cell (CSE).

The data also indicate that if the average chloride content at the level of the steel is greater than about 1.0 lb/yd^3 (0.59 kg/m^3), an analysis to determine the maximum statistical amount may have no practical significance because the chloride content is already too great.

Sampling for Chloride Determination

On 6 bridges it was observed that it did not seem to make any significant difference in the accuracy of the chloride determination whether the concrete sample was obtained by coring or by drilling. This would indicate that the major variable in concrete sampling is controlled by the variation in the salt content per se. In this respect, it may be that the variations in salt content are controlled by concrete properties, salting and snow-removal practices, drainage, etc., but the size of the sampling apparatus as used in this investigation demonstrates that this latter variable, per se, is not significant. Therefore, it is obvious that concrete samples for chloride analysis could be obtained by recovering drilling dust or by cutting and pulverizing concrete cores.

With a coefficient of variation of the chloride contents found to be in the average range of 33 to 36 percent, no less than 6 samples for chloride analysis should be obtained to make a valid survey.

Potentials and Concrete Condition

The average potential of the steel in nondelaminated concrete was found to be -0.180 volts saturated copper-copper sulfate half-cell (CSE). For delaminated concrete, the average potential in small isolated areas was -0.385 volts whereas the average of the minimum and maximum values found in large corroding areas was -0.334 and -0.453 volts CSE respectively. These values confirm that there is great assurance of active corrosion when the potential of the steel is numerically greater than -0.35 volts CSE.

The determination of active corrosion at half-cell potential values numerically less than -0.35 volts CSE will require interpretation and consideration of evidence, such as corrosion-caused concrete delaminations being present, or plotted equipotential contours that indicate an anodic or corrosion area at a lesser maximum potential.

Potentials, Random Survey

It was found that a random survey of the electrical half-cell potential could be a rapid and economical means for evaluating the corrosion activity of the steel in numerous bridges. This method has limitations, as has any other statistical sampling, but it appears to be economically worthwhile where a rapid evaluation of numerous structures is concerned.

Potentials, Before and After Repair

From potential measurements made on 5 bridge decks both before and after the repair of concrete delaminations, it was found that the percentage of corrosive potentials was reduced by about one-half. Therefore, repair of delaminations in chloride-contaminated concrete is a mechanical type of repair that can initially reduce some but not all locations of corrosion activity.

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JOINT EFFORT TO IMPLEMENT RESTRICTED PERFORMANCE SPECIFICATIONS IN PENNSYLVANIA

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One of the key elements in the Pennsylvania Department of Transportation program for the implementation of statistically based restricted performance specifications for highway construction material is concerned with the necessary education of department and industry personnel. This paper discusses the experiences of Pennsylvania Department of Transportation and Pennsylvania State University in the past 2 years in their joint effort to provide the first phase of the training required. The background information for the series of training courses and for the development of a quality-control manual, the guidelines that were set up for the courses, and the objectives of each course session are discussed. The experiences and insight gained as a result of the first 5 courses in the series are discussed by relating some of the observations that participants made about the course itself and also about the proposed program of implementation that Pennsylvania Department of Transportation will follow. The objective of the paper is to share these experiences with other state departments of transportation that are at similar stages in the implementation of these types of specifications.

•IT is generally acknowledged that statistical quality control (SQC) had its origins in the work of Shewart, Dodge, and Romig at the Bell Telephone Laboratories in the 1920s. The concepts and tools resulting from their work have been applied in industry since that time. It appears, however, that people within the highway construction industry did not begin to seriously consider SQC until the problems arising from construction control during the AASHO Road Test indicated the need for a reevaluation of existing control procedures.

In May 1966, a National Conference on Statistical Quality Control Methodology in Highway and Airfield Construction was held at the University of Virginia in an attempt to summarize existing knowledge and provide a forum for examining and discussing the techniques and implications of statistical procedures within the industry. The proceedings of the conference indicate that many excellent papers were presented.

The last section of the proceedings presented 7 papers under the general title, "Implications of Statistical Methods". It is significant to note that little mention was made in these papers (or throughout the conference, for that matter) of an area that the writer feels is one of the keys to the successful implementation of SQC in the highway construction industry. Wescott perhaps came closest to this key when he stated:

> It has often been said that quality control is people, and this is, indeed, a fundamental truth. Certain it is that quality control is not just statistics.... The strategies are there—control charts, frequency distribution analysis... but these strategies, however potentially effective they may be, become sterile—and indeed damaging—in the hands of amateurs or in an environment of suspicion, misunderstanding, or outright opposition. If the quality control function is to be

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sharpened and modernized by the advent of modern statistical science as an integral part of it, there must be a well-planned incubation period during which the infiltration of SQC training of personnel and the gradual development of experience with these methods is given time to take root.

The objective of this paper is to discuss the experiences of the Pennsylvania Department of Transportation and the Pennsylvania State University in the last 2 years in their joint effort to provide for the important key of education that is required if restricted performance specifications are to be successfully implemented. It is hoped that the experiences presented here will serve as models for those in other states faced with the same situation.

COURSE BACKGROUND

The Pennsylvania Department of Transportation began the program of implementing "restricted performance specifications" (also referred to as "statistically oriented end-result specifications") in 1961 when a random sampling system was instituted. In 1967 the department required contractor technicians to control the quality of portland cement concrete and bituminous concrete, and in 1970 the department started gathering data for the development of a bituminous concrete specification. The first "bituminous" projects were constructed under this specification in 1974.

The implementation work plan (1, p. 1.22) indicates a projected completion date of 1980 for full implementation. It was recognized during the early stages in the development of this work plan that the total concept of switching from the present acceptance system to the statistical approach required extensive training and reeducation of both department material inspection and construction forces and contractor and material supplier personnel. The task was recognized as being extensive, and it was decided that it should be approached in an orderly manner so there would be a complete understanding by all concerned and a smooth transition into the new system.

In the spring of 1973, the writer was requested by PennDOT to prepare a program that would provide the training needed for Phase 1 of PennDOT's "Implementation Work Plan". This "basic statistical training" phase was viewed as the required first step to ensure that a proper foundation of understanding would be provided for all PennDOT personnel who were to be involved in the use of restricted performance specifications.

PennDOT expressed an interest in presenting the training in an environment conducive to learning, one that would take the personnel away from the day-to-day problems they would encounter if the training occurred at the local district level. The writer therefore proposed that the ideal location for the training would be at the continuing education facilities available at the Pennsylvania State University main campus. The writer proposed that Phase 1 training could best be handled by presenting a series of 8 "short courses", each of 5 days' duration, for 240 PennDOT personnel in 1974.

Work was begun on the project in July 1973, and the first course was presented during the week of February 18-22, 1974. It was the writer's opinion that one of the most important characteristics of such a series of courses is that the material be presented in a uniform fashion. It was felt that this could best be achieved by using the same instructional manual for all of the courses. Although many textbooks in the general area of statistical quality control have been written, none were available that were specifically designed for the highway construction industry. Although a considerable amount of research had been completed that defined SQC in highway construction, it appeared that no one has attempted to distill this information into a concise package that could be used for instructional purposes. In light of this situation it was proposed that a manual on SQC, geared toward the specific needs of PennDOT highway construction, be prepared for use in the series of short courses. This suggestion was approved by PennDOT.

The various parts of the paper that follow (a) explain the guidelines used in the development of the course, (b) present an outline of the course sessions (which is also an outline of the course manual), (c) discuss the observations of the course made by the students who attended, and (d) highlight some of the observations about the implementation of restricted performance specifications that were made on the last day of each session.

GUIDELINES FOR COURSE DEVELOPMENT

A number of guidelines were developed during the early stages of planning as well as during the period when the first 5 courses were given that greatly influenced the mode of operation and the success of the course. This section of the paper highlights some of these to provide some background and understanding of the course outline.

General Guidelines

A number of guidelines were general in nature and are therefore mentioned first:

1. As noted previously, PennDOT already had a restricted performance specification for bituminous concrete (as a special provision to their Form 408 Specification) at the time the course was being planned. This was considered to be the model for the future material specifications to be implemented. The overall guiding principle in the selection of session topics was therefore to provide the student with (a) an understanding of the theory that formed the basis for the various parts of this specification and (b) a working knowledge of the techniques needed in order to implement the specifications.
2. It was recognized by PennDOT that considerable benefits could be obtained by having a student mix in each course that included not only members of PennDOT's Bureaus of Construction, Design, and Materials, Testing and Research but also members of the contracting and material supplier groups. PennDOT therefore underwrote the cost of course development but agreed to allow 10 industry people from the Associated Pennsylvania Constructors, the Pennsylvania Ready Mix Concrete Association, the Pennsylvania Sand and Gravel Association, and the Pennsylvania Asphalt Paving Association to attend each course. It is felt that this common training and interaction opportunity for all parties will make future implementation of these specifications much easier.
3. Once tentative agreement on the session topics had been accomplished, it was recognized that certain of these were theoretical in nature whereas others involved opportunities for PennDOT philosophy and policy to be presented. It was therefore decided that a neutral party (university faculty) should present the former topics while PennDOT personnel should present the latter ones. The "faculty" selected consisted of the writer, a graduate student, and 3 members of PennDOT's Bureau of Materials, Testing and Research staff. Each party served as a "devil's advocate" for the others throughout the week in order to emphasize key points.
4. It was considered extremely important to obtain feedback from the students after they had taken the course. The first reason for this was to determine if the material had been presented in a fashion that most people understood. The second and perhaps more important reason was that this feedback would provide the personnel who were responsible for specification development with 240 "educated" responses from field people about the system, its shortcomings, loopholes, and potential problem areas from an implementation standpoint. Accordingly, all of Friday morning was essentially designed to be a forum feedback session. Some of the valuable points raised during these sessions have resulted in PennDOT's reassessment of parts of the specification.
5. The final general guideline was that the courses should be split, with half of them being presented in the spring of 1974 and the other half in the fall. It was recognized that the lecture notes would have to be revised at least once, based on the experiences gained in the early courses. It was decided that the rewriting would be done during the summer. The final course manual that resulted was issued in 2 volumes (1, 2).

Guidelines for Course Operation

There were some guidelines concerning details of course operation that had a great influence on the course. Some of these are discussed in the following.

1. It was decided that each student should be given a complete set of typed notes at the beginning of each session. It was recognized that most of the students would have been away from academic life for a long time, and hence the requirement for taking lecture notes should be kept to a minimum. Each instructor attempted to follow the notes as closely as possible, and extensive use was made of overhead transparencies of the notes to give the student a chance to read along as the lecture proceeded.
2. An attempt was made to keep the material as practical as possible with a reduction of mathematical complexity, because it was recognized that the mix of students would consist of those who had recently completed master's degree work to those who had been out of high school for 20 years. At the same time, however, it was recognized that if the objective of the course was to merely present "rules of thumb" related to the bituminous specification the course could have been given in 1 or 2 days. This was, however, not the objective, so the sessions were designed to provide an understanding of the underlying theory as well as training in the statistical techniques required. It should be noted that the latter material was reinforced by workshops interspersed throughout the week.
3. It was decided that a certain amount of homework would be given throughout the week. Aside from the occasional grumbling observed, all of the instructors were impressed by the attitude of the students regarding the homework assignments. It was felt that, unless the students were required to review the session notes each night because of a homework requirement, they would not gain the understanding required. The problem with all compressed-time courses is that there is not enough "recovery time" for the student to assimilate knowledge before another topic is covered. It was felt that the homework assignments partially compensated for this problem.
4. It was recognized that once the student understood the basic calculations he should also be provided with a system in the field that would reduce the required paperwork. Accordingly, in parallel with the development of the course, PennDOT's staff developed a statistical package of computer programs that could be utilized by people in all districts of PennDOT to aid them in their application of the techniques that had been discussed.
5. One of the major points in the course was the responsibility for process control that the contractor and/or material supplier had under the provisions of the specification. To provide some guidelines for the industry people in the last few courses of the 8-course sequence, the graduate student working with the writer was assigned the task of working with a bituminous producer during the summer between courses to develop a workable process-control system that would supplement the "Suggested Guidelines for Process Control" available in the specifications.

COURSE OUTLINE

The previous section of this paper presented the guidelines used to develop the course. The first 5 courses resulted in several changes in the original topics selected and the order in which they were presented. This part of the paper presents the outline that resulted from the evolutionary process and lists the objectives and topics covered in each session. The course sessions, as presently designed, extend from 12:15 p.m. on Monday until 12:00 noon on Friday. A typical day starts at 8:00 a.m. and ends at 5:00 p.m. The order of the sessions follows.

Session 1: PennDOT Overview (Monday 12:15-1:15 p.m.), J. Moulthrop, PennDOT

The objective of this session is to inform the student of how the material to be covered during the course relates to the current and future plans of PennDOT with regard to restricted performance specifications. The speaker is from the PennDOT Bureau of Materials, Testing and Research and is at an organizational level that provides him with an overall perspective of the topic. In the first part of the session the appropriate definitions for statistical quality control are presented. The student is made aware of the fact that 3 distinct levels of testing will be required if the quality-control system envisioned by PennDOT is to be workable.

The student is informed of the changes that will occur as these new specifications are implemented through a brief review of the current practices and problems connected with (a) the different levels of control a PennDOT inspector presently possesses (depending on the type of material) and how this often results in a conflict between process control and inspection; (b) the different types of specifications that are currently used (100 percent compliance, satisfaction of engineer, substantial compliance) and the problems involved with each type; and (c) the current concept of sampling, which is based on the single-representative-sample philosophy.

The student is then briefly introduced to some aspects of the new approach that he will learn in the next few days. The concepts of specifications recognizing variability, the relationship of specification limits to sample size, etc., are introduced. The 3 types of sampling and testing that are the framework for the specification and the responsibilities connected with each type are then discussed in greater depth.

The final part of the session covers the plans for implementation that PennDOT has developed. The concept of a deliberate approach to implementation, the development of specifications for each area in an orderly fashion, and an explanation of the plans for training at each step in the process give the student the proper overview of the subject before he begins to get involved with the various technical topics that will be covered.

Session 2: Collection and Organization of Data (Monday 1:15-2:30 p.m.), J. Willenbrock

The objective of this session is to outline briefly the 4 basic phases of statistical analysis (i.e., collection, organization, analysis, and interpretation of data) and then to discuss the first 2 in more detail. The different types of data collection included in the new specification are discussed. The main thrust of this session, however, involves the presentation of a series of tabular and graphical techniques (i.e., frequency tables, histograms, and polygons) that the student can use in the "organization of data" phase to obtain the maximum amount of significant information from a set of data.

Session 3: Sampling Experiment 1 (Monday 2:30-3:00 p.m.), J. Willenbrock

The objective of this session is to illustrate the results obtained when each student randomly draws a sample of size $n = 1$ from a bowl containing a population of concrete strengths that is slightly skewed to the right. The ungrouped data and a frequency table and histogram for the parent population are first presented. As each student draws a value from the bowl (and replaces it) the result is recorded on the population histogram and in a frequency table that is divided in a fashion similar to the one presented for the population. The point stressed is that 1 sample of size $n = 1$ does not give a very satisfactory indication of the characteristics of the population from which it is drawn. The point is also made that the histogram of a sample of size 40 (40 samples of size $n = 1$) has a shape that is similar to the shape of the histogram for the population.

Session 4: Analysis of Data (Monday 3:15-4:15 p.m.), J. Willenbrock

The objective of this session is to present the formulas required in order to determine the 2 most important characteristics of a set of data, i.e., the central tendency and the dispersion. The student is made aware of the fact at this point that the mathematical complexity of the presentation will be at a level he can understand. It is pointed out that the course is not intended for mathematical statisticians.

The student is introduced to the basic algebraic symbolism that will be used, and then the formulas used to calculate the arithmetic mean and the standard deviation are presented. It is pointed out that these are the 2 indicators of central tendency and dispersion he will most often use to describe the characteristics of data, although the range is sometimes substituted for the latter because of its ease of calculation. In each case, the formulas for both population data and sample data are presented (for both ungrouped and grouped data). The student is also introduced to the coefficient of variation at this point, and the situations where it is applicable are discussed.

Session 5: Workshop 1 (Monday 4:15-5:00 p.m.), J. Willenbrock

The objective of this session is to give the students some experience with the organization and analysis of data. Each student is required to use the results of Sampling Experiment 1 to (a) construct a relative frequency histogram for the data and compare it to the relative frequency histogram of the population; (b) construct a frequency polygon and a cumulative frequency polygon for the grouped data; and (c) calculate the mean and standard deviation of the grouped data.

Session 6: Additional Aspects of Statistical Analysis (Tuesday 8:00-9:45 a.m.), J. Willenbrock

The objective of this session is to (a) develop the relationship between a frequency histogram of a set of data and an idealized smooth curve approximation and (b) indicate the suitability of a normal distribution in light of the PennDOT data presented. A description of the shape of the normal curves, skewed curves, and other less frequently occurring shapes is also presented. A more detailed explanation of the mean, median, and mode is given and their relationship with respect to the type of skewness is explained.

The significance of a bimodal or multimodal distribution, as it relates to statistical quality control of construction materials, is also explained. Some of the points are illustrated by means of the histograms and polygons of sand cone, nuclear, and Proctor test results from the PSU-PennDOT Test Track. A brief explanation of the properties of a normal curve is also given by presenting the "empirical rule" that relates areas under the curve to sigma limits.

Session 7: Sampling Experiment 2 (Tuesday 10:00-10:45 a.m.), J. Willenbrock

The objective of this session is to indicate to the student that the procedure of multiple sampling ($n > 1$) has advantages for both PennDOT and the contractor of material supplier. This objective is achieved by first reviewing the results of Sampling Experiment 1 and indicating that a single sample (of size $n = 1$) may be quite far from the population average and that an individual high result does not indicate any more about the "true average" than an individual low result.

The discussion of Sampling Experiment 1 leads into the idea of multiple sampling. At this point, therefore, Sampling Experiment 2 (where each student draws 4 samples from a bowl and determines an average value to represent all 4) is performed. The student is informed that there is some sound theoretical basis for multiple sampling by mentioning the concept of the sampling distribution of the means and the central limit theorem.

Session 8: Workshop 2 (Tuesday 10:45-11:00 a.m.), J. Willenbrock

The objective of this workshop is to give the student an appreciation for the results of a multiple-sampling ($n > 1$) experiment as well as to provide an additional opportunity for practice with the organization and analysis of data. The student is required to determine the frequency histogram, the mean, and the standard deviation for the data and discuss the implications indicated by the "grouped data" presentation. The objective is to compare the "distribution of the sample means" with the distribution of the population.

Session 9: Additional Aspects of Dispersion (Tuesday 11:00 a.m.-12:00 noon), J. Willenbrock

The objective of this session is to give the student a further appreciation for uses and implications of the term standard deviation. The first item discussed is that standard deviation may be used as a new measuring scale to indicate the difference between 2 numbers. This concept, particularly when the difference is between a given number and the arithmetic mean, is important when the calculation of the percent within limits for the normal distribution is discussed.

The next item covered is the relationship between the standard deviation of a sample and the standard deviation of the population. It is noted that the standard deviation of a sample of size n does not give the exact value of the population standard deviation and that quite often the population standard deviation is an idealized concept that must be estimated as closely as possible from the information from sample data.

The final item discussed in this session is a method, called the "average range method", for estimating a population standard deviation from sample data. It is the method that is used later in the course to establish the control limits for control charts.

Session 10: Normal Distribution (Tuesday 1:00-2:00 p.m.), J. Willenbrock

The objectives of this session are to (a) provide the student with an understanding of the normal distribution and its usefulness as a theoretical distribution that "models" actual construction material data and (b) illustrate the use of this theoretical model for the purpose of calculating the area under a distribution. This leads to the determination of the "percent within limits", a concept that is an extremely important part of the PennDOT acceptance plan procedure.

Session 11: Workshop 3 (Tuesday 2:00-3:45 p.m.), J. Willenbrock

The first part of the session is devoted to a series of progressively more difficult example problems related to the normal distribution. The validity of the "empirical rule" is explained as these example problems are discussed. After the student understands these calculations, the concrete compressive strength data previously presented in session 2 are reexamined to determine the areas under the distribution and whether the "normal" approximation was a valid assumption for this set of data.

Session 12: Normality Test (Tuesday 3:45-4:15 p.m.), J. Willenbrock

This session is devoted to an explanation of the graphical technique (using normal probability paper) for determining if the assumption of "normality" for a set of data is valid. The various "goodness of fit" tests available are discussed, but the one that is emphasized for practical field application is the graphical method. An example problem using grouped data is presented to illustrate how the procedure is used and how it compares with the cumulative frequency plot on conventional graph paper.

Session 13: Distribution of Sample Means (Wednesday 8:00-8:45 a.m.), J. Willenbrock

At this point in the course the student will have been exposed to 2 sampling experiments, and he should understand the difference between a histogram (or distribution) of a population and that of a sample. He should realize that there is probably a relationship between the parent population distribution and the various "distributions of sample means" that are developed from it as the sample size n is changed. Since the acceptance criteria in the PennDOT specifications are written on the basis of $n > 1$, it is important at this point to establish the theoretical basis for the sampling distribution of the means and to discuss the central limit theorem and the standard error of the mean as the underlying principles for the concept of multiple sampling ($n > 1$).

Session 14: Uses of Sample Mean Theory (Wednesday 8:45-10:00 a.m.), J. Willenbrock

The objective of this session is to indicate the applications of the sampling distribution of the means to the statistical quality control of construction materials. The first application is made to hypothesis testing in order to indicate how the calculation of percent within limits, area in the tails, etc., is changed if a decision must be made on the basis of a sample of size $n > 1$.

The second application is in the area of material specification development. Several examples are presented to illustrate how the desired properties and specification limits of the population are transformed into equivalent specification limits based on a specified sample of size $n > 1$. The correlation between specification limits and the size n of the sample is stressed in this session. The relationship is further emphasized by briefly discussing the statistical significance of retesting.

Session 15: Student-t Distribution (Wednesday 10:15-11:00 a.m.), J. Willenbrock

The objective of this session is to indicate what procedure is followed when information about the population must be inferred (statistical inference) from information obtained from a small sample. The standard central Student's t-distribution is presented and its shape is compared to that of the normal distribution as size n of the sample increases. The student is acquainted with the fact that quite often the target value of the population mean is specified and he must perform a percent-within-limits calculation for the population based on the value of the sample mean and standard deviation he obtains from a sample of size $n = 4$ or 5. In this case, since his best estimate of the population standard deviation σ is the sample standard deviation s, it is suggested that Student's t-distribution be used.

Session 16: Sampling for Quality Control (Wednesday 11:00 a.m.-12:00 noon, 1:00-1:30 p.m.), R. Cominsky, PennDOT

The objective of this session is to explain some of the practical factors involved in sampling techniques under the restricted performance type of specification. The first part of the session is devoted to the levels of sampling responsibility that exist under this type of specification. The 3 types of sampling, for process control, acceptance control, and assurance control, and the relationship between the 3 respective parties involved, the contractor and/or material supplier, PennDOT Bureau of Construction, and PennDOT Bureau of Materials, are discussed in detail. The point is made that PennDOT will use a method of stratified random sampling in the acceptance phase of quality control. The use of random sampling tables and PTM-1 (which deals with this type of sampling) is explained, and several examples of random sampling are presented to illustrate the method of locating points on a random basis.

The last part of the session covers the various aspects of the measuring process that must be considered when a sample is obtained. First, the terms precision, reproducibility, and accuracy are discussed. This leads into a discussion of the round-off rules that will be followed under the PennDOT specification.

Session 17: Development of Statistically Based Restricted Performance Specifications (Wednesday 1:30-3:00 p.m.), R. Nicotera, PennDOT

The objective of this session is to provide the student with a background in the various principles underlying restricted performance specifications. The first part of this session introduces the student to the fact that the specifications for construction materials are the framework for the quality-control system. The 2 more common types of specifications, i.e., end-result and material and methods, are compared to PennDOT's restricted performance type. A discussion of the essential elements that are found in all restricted performance specifications covers items such as (a) the levels of quality-control responsibility, (b) the materials characteristics that will be tested, (c) the location of a sample, (d) the definition of the size of the lot and sublot, (e) the definition of a sample, (f) the definition of the method of test, (g) the establishment of limits of acceptance, (h) the development of the ground rules for acceptance determination, and (i) the existence of a reduced-payment provision for noncompliance.

The final part of this session is devoted to an explanation of how the various components of variance for a material characteristic are established so that realistic limits of acceptance for the material can be established. The various components of the overall variation of the material characteristic are first identified, and the role they play in variation is established. The need for a planned experiment that allows for an analysis of variance (ANOVA) is discussed.

Session 18: Development of Acceptance Plans (Wednesday 3:15-5:00 p.m.), R. Cominsky, PennDOT

One of the primary parts of PennDOT's restricted performance specification is the acceptance plan. This plan defines the procedure that will be used to determine the characteristics of the construction materials as they are estimated from the results of a small sample. The objective of this session is to give the student an understanding of the underlying principles used to develop such an acceptance plan.

The first part of the session is devoted to explaining the concept of acceptance testing. The different types of acceptance plans are discussed, and the student is informed that PennDOT will mainly use acceptance plans for variables based on controlling the percent within limits. The different types of risks (i.e., α, β) involved in statistical decision-making are explained, and the role played by operating characteristic curves in the development of acceptance plans is discussed.

The final part of this session is devoted to an explanation of the parts of Mil. Std. 414 that affect PennDOT's acceptance plans. The student is given a brief explanation of the important parts of Mil. Std. 414 and is further informed that the range approach to variables sampling based on percent within limits is the one that PennDOT has adopted. This method is discussed in detail. The various parameters (i.e., Q_U, Q_L, etc.) are defined and the associated tables for this method are presented. This final part of the session should bridge the gap between calculating percent within limits with a normal curve assumption and calculating the same factor by using the tables in Mil. Std. 414.

Session 19: Review of Sessions and Homework (Thursday 8:00-8:30 a.m.), R. Nicotera, PennDOT

The objective of this session is to tie together some of the concepts covered on Wednesday by reviewing the sessions and the assigned homework problems.

Session 20: PennDOT's Restricted Performance Bituminous Specifications (Thursday 8:30-10:00 a.m.), R. Nicotera, PennDOT

One of the objectives of this session is to present PennDOT's restricted performance bituminous specification (currently a special provision to the Form 408 Specification) as an example of the format that a statistically based specification will have. The second objective is to provide the student with a working understanding of all parts of the specification in light of the statistical background acquired during the week.

The first part of the session covers the changes that have been made in the specification. Some of the important changes stressed are as follows:

1. The PennDOT construction engineer is no longer required to use only his own opinion when judging the acceptability of material, since he now has a set of specification limits and an acceptance plan to aid him in decision-making.
2. The contractor must be operating with an approved quality-control system guiding his process-control activities.
3. The acceptance rules, tables, and formulas will be used to determine the percent within limits of a particular characteristic as well as the related adjusted payment.

It is pointed out that the concept of an approved job mix formula still applies in this specification. A set of tolerances for aggregate gradation and mix temperature are provided in the specification as process control criteria. It is noted that the acceptance criterion for the material at the plant will be the bitumen content based on the average of 5 tests taken on a random basis within a lot. An "adjustment of contract price" table is also presented in conjunction with the formula for determining the percentage of material within the tolerance limits. In addition, the material will be accepted in the field based on a density criterion, the target value of which will be determined on the basis of a control strip concept. An adjustment of contract price table based on the average and range of 5 density tests from each lot is also presented for this acceptance criterion.

The final part of the session involves a discussion of the problems connected with outliers and develops a procedure for dealing with them as well as presenting the suggested Guidelines for a Contractor's Quality Control System. The point is emphasized that the contractor must present a quality-control system for approval that is at least equal in scope to the one discussed.

Session 21: Workshop 4 (Thursday 10:15 a.m.-12:00 noon), R. Nicotera, PennDOT

The objective of this session is to provide the student with some exposure to the type of calculations that will be required by the specification for acceptance based on bitumen content, density, etc. This is accomplished with a practicum session covering (a) problems related to acceptance calculations for bitumen content; (b) problems related to acceptance calculations for density; (c) determination of reduced payment for a typical project; (d) problems related to outliers; and (e) problems involving the establishment of a process-control system for a contractor.

Session 22: Control Charts and Contractor-Supplier Quality-Control Systems (Thursday 1:00-2:45 p.m.), J. Willenbrock

The objective of this session is to introduce the concept of control charts as a process-control technique and to develop the equations necessary to implement the method. The first item discussed is the purpose of control charts and the need to differentiate between "chance causes and assignable causes" in the day-to-day control of a process. It is noted that the drawback to using samples of size $n = 1$ is that assignable causes cannot be easily identified.

The control chart equations for the target value and the upper and lower control limits of the $\overline{X}$ and $\overline{R}$ charts are developed for the cases where (a) the population mean and standard deviation are assumed known or given by the material specification and (b) the population mean and standard deviation are assumed unknown. The use of tabulated factors for the various constants is indicated in the development of the equations. After these equations are developed the student is given an understanding of the pattern of data points he might expect to find in a control chart as various external factors influence the process.

Session 23: Control Chart Applications (Thursday 3:00-3:45 p.m.), J. Marcin, Penn State

The objective of this session is to illustrate the use of the control chart technique in an actual situation that occurred on a PennDOT project. The data used are for Type 2A aggregate that was used for subbase material. The contractor had collected aggregate gradation data over a 3-month period and had randomly sampled twice during each 4-hour period.

Session 24: Workshop 5 (Thursday 3:45-4:00 p.m.), J. Marcin, Penn State

The objective of this session is to give students the opportunity to develop the equations for the control charts for the same data presented in Session 23 if the subgroup is changed. This problem is finished as a homework assignment.

Session 25: PennDOT Computer Programs (Thursday 4:00-5:00 p.m.), R. Nicotera, PennDOT

The objective of this session is to explain how the student can utilize the computer programs PennDOT has developed to simplify some of the statistical calculations required. The first program discussed is "Data Summary", which may be used to organize and analyze the variance, standard deviation, coefficient of variation, and skewness of the data set as well as to plot the histogram.

The second program discussed is "ANOVA", which may be used by someone from

PennDOT or a contractor or material supplier to perform an analysis of variance on a given material characteristic. It establishes each of the components of variance based on the data determined from a designed experiment that is explained in the synopsis for the program.

The third program discussed is "Control Chart". It may be used to plot the $\overline{X}$ and $\overline{R}$ control charts for a given set of process-control data. The program determines the estimate of the population central tendency and dispersion and then presents a control chart printout using these parameters to develop the target value and upper and lower control limits.

Session 26: Review of Sessions and Problems (Friday 8:00-8:30 a.m.), J. Marcin, Penn State

The objective of this session is to review the material covered on Thursday in relation to PennDOT's bituminous restricted performance specification and the control chart technique. Questions are answered and the results of the homework assignment in control charts are discussed.

Session 27: Participant Feedback Session (Friday 8:30-9:45 a.m.), J. Willenbrock

The objective of this session is to determine what the students felt was the most valuable information they obtained from the course. This objective is met by first allowing each student to verbalize the points he felt were most valuable, would be the hardest to implement, caused the most confusion, etc. These comments are written on the blackboard to provide everyone with a review of the course. After this phase is completed each student is asked to fill out a questionnaire that explains his observations about the course in more detail.

The information provided is extremely helpful in determining (a) improvements that must be made in the course in order to improve general understanding, (b) potential areas of confusion that may arise when field personnel attempt to implement some of the statistical concepts, and (c) revisions that should be made to the specifications in order to eliminate the confusion.

Session 28: PennDOT Plans for Implementation (Friday 10:00-10:15 a.m.), J. Moulthrop, PennDOT

In this session, PennDOT's plans for implementation of restricted performance specifications are again reviewed. The students now have a much better understanding of the items discussed than they had when the PennDOT overview was presented on Monday. This session also stimulates some questions that can be raised during Session 29.

Session 29: Panel and Open Discussion (Friday 10:15 a.m.-12:00 noon), J. Moulthrop, PennDOT

The objective of this session is to provide a forum where questions related to any material covered during the week will be answered. A panel is assembled and usually consists of representatives from (a) PennDOT's Bureau of Construction, (b) PennDOT's Bureau of Materials, Testing and Research, (c) a construction company, (d) a material supplier, and (e) a federal agency. The panel members are to be drawn from the class if possible and really serve only as a focusing point for questions from the floor.

OBSERVATIONS ABOUT THE COURSE

In general it may be stated that the course was well received by the people who attended. Almost all of the participants approached the educational experience in a businesslike fashion, attempted to do the homework and keep up with the session notes, and realized that it was necessary to understand the material in order to properly carry out their day-to-day activities. When the diversity of mathematical and statistical expertise of the participants is recognized it may be stated that all of them received a much better understanding of restricted performance specifications, although some expressed a need for more time and workshop experience in order to understand how to handle all of the calculations.

It is understandable, therefore, that quite a number of the participants felt that the course should have been longer than 5 days whereas only a few felt the period of time should be reduced. It is the writer's opinion, therefore, that if the objectives of a course like this one are to go somewhat beyond the rules-of-thumb approach a minimum duration for the course should be 5 days. Although several comments were expressed that the homework assignments were burdensome and reduced the amount of free time the people had, there were also comments suggesting that one of the best features of the course was the need to review the material because of the homework assignments.

One need that was mentioned a number of times and that subsequently was added to the course was a glossary of terms and a summary of equations and symbols. Many concepts, terms, and equations were presented at a fairly rapid pace with very little time for reflection. This is the perfect set of conditions for confusion. It is felt that the presentation of such a glossary is one method of alleviating this problem.

There was a recognition at the beginning of the planning session that not all of the statistical topics could be covered in sufficient detail in a first course such as this. Trade-offs were therefore made, and areas such as hypothesis testing, confidence limits, and statistical decision theory related to risks and acceptance plans were not covered in sufficient detail. Even when this was done, some comments were made that more time should have been spent on practical examples, applications to practice, and problem-solving in workshop sessions. Perhaps the answer to this dilemma involves the inclusion of some of the additional statistical material in a second-level course that concentrates more on one particular type of construction material.

In addition, it must be recognized that this was a course primarily intended to train PennDOT personnel. If a course were given strictly for industry personnel, greater emphasis would have been placed on techniques for process control, determination of the actual risk levels implied by the specifications, etc. There were a number of comments expressed by industry people about the shortcomings of the course in this regard.

OBSERVATIONS ABOUT THE IMPLEMENTATION OF RESTRICTED PERFORMANCE SPECIFICATIONS

Some of the items discussed in the first 5 courses during the panel discussion (Session 29) indicate the types of concerns that were expressed after the students (field personnel who would essentially be directly involved with the specification) had spent a week of intensive study related to statistically based specifications. A few of the major items are given here to indicate the areas that the writer feels merit further investigation if these types of specifications are going to be extensively implemented in Pennsylvania as well as in other states.

1. It was generally felt that the specifications outline in great detail the ground rules that will be followed for acceptance sampling but are not nearly as explicit with respect to process-control requirements and guidelines. Although this might be the result of a preference indicated by some people in the industry (perhaps prior to understanding the concepts of statistical quality control), it was a real concern of many of the PennDOT

and industry people who attended the course. Questions about how a PennDOT construction inspector was to evaluate the process control of a contractor, what the role of the PennDOT inspector was when the contractor claimed his process was being controlled but the inspector found obvious local deficiencies, and what the extent of the process-control system would have to be to ensure that the contractor and/or material supplier would not be penalized at the acceptance stage all indicate that additional information about process control is desirable. It is the writer's opinion that research into this phase of statistical quality control has not kept up with research into the acceptance phase of the system.

2. Many questions revolved around the concept of a penalty clause as it relates to statistical-type specifications. Questions were raised regarding the contractual agreement that must exist between a contractor and his material supplier in light of the penalty system. Was there a need for an acceptance-plan approach at this interface? What would happen if the contractor put very tight requirements on the material supplier and because of the demand for the supplier's product (in the present state of the economy) this caused a termination of their relationship? Other questions revolved around whether a contractor could optimize his profit at the expense of quality by accepting a penalty as part of doing business, whether a bonus was anticipated for satisfactory material if it was within the specification limits, and whether past performance related to a penalty history could be used as a basis for prequalifying contractors.

3. Questions were raised about the ramifications of the concept of multiple sampling in a project as it related to the number of additional inspectors and technicians that would be required, the time required to take each of the tests, etc.

4. Some people also expressed concern about the extent of the training that would be required for this new approach to be understood by the majority of the people in the industry, both those working for PennDOT and those working for contractors and materials suppliers. It was felt that the training that was accomplished in the 8 courses would have to be filtered down in the PennDOT districts to the organizational level that would be directly involved with implementation. It was felt that this need would be partially satisfied if all of those who had received training at Penn State would act as instructors during winter training sessions at the district or company level.

5. Concern was also expressed about how the information flow would be accomplished for all the data that would have to be processed and evaluated for the 3 levels of sampling (i.e., process control, acceptance testing, and assurance sampling) required by the specifications. The assimilation of this information into decision-making at the district level was also viewed as a problem.

6. Questions were also raised about how the new type of specification would affect contractors of various sizes. Concern was expressed that the smaller contractors would suffer and that the producer with an automatic batch plant, for instance, would have an advantage over the one with a conventional plant under this type of specification.

In summary, then, it could be stated that some people felt that not all the details and ramifications of this type of specification had been worked out. It was felt that the entire concept had to be examined and its impact on each facet of the industry had to be evaluated. It was recognized that in a state as large as Pennsylvania the people at the district levels had to be given a system that was fairly well defined to ensure uniformity of approval, particularly at the process-control interface between PennDOT and the contractor.

SUMMARY AND CONCLUSIONS

This paper has attempted to present guidelines of how the educational requirements connected with the implementation of statistically based specifications may be satisfied in those states that are at a stage of implementation comparable to Pennsylvania. The paper has covered the background and guidelines for the series of short courses presented for PennDOT and construction personnel in Pennsylvania by the Department of Civil Engineering of Pennsylvania State University. Included are a summary of the objectives and an outline of each session presented in a typical course. In addition, sections of the paper are devoted to the primary observations of the participants in these courses with respect to the course content as well as PennDOT's plans for implementation.

In conclusion, it is the writer's opinion that PennDOT approached the implementation of these specifications in the correct fashion by recognizing the need for an extensive educational program to ensure that these specifications will be accepted by the people who will use them. It is hoped that other states will also recognize this need.

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The opinions, findings, and conclusions expressed in this publication are those of the author and not necessarily those of the U.S. Department of Transportation or the Commonwealth of Pennsylvania.

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ROLLING STRAIGHTEDGE SAMPLING PLAN SIMULATION AND SPECIFICATION DERIVATION

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The rolling straightedge is a device designed to measure large-scale surface irregularities, thus providing a means for evaluating the riding quality of a pavement. In this paper, it is used as an example to illustrate the derivation of a statistically based specification. A Monte Carlo simulation procedure is used to determine the component of variance attributable to the actual pavement roughness and the manner in which it is sampled. This is combined with the variance associated with the precision of the device to obtain the overall variance of the measurement process. The desired quality levels are selected and appropriate producer's and consumer's risks are chosen to derive the necessary specification acceptance limits for both bituminous and concrete pavement. A table is included that gives the variability associated with several possible sampling plans. Operating characteristic curves are presented that illustrate the capability of the resultant specifications. A discussion of the various ramifications of this method is included to provide assistance to those wishing to apply this general approach to other measurement processes to derive specifications with balanced producer's and consumer's risks.

•TO use a measurement process effectively, it is necessary to know how precise it is. In statistical terms, the variance is a measure of the repeatability and, therefore, the precision of the measurement. When the measurement is to be used as the basis for accepting or rejecting some item of construction (pavement in this example), the variance must be known in order to develop an acceptance procedure that will distribute the producer's and consumer's risks in an equitable manner. Ideally, the producer's risk (rejection of satisfactory work) and consumer's risk (acceptance of unsatisfactory work) should be zero, but this is often impossible or impractical to achieve. Alternatively, it is desired that both risks be equal and as small as possible. This paper describes the determination of the variance for a particular measurement process (rolling straightedge) and the development of a specification that satisfies these risk requirements.

BACKGROUND

The rolling straightedge, shown schematically in Figure 1, is a mechanical device designed to measure large-scale surface irregularities. It is pushed by hand along the pavement and automatically dispenses a dye to mark areas that deviate from a perfectly flat surface by more than some specified amount (usually ⅛ in. within a length of 10 ft). From these dye marks it is possible to record the number of defects per unit length of pavement; measure the total length of the defects per unit length of pavement; or, if the depths or heights of the defects are noted, calculate an integrated value that accounts for both frequency and severity of these deviations.

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The second option, which is termed the "percent defective length" of the pavement, has been recommended for quality assurance applications (1, 2). Correlation tests (1) with a BPR roughometer have demonstrated this to be a valid approach for the determination of pavement riding quality. Straightedge data in terms of the defective length parameter are currently being used for riding quality control in several states and are endorsed by the FHWA (2).

For a typical bituminous pavement in New Jersey, the percent defective length is about 1.0 percent. For New Jersey concrete pavement (expansion joints at regular intervals), the percent defective length may typically be as large as 9.0 percent.

The FHWA has recommended the use of the rolling straightedge for final acceptance of bituminous pavements with a graduated penalty schedule being applied for varying degrees of noncompliance (2). A knowledge of the precision of the measurement process is of obvious importance for this function and is also required for the establishment of the acceptance limits.

STATISTICAL CONCEPTS

There are two primary components of variance to be considered when making measurements with the rolling straightedge. These are the variability related to the precision of the instrument (σ_I^2) and the variability associated with the actual pavement roughness and the manner in which it is sampled (σ_S^2). Using the principle that variances of independent factors are additive, the total variance (σ_T^2) may be expressed as follows:

$$\sigma_T^2 = \sigma_I^2 + \sigma_S^2$$

In order to determine σ_T^2, it is first necessary to find σ_I^2 and σ_S^2. Of these, σ_I^2 is the easier to obtain since it may be calculated from several repeat readings on the same section of pavement. Strictly speaking, the value of σ_I^2 determined in this manner may contain a small component of variance associated with the roughness of the pavement. This is so because, if the operator strays off the intended line of travel, the pavement surface at that point may be of a slightly different roughness level and may produce a different (and thus more variable) reading. Since it is relatively easy to guide the rolling straightedge along the desired line, it is believed that this "pavement component" is a very small part of σ_I^2. For the purposes of this study, it can be ignored because the repeat runs are typical of those made when evaluating an actual job. Therefore, whatever value of σ_I^2 is obtained can be expected to apply when future jobs are evaluated.

A previous study has shown that the standard deviation associated with the precision of the instrument is influenced to a small degree by the general roughness level of the pavement. On bituminous pavement, which is comparatively smooth, the instrument standard deviation was found to be approximately $\sigma_I = 0.30$ percent defective length. On concrete pavement, which is substantially rougher than bituminous pavement, a typical value for the instrument standard deviation is $\sigma_I = 0.40$ percent defective length.

DETERMINATION OF SAMPLING PLAN VARIANCE

Determination of the component of variance associated with the sampling plan (σ_S^2) is more involved, due in part to the several possible sampling plans that might be used. If 100 percent sampling were employed, there would be no sampling error (i.e., error due solely to fractional sampling) and the total variance would consist only of the variance due to the precision of the instrument. However, depending on the availability of both equipment and manpower, it is probably neither practical nor necessary to require 100 percent sampling.

Various fractional sampling plans must be explored to determine which is the most appropriate. In the reference cited, the FHWA recommends continuous longitudinal sampling with the provision that the transverse location be chosen at random every 300 ft. Since it is our practice to make rolling straightedge measurements only at the approximate locations of the wheelpaths, there are then 2 possible transverse locations per lane that may be randomly selected. For pavement 2 lanes wide, this plan would result in a sampling fraction of 25 percent, since 1 of 4 possible wheelpaths would continuously be sampled. For the purposes of this study, sampling rates of 50 percent and 12.5 percent will also be investigated.

A prohibitive amount of work would have been required to perform these tests on actual pavement with the rolling straightedge. If this method had been used, approximately 200 man-days would have been required to obtain the data for the 8 tests listed in Table 1. By comparison, it required only 3 man-days to simulate these tests using Monte Carlo techniques. This illustrates the tremendous savings in both time and expense that can be realized by the use of Monte Carlo simulation.

Plots of the locations of defects on several New Jersey pavement sections proved to be especially useful for the simulation procedure. A typical plot is shown in Figure 2. The 4 closely drawn parallel lines represent the 4 wheelpaths of 2 adjacent lanes. Each run was ¼ mile long and was plotted as four 330-ft sections, with the stations indicated throughout each section. The defects were measured and plotted to the nearest foot, with no distinction being made between high and low readings.

These ¼-mile plots thus represent roadways of known roughness. To test any sampling plan, the locations for sampling are determined as prescribed for that particular plan, and the number of defects is counted directly from the plot. Since it is possible to count the defects exactly, this procedure excludes instrument error and isolates the variance due solely to the sampling plan. Each sampling plan was repeated enough times so that a reliable determination of this variance could be made.

The particular roughness plots chosen for this study were selected so that several average levels of roughness would be represented. Each is a graphical representation of a section of an actual pavement in New Jersey judged to be reasonably typical of its type, whether bituminous or concrete. This selection was subjective but was based on the judgment of experienced engineers who have made roughness measurements on many miles of pavement within the state.

Although the FHWA report recommends that the wheelpath be randomly chosen at 300-ft intervals, this was approximated by using 330-ft intervals because it greatly simplified the use of the plot shown in Figure 2. A starting station was selected, a ruler was placed on the plot at this station perpendicular to the lines depicting the wheelpaths, and the defects were counted along the randomly selected wheelpaths using the ruler to mark the starting and stopping points. This use of 330-ft intervals greatly speeded the gathering of data and was assumed to have no significant effect on the results.

There are several methods by which the starting stations and wheelpaths could be randomly selected. In the cases studied, there were 4 possible wheelpaths (2 each in 2 lanes) and almost exactly thirteen 100-ft sections (actually 1,320 ft) in a run. Therefore, a deck of cards was used in conjunction with a 2-digit random number table. Selection with replacement from the deck of cards was used with suits determining the wheelpaths and face values determining the first digits of the starting stations. Selections from the random number table then provided the remaining 2 digits for the starting stations.

In all cases except one, the sampling plans were performed randomly and were repeated between 60 and 78 times to obtain reasonably accurate estimates of σ_s. For the case in which sampling was started at the beginning of the job rather than at a random starting point, it was possible to compute an exact numerical value for the sampling plan variance. Since there were four 330-ft sections, each with 4 wheelpaths, the total possible number of different samples was $4^4 = 256$. These were tabulated and used to calculate an exact value of σ_s for this particular case.

Although they are not shown in this report, histograms were plotted for each data set. In every case they were very nearly normal. This result was expected and served to confirm the validity of the computation of σ_s and its use in the procedures that followed.

Figure 1. Schematic drawing of a rolling straightedge.

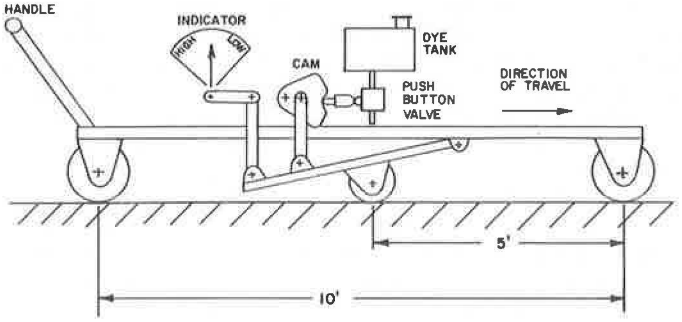


Figure 2. Typical graphical representation of a roadway of known roughness.

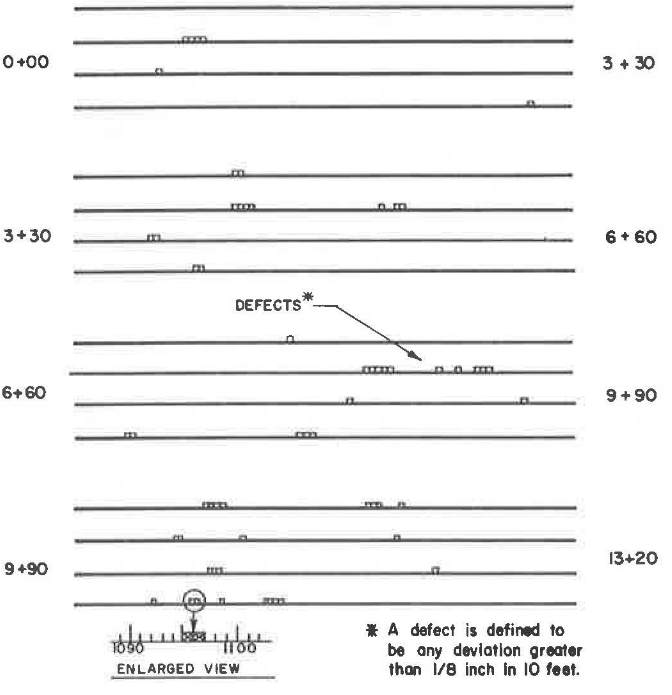


Table 1. Rolling straightedge sampling plan simulation tests.

Test	Pavement Type	Number of Replicate Runs	Sampling Plan		True Mean Percent Defective	Sample Mean Percent Defective	σ_s Due to Sampling Plan
			Percent	Details			
1	Bituminous	78	25	Random starting station, random selection of wheelpath every 330 ft	0.38	0.42	0.24
2	Bituminous	72	25	Same	0.64	0.64	0.37
3	Bituminous	72	25	Same	2.44	2.49	0.72
4	Concrete	60	12.5	Random starting station, 2 (only) successive randomly selected 330-ft sections	8.21	8.08	1.99
5	Concrete	60	25	Random starting station, random selection of wheelpath every 330 ft	8.21	8.34	1.20
6	Concrete	60	25	Start at beginning of job, random selection of wheelpath every 330 ft	8.21	7.94	1.17
7	Concrete	256	25	Same (exact numerical simulation)	8.21	8.23	1.09
8	Concrete	60	50	Start at beginning of job, 2 different randomly selected wheelpaths from each 330-ft section	8.21	8.18	0.65

Table 1 gives the results obtained with various sampling plans and varying levels of roughness. (The characteristically higher percent defective level of concrete pavement is attributed to the expansion joints that occur at regular intervals of approximately 80 ft.) Four general observations can be made from the data in this table:

1. σ_s increases as the level of roughness increases, although the increase is not linear. The approximate relationship between σ_s and the percent defective length is shown in Figure 3. Values near the origin were obtained from bituminous pavement whereas the higher values were obtained from concrete pavement. This combination of the 2 types of data is felt to be appropriate because the rolling straightedge measures surface irregularities without regard to the nature of the surface. It would appear from the continuity of the curve in Figure 3 that this assumption is valid.
2. For a pavement that is quite rough, σ_s is strongly influenced by the level of the sampling fraction. This is shown in Figure 4 for a typical New Jersey concrete pavement. It is expected that this effect would be less pronounced on bituminous pavement, which typically has a much lower average level of roughness.
3. Essentially the same value for σ_s was obtained by starting the sampling at the beginning of the job as was obtained with a randomized starting station (1.17 versus 1.20).
4. The single exact numerical determination of σ_s provided a close check with the value obtained by the Monte Carlo simulation procedure on the same section of pavement (1.09 versus 1.17).

DEVELOPMENT OF THE SPECIFICATION

The components of variance associated with both the instrument and the various sampling plans are now reasonably well determined. These values, combined with engineering judgment concerning the acceptable quality level (degree of roughness), are sufficient to establish an appropriate specification. The steps are as follows:

1. Determine what constitutes satisfactory work and unsatisfactory work. The practical approach (assuming no other information is available) is to make a statistical survey of many jobs that are judged to represent both satisfactory and unsatisfactory workmanship. The parameters thus obtained can be used to establish appropriate criteria for the evaluation of future work.
2. Establish specification limits that, if complied with, will yield the desired results. Consideration must be given to the risks that can be tolerated by both the producer and the consumer. This will involve engineering judgments such as, "We can afford to accept up to 10 percent of the work below some specified level," or what is the same thing in the long run, "We can afford to run a 10 percent risk that we will accept work below this specific level." Another requirement might be, "We want no work to fall below some minimum level." In terms of producer's risk, an additional requirement could be stated, "We want any contractor who does satisfactory work to run no more than a minimal risk for having his work rejected."
3. Select a measurement process and a sampling plan that will determine whether the desired objectives are being achieved.

These steps will now be followed to illustrate how a rolling straightedge specification and sampling plan may be derived. For illustration purposes, suppose the engineering requirements are as follows:

1. Producer's risk—Based on historical data, a contractor who constructs a bituminous pavement with a percent defective value of 0.75 or less is doing good work and should run essentially a zero risk for nonacceptance.
2. Consumer's risk—Based on the previously cited correlation with the BPR roughometer and a study of historical data, a percent defective value greater than 2.5 is judged to be totally unacceptable for bituminous pavement. The risk for acceptance of

Figure 3. Relation between σ_S and percent defective length for 25 percent sample fraction.

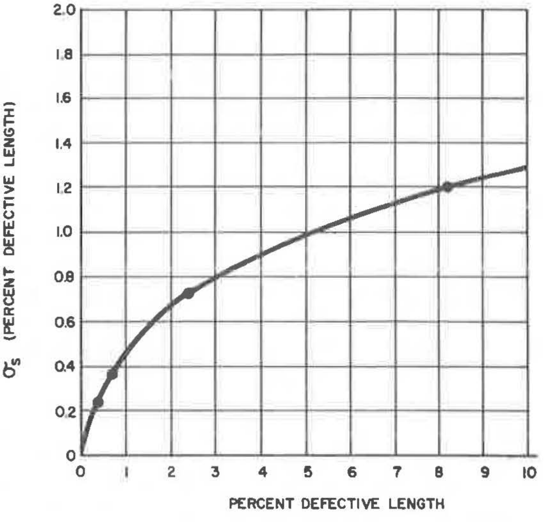


Figure 4. Relation between σ_s and size of sample fraction for concrete pavement with 8.21 percent defective length.

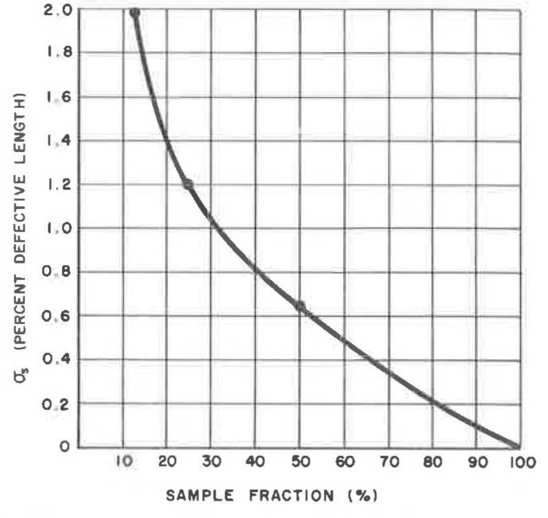
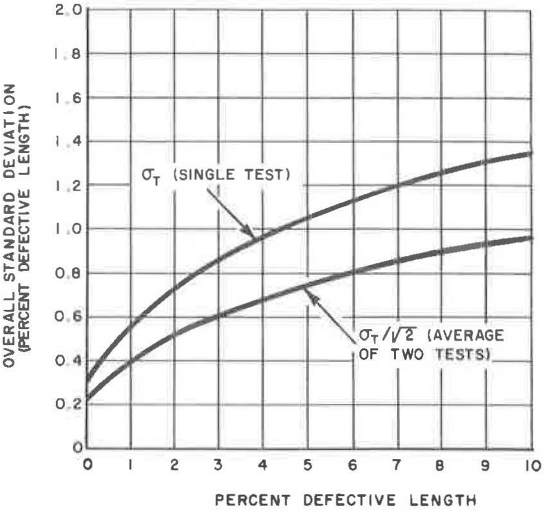


Figure 5. Relation between overall standard deviation and percent defective length for 25 percent sample fraction.



a pavement of this quality should be essentially zero.

At this point, in order to formulate the appropriate specification requirements, it is necessary to do some trial-and-error work with operating characteristic curves. An operating characteristic curve is simply a graphical representation of the probability of acceptance for all possible quality levels of the work. To construct such a curve, the standard deviation of the process must be known and an acceptance limit chosen.

Normally, the standard deviation is assumed to remain constant throughout the range covered by an operating characteristic curve. However, as can be seen from Figure 3, this will not be true for this example. Although the instrument standard deviation is assumed to remain constant ($\sigma_I = 0.30$) throughout the normal percent defective range for bituminous pavement (0 to 3.0), the sampling plan standard deviation (σ_S) is seen to increase rapidly with increasing average roughness of the pavement. The overall standard deviation (σ_T) will vary accordingly since it represents a combination of both instrument and sampling variability. For convenience in constructing the operating characteristic curves, the σ_T values for varying average levels of roughness are plotted in Figure 5.

For example, at a percent defective length of 3.0, σ_S is found from Figure 3 to be 0.80. Since σ_I is approximately 0.30 in this range of roughness, σ_T is calculated from the expression

$$\sigma_T = \sqrt{\sigma_I^2 + \sigma_S^2} = \sqrt{0.30^2 + 0.80^2} = 0.85$$

and is plotted at a percent defective level of 3.0 in Figure 5. Because σ_I is known to be approximately 0.40 at higher levels of percent defective length, this value is used to calculate σ_T for the upper part of the curve. Because it will later be seen to be useful, $\sigma_T/\sqrt{2}$ is also plotted in this figure. This represents the standard deviation associated with the average of 2 tests.

Before discussing the results, it may be worthwhile to describe how the operating characteristic curves are derived. A trial-and-error procedure is required, the objectives being to balance the risks while keeping both the risks and the sampling rate at reasonably low levels. If extremely low risks are insisted on, then the sampling rate will be unnecessarily high. On the other hand, if a low rate of sampling is arbitrarily selected, then the risks (both to the producer and the consumer) may be too great. This turns out to be a situation that cannot be optimized but, instead, is one in which judgment must be used to select the plan that accomplishes all objectives to a satisfactory degree. The plans presented herein are felt to achieve this purpose, but it should be understood that they are by no means unique. Similar plans could be designed that might be considered equally good.

The operating characteristic curves illustrating the capability of the acceptance procedure for bituminous pavement are shown in Figures 6 and 7. Figure 6 shows the separate curves for the first test and (when required) a retest. Figure 7 shows the single operating characteristic curve that represents the combined probability of the acceptance procedure, i.e., the probability of acceptance by either the first or second test.

The plan shown in Figure 6 requires a sample fraction of 25 percent and involves 2 distinct acceptance limits: 0.9 maximum percent defective for the first test and 1.2 maximum percent defective for the average of 2 tests. In actual practice, if a value of 0.9 or less is obtained on the first test, the work is accepted. If a value larger than 0.9 is obtained on the first test, the complete test procedure is repeated (including the random selection of wheelpaths) and the average for the 2 tests is then required to be less than or equal to 1.2 to be accepted. The average of the 2 tests is taken as the final result, and no further tests are permitted. If a reduced payment schedule is to be applied, it should be based on this average result, and the onset of penalties should begin just above a percent defective level of 1.2.

Figure 6. Individual operating characteristic curves for a 25 percent sample fraction on bituminous pavement.

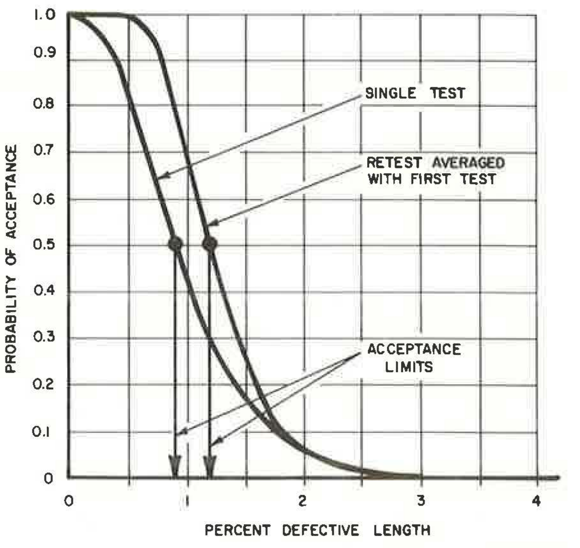


Figure 7. Operating characteristic curve for the complete test procedure with a 25 percent sample fraction on bituminous pavement.

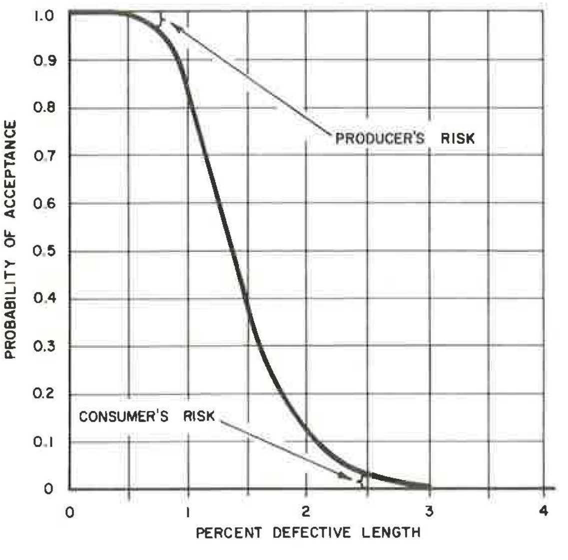


Figure 8. Operating characteristic curve for a 25 percent sample fraction on concrete pavement.

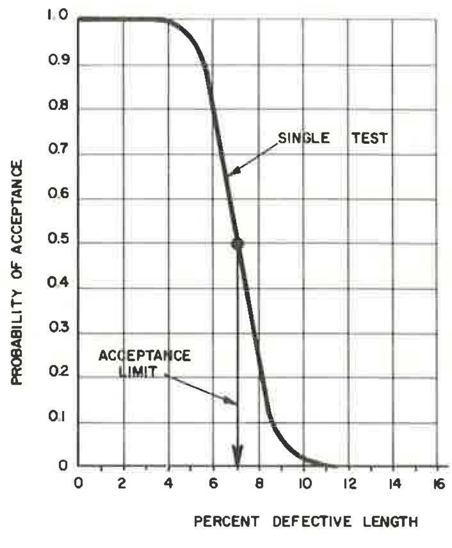


Table 2. Summary of the specification.

Item	Bituminous Pavement	Concrete Pavement
Upper limit, which desirably is never accepted	2.5 percent defective length	11.0 percent defective length
Lower limit, which desirably is never rejected	0.75 percent defective length	4.0 percent defective length
Sampling procedure and calculation required for first test	Continuous measurement starting at beginning of job, random selection of wheelpath every 300 ft, calculate percent defective length	Continuous measurement starting at beginning of job, random selection of wheelpath every 300 ft, calculate percent defective length
Acceptance criteria for first test	Percent defective length must be 0.9 percent or less	Percent defective length must be 7.0 percent or less
Sampling procedure and calculation required for second test	Repeat first test procedure (again choosing random wheelpaths) and average the results of both tests	Second test not permitted
Acceptance criteria for second test	Average percent defective length must be 1.2 percent or less	Second test not permitted

Note: In order to simplify the illustration, this specification applies only to pavement that is 2 lanes (four wheelpaths) wide. To satisfy the same requirements at the same risk levels for pavements of a different width, it may be necessary to change the sample fraction or the acceptance criteria or both. For a more complete treatment of this aspect of the subject, the reader is referred to the work of Croteau (1).

To illustrate how the operating characteristic curves are actually plotted, consider the "single test" curve of Figure 6. At a percent defective length of 2.0, for example, it is desired to know the probability that the pavement will be accepted (i.e., the probability that the testing procedure will measure a value of 0.9 or less). For a single test, σ_T is found from Figure 5 to be 0.73 at a percent defective length of 2.0. By using a table of the cumulative normal distribution, the probability of acceptance is found as follows:

$$Z = \frac{X - \mu}{\sigma} = \frac{0.9 - 2.0}{0.73} = -1.51$$

$$\alpha = 0.066$$

The probability of acceptance is then 6.6 percent, which is plotted at a percent defective length of 2.0 in Figure 6.

The operating characteristic curve in Figure 7 is obtained in a different manner. Here it is desired to know the probability of acceptance for the test procedure as a whole. Since the pavement could be accepted by either the first test or the second test, this curve is obtained by adding the probabilities of these 2 events. If the probability of passing the first test is designated P_1 and the probability of passing the second test is designated P_2, then the combined probability of acceptance (P_c) can be calculated from the relationship

$$P_c = P_1 + (1 - P_1) P_2$$

For any particular level of percent defective length, the values for P_1 and P_2 are read directly from Figure 6, the calculation is performed, and the result is plotted in Figure 7.

Since Figure 7 represents the application of the test procedure as a whole, it is here that we check to see if our objectives have been achieved. It can be seen that the risks are balanced and small, being approximately equal to 3.5 percent at the critical levels of 0.75 and 2.5 percent defective length.

One final remark is appropriate in regard to this sampling plan. Recognizing that the 2 distinct acceptance levels of 0.9 and 1.2 do not constitute a unique solution, one might wonder if it would be possible to find a single value for both acceptance levels that would permit the risks to be balanced and, if so, what the risks would be. The answer is yes, it can be done, and the result is a plan whose slightly higher risks might be considered by some to be justified by the simplicity of the single acceptance level for both tests. The value for the single acceptance level is 1.07 and the risks are balanced at about 5 percent. If this acceptance level is rounded off to 1.1, the risks go slightly out of balance, being approximately 4 percent and 5 percent for the producer and consumer respectively.

The same general approach is followed to develop the test procedure for concrete pavement. Because our concrete pavements are not only rougher but also more variable, it is somewhat difficult to select a specific roughness that separates acceptable and unacceptable work. For illustration purposes, let us define a level of 4.0 as definitely acceptable and a level of 11.0 as definitely unacceptable. A 4.0 percent defective pavement should almost always be accepted whereas an 11.0 percent defective pavement should almost always be rejected.

In this case, the resulting specification will be different from that obtained for bituminous pavement. The 2 levels at which a risk requirement is imposed are far enough apart so that a single test with a sample fraction of 25 percent is capable of satisfying both requirements. An acceptance level of 7.0 should be specified and only a single test would be permitted. The operating characteristic curve for this test is shown in Figure 8.

SUMMARY

This study illustrates the use of a Monte Carlo simulation procedure to determine the component of variance associated with the sampling plan for a measurement process. For this particular example (rolling straightedge), the sampling plan variance was found to be a substantial part of the overall variance. The overall variance, combined with acceptable quality levels and appropriate risk levels, was used to derive a statistically based specification, which is summarized in Table 2. It was also indicated by the simulation tests that the starting location for sampling may always be the beginning of the job, if desired, with no appreciable effect on the precision of the method.

Although the results obtained in this study apply directly to New Jersey pavement and the particular model of rolling straightedge used, this work should serve as a useful guide to anyone who plans to use a device of this type. It may be of still further use to anyone who might wish to apply this general approach to other measurement processes to derive specifications with balanced producer's and consumer's risks.

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2. Improved Quality Assurance of Bituminous Pavements. Federal Highway Administration, New Jersey Project Report, Region 15, Jan. 1973.

THE PRECISION OF SELECTED AGGREGATE TEST METHODS

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The interlaboratory correlation program pilot study is briefly discussed. Precision statements derived from the study are presented for the following test methods: sieve analysis, percent crushed particles, L.A. rattler, sand equivalent, cleanness value, durability index, and R-value. Relative amounts of general error types such as between operator and between laboratory are given for each test method, and possible causes are discussed. Laboratory performance is shown through the use of scatter diagrams and ranking summaries. Recommendations for improving test precision are given.

•OVER the years a number of valuable test methods have been developed for judging the quality of aggregate used in portland cement concrete, asphalt concrete, and base and subbase construction. When applied properly, these tests have been used consistently to accept material of adequate quality and to reject material of inferior quality. Until recently, however, only a minimum effort has been made to measure and improve the precision of these tests. Active calibration and certification programs have sought to identify testing errors so that they might be reduced. However, these programs have been handicapped by lack of knowledge about the magnitude and source of these errors. An integrated method for continually monitoring test precision and evaluating laboratory and operator performance is needed.

This report summarizes the results of a year-long pilot study that measured the precision of a number of aggregate test methods, quantified the sources of testing error, and evaluated laboratory performance. The test methods studied were coarse and fine sieve analysis, R-value, L.A. rattler abrasion, fine durability, coarse durability, cleanness value, and percent crushed particles. The precision of the sand equivalent test, determined under a separate study (4), is also included in this report.

The results contained herein were analyzed by a series of computer programs developed especially for this study. These programs are fully explained in another report (1).

To clarify some of the conclusions reached in this report, a discussion of the concepts of precision and testing error is necessary. California has adopted a method of reporting test precision recommended in ASTM Designation C 670-71T. This is based on a statistical parameter called the difference 2-sigma limit (D2S). ASTM uses the D2S limit to form 2 different types of precision statements:

1. Single-operator precision—a measure of the greatest difference between 2 results that would be considered acceptable when properly conducted determinations are made on uniformly prepared portions of material by a competent operator using 1 set of equipment.

2. Multilaboratory precision—a measure of the greatest difference between 2 results that would be considered acceptable when properly conducted determinations are made by 2 different operators in different laboratories on uniformly prepared portions of material.

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Single-operator and multilaboratory precision statements are given in this report for each test method studied. The D2S limit is referred to as the "acceptable range of 2 results" in these statements. For many of the tests, precision was found to vary significantly according to the range of material tested. The precision statement is given in a tabular form for these test methods. The overall range of material studied for each test method is also given. Precision statements are accurate for this range only and should not be extrapolated.

Testing error was divided into 2 general categories for the purposes of this study. The first, systematic error, is composed of errors whose sources are identifiable. For this experiment the identifiable sources of error were between laboratories, between operators in the same laboratory, and scale-type error (3). A large between-laboratory error might indicate significant variations from laboratory to laboratory in technique, environment, or equipment. A large between-operator error could indicate inadequate training and certification programs at the local level. Scale-type errors are caused by inconsistencies between expected and observed test results from one range of results to another. Significant scale-type errors usually occur in test methods that use different equipment or techniques for each range of material tested. For instance, a set of poorly calibrated standard weights would yield a large scale-type error when weighing objects of varying sizes. Systematic errors can often be minimized because their causes are usually known.

The second type of testing error, residual error, represents the total of all errors not accounted for by the systematic components of operator, laboratory, and scale-type effects. Minimizing this type of error can be more difficult. If additional experimentation does not reveal more systematic components of the residual error, the precision of the final result can only be improved by averaging a predetermined number of repeated tests for each test result or by tightening method and equipment tolerances. Before this is done, however, the magnitude of the sample-preparation error (a measure of uniformity of the sample-preparation procedure) should be checked. If this error is a large part of the overall residual error, then the actual test precision will be better than indicated and may not need improvement.

Single-operator precision was calculated from the residual error, and as such included the random errors inherent in both the test method and the sample-preparation procedure. Multilaboratory precision was derived from a combination of systematic and residual errors and therefore included effects of laboratory environments, equipment, and operator technique in addition to the residual error.

DESCRIPTION OF WORK

The California Department of Transportation's 11 district materials laboratories and its headquarters laboratory were the participants in this pilot study. Sample preparation and data analysis were handled by Transportation Laboratory personnel in Sacramento. The testing program was spread out over almost 2 years, and the analysis phase, speeded by the use of the computer, was completed in several months.

The samples were prepared and distributed in sets of 2. Samples in each set were of the same aggregate type (i.e., AC, PCC, AB, or AS) but were obtained from 2 different sources. The test methods performed on each set of samples are given in Table 1. A total of 10 individual samples were studied. The total amount of testing to be done was determined by theoretical design considerations tempered by practical constraints.

Each sample was split into 64 subsamples. Each of these subsamples contained enough material to perform 1 series of tests, and 48 of these were randomly assigned to the different laboratories. The remaining 16 subsamples were kept as a contingency. Thus, each of the 12 participating laboratories received 4 subsamples from each sample (Figure 1).

At the beginning of each 3-month interval the laboratories received their 2 sets of 4 subsamples each. They then chose 2 operators and set aside 1 set of equipment. On the day or days that the tests were to be made, each operator was given 2 subsamples

Table 1. Summary of testing.

Samples	Aggregate Type	Dates Tested	Tests Studied	Calif. Test Method No.
1 and 2	PCC (fine)	3/72-5/72	Fine sieve analysis	202G
			Fine durability index	229E
3 and 4	PCC (coarse)	3/72-5/72	Coarse sieve analysis	202G
			L.A. rattler	211D
			Cleanness value	227E
5 and 6	AB	6/72-10/72	Sieve analysis	202G
			Durability index	229E
			Percent crushed particles	205E
			R-value	301F
7 and 8	AC	7/73-11/73	Sieve analysis	202G
			L.A. rattler	229E
			Percent crushed particles	205E
9 and 0	AS	4/73-6/73	Sieve analysis	202G
			R-value	301F

Figure 1. Sample distribution.

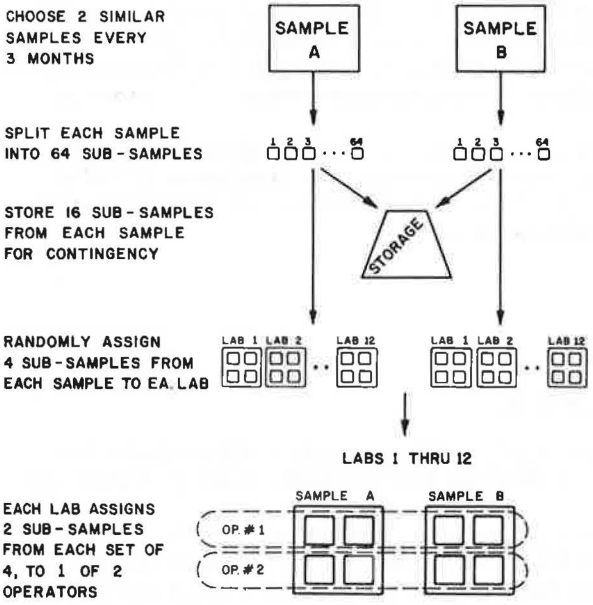
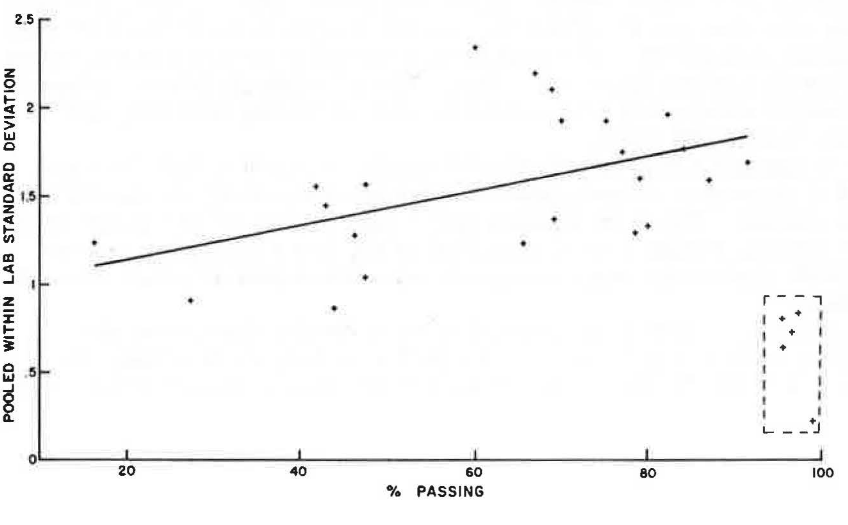


Figure 2. Test precision versus material range, coarse sieve analysis.



from each sample. The operators ran the indicated tests following their usual procedure. The operators used the same set of equipment for the 4 subsamples tested.

ANALYSIS

Only brief mention of the analytical techniques employed in this study will be made here. A more complete discussion can be found elsewhere (1).

The relationship between test precision and material range was investigated first for each of the test methods studied. This was done by linearly regressing the pooled within-lab standard deviation against the overall sample average. If a significant relationship was indicated, data transformation was required before further analysis could be made.

Precision statements were determined by using a 3-factorial analysis of variance and isolating the components of variance according to expected mean-square equations. These same components were used to estimate the relative distribution of the general error types: between-operator, between-laboratory, and residual.

Cell variances were not homogeneous for the test methods whose precision varied with material range. Because overall homogeneity of cell variance is a prime assumption on which the theory of analysis of variance relies, logarithmic transformations were used extensively. Results were then retransformed according to the common rules for propagation of error (2).

Scale-type errors were derived from Mandel's linear model analysis (2, 3). Variations in the distribution of errors from different sources as a function of material range were also studied using this method. Laboratory performance was monitored by using scatter diagrams and ranking summaries.

FINDINGS AND CONCLUSIONS

Sieve Analysis (Test Method No. Calif. 202-G)

The sieve analysis test method is divided into 2 parts—a coarse analysis and a fine analysis. Because these are, in effect, 2 different test methods, their precision was studied separately.

The coarse analysis procedure is used for material retained on the No. 4 and coarser sieves. Test precision for these sieves was found to be roughly dependent on the total weight of material passing them. Except for the range of 95 to 99 percent passing, the greater the weight of material passing a coarse sieve, the less repeatable were its results. Apparently, shaking time became more critical and errors from sieve defects were magnified as a greater weight of material passed through a given sieve. The dependent nature of one sieve result on another makes this impossible to prove, however. The assumption was made for this study that the percent passing a sieve was a reasonably consistent representation of the actual weight of material passing the sieve, since sample sizes were fairly uniform from test to test. The relationship between percent passing and repeatability should only be considered a rule of thumb, however, and should not be applied in extreme cases.

Figure 2 shows the pooled within-lab standard deviation (a result of both between-operator and residual sources of error) plotted against percent passing for all coarse sieve-sample combinations. The least squares linear plot shown, which was not based on 95 to 99 percent passing results (shown in dashed area), has a coefficient of correlation of 0.49. Table 2 gives the single-operator and multilaboratory precision of the coarse sieve analysis.

A fine analysis procedure is used for material passing the No. 4 and finer sieves. This method combines hydraulic and mechanical agitation techniques to gradate the sample and wash out clay and silt particles. Table 3 gives its precision over the range studied.

The fine sieve results are weighted according to the amount of material passing the No. 4 sieve to yield combined or overall results for sieves No. 8 through No. 200. Figure 3 shows the pooled within-lab standard deviation plotted against the percent passing for these results. The coefficient of correlation for this linear regression is 0.89. Table 4 summarizes the precision of the combined sieve analysis.

The most dominant source of error for both the coarse and fine sieve analyses was residual error. It is presumed that the largest part of this error was caused by the inability to accurately split samples into identical subsamples.

Percent Crushed Particles Retained on No. 4 Screen (Test Method No. Calif. 205-E)

The relative amount of crushed material contained in a sample of aggregate is evaluated by inspection. The 4 samples tested by this method ranged from approximately 55 percent to 95 percent crushed particles. The test exhibited very large systematic errors, particularly between laboratories. The error distribution was as follows: between laboratory, 65 percent; between operator, 20 percent; and residual error, 15 percent. The precision of the crushed particle test was shown to be very poor, especially for materials with low crushed-particle counts (Table 5). Discrepant results roughly correlated with geographical location, with laboratories in the southern part of California getting significantly lower results than the rest of the state.

The large errors measured for this test method are most likely caused by the highly subjective nature of the test. If this test is to be used as a contract control test, the source of these errors must be identified and minimized.

L.A. Rattler (Test Method No. Calif. 211-D, 500 Rev.)

The L.A. Rattler Test is used to measure the resistance of coarse aggregate to degradation caused by impact. The range of results studied for this test method was 13 to 18 percent loss. The precision measured was constant over this range, as shown in the following precision statement:

Type	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator	1.10	1.05	3.0 percent loss
Multilaboratory	3.53	1.88	5.3 percent loss

An analysis of the components of variance revealed that between-laboratory error constituted 70 percent of the overall error. Residual error made up the remaining 30 percent, while between-operator error was negligible. Since each laboratory has only 1 Los Angeles Abrasion Testing Machine, it becomes obvious that equipment, not operator technique, is the most critical factor affecting the precision of the test.

Sand Equivalent (Test Method No. Calif. 217-I)

The precision of the sand equivalent test method was determined and reported under a separate study (10) and is included here for completeness. The single-operator precision was as follows:

Table 2. Precision statement tabulation, coarse sieve analysis (¾-in. through No. 4).

Percent Passing	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator precision			
20	1.09	1.04	3.0
40	1.49	1.22	3.5
60	1.96	1.40	4.0
80	2.48	1.58	4.5
1 to 5 and 95 to 99	0.56	0.75	2.1
Multilaboratory precision			
20	1.58	1.26	3.6
40	2.16	1.47	4.2
60	2.83	1.68	4.8
80	3.59	1.89	5.4
1 to 5 and 95 to 99	1.17	1.08	3.1

Table 3. Precision statement tabulation, fine sieve analysis (No. 8 through No. 200).

Percent Passing	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator precision			
20	1.11	1.06	3.0
40	1.95	1.40	4.0
60	3.02	1.74	4.9
80	4.32	2.08	5.9
Multilaboratory precision			
20	1.68	1.30	3.7
40	2.95	1.72	4.9
60	4.57	2.14	6.0
80	6.54	2.56	7.2

Figure 3. Test precisions versus material range, combined sieve analysis.

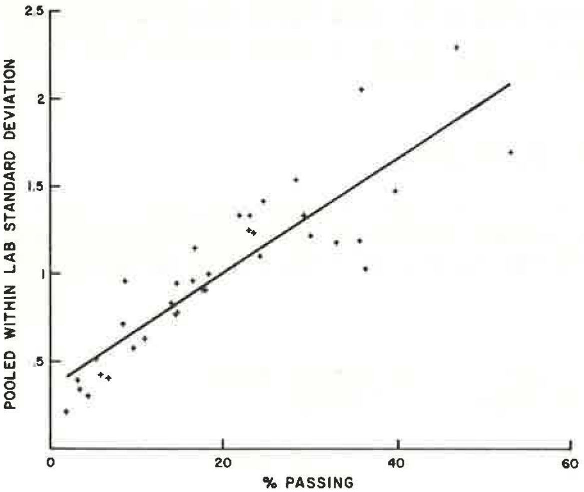


Table 4. Precision statement tabulation, combined sieve analysis (No. 8 through No. 200).

Percent Passing	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator precision			
10	0.41	0.64	1.8
20	0.91	0.95	2.7
30	1.60	1.26	3.6
40	2.48	1.57	4.5
50	3.55	1.89	5.3
Multilaboratory precision			
10	0.56	0.75	2.1
20	0.22	1.11	3.1
30	2.15	1.47	4.1
40	3.34	1.83	5.2
50	4.78	2.19	6.2

Table 5. Precision system tabulation, percent crushed particles (retained on No. 4).

Percent Crushed	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator precision			
60	30.74	5.54	16
70	20.02	4.47	13
80	11.59	3.40	10
90	5.45	2.33	7
Multilaboratory precision			
60	218.90	14.80	42
70	142.56	11.94	34
80	82.52	9.08	26
90	38.80	6.23	18

Sand Equivalent Range	Variance	Standard Deviation	Acceptable Range of 2 Results
Below 45	1.87	1.37	3.9
45 to 65	8.72	2.95	8.4
Above 65	4.27	2.07	5.9

The multilaboratory precision was as follows:

Sand Equivalent Range	Variance	Standard Deviation	Acceptable Range of 2 Results
Below 45	2.90	1.70	4.8
45 to 65	14.05	3.75	10.6
Above 65	7.03	2.65	7.5

Cleanness Value (Test Method No. Calif. 227-E)

The cleanness test indicates the amount, fineness, and character of clay-like materials and coatings present in coarse aggregate. Precision of the test was based on 2 samples in the 90 to 95 percent cleanness value range. The conclusions drawn from the limited data are preliminary and will be augmented in the future by a continuous correlation program that has already been implemented.

Between-operator error was found to be insignificant, whereas between-laboratory error constituted over 40 percent of the total error. This tends to indicate that there are either equipment calibration deficiencies or lack of uniform application of testing procedures from laboratory to laboratory. The actual errors are reasonably small, however, as illustrated by the precision statement:

Type	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator	0.69	0.83	2.3 CV units
Multilaboratory	1.21	1.10	3.1 CV units

Durability Index (Test Method No. Calif. 229-E)

The durability index is a measure of an aggregate's resistance to producing detrimental clay-like fines when subjected to certain chemical and mechanical forms of degradation. Both fine and coarse durability methods are used. The precision of the 2 methods is given in Tables 6 and 7 respectively. Test precision improves with increased durability.

Since coarse durability was measured for only 2 samples, the precision measurements in Table 7 should be considered preliminary. However, fine durability results were recorded for 4 samples, permitting fairly reliable measurement of the systematic errors. The breakdown of the overall fine durability error was as follows: between laboratory, 50 percent; between operator, 30 percent; and residual error, 20 percent. For high-range material, however, between-laboratory error diminished to 20 percent, whereas for low-range material it increased to 60 percent. This indicates that the test is more sensitive at low durabilities than high durabilities to some source of error occurring between the laboratories. This error could be caused by differences

Table 6. Precision statement tabulation, fine durability.

Fine Durability	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator precision			
50	5.74	2.40	6.8
60	4.33	2.08	5.9
70	3.11	1.76	5.0
Multilaboratory precision			
50	26.07	5.11	14.4
60	19.65	4.43	12.5
70	14.14	3.76	10.6

Table 7. Precision statement tabulation, coarse durability.

Coarse Durability	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator precision			
60	12.85	3.58	10.1
70	6.53	2.56	7.2
80	2.33	1.53	4.3
Multilaboratory precision			
60	18.88	4.35	12.3
70	9.59	3.10	8.8
80	3.42	1.85	5.2

Table 8. Precision statement tabulation, R-value.

R-Value	Variance	Standard Deviation	Acceptable Range of 2 Results
Single-operator precision			
30	38.54	6.21	18
40	27.87	5.28	15
50	18.92	4.35	12
60	11.69	3.42	10
70	6.20	2.49	7
80	2.43	1.56	4
Multilaboratory precision			
30	76.40	8.74	25
40	55.24	7.43	21
50	37.49	6.12	17
60	23.18	4.81	14
70	12.29	3.51	10
80	4.83	2.20	6

in calcium chloride solutions, tap water, temperature control, or agitators. Further study to identify which of these factors significantly affects the precision and then eliminating that error should substantially improve the precision of the test.

The 2 sets of samples on which fine durability measurements were made were sent out 3 months apart. For the most part, the same operators ran the tests using the same equipment. However, a significant within-laboratory scale-type error was measured. It appears that the most probable source of this error was a change either in laboratory temperature or calcium chloride solution concentration during the 2-month period.

R-Value (Test Method No. Calif. 301-F)

The 4 samples tested ranged in R-value from 30 to 85. As with many of the other tests, precision was found to vary according to the range of material tested. In the range tested, low R-value material yielded less precise test results than high R-value material.

Table 8 summarizes the single-operator and multilaboratory precision for the R-value test. The overall distribution of errors was as follows: between laboratory, 30 percent; between operator, 20 percent; and residual error, 50 percent. Between-laboratory error was greater than the 30 percent listed for low-range material. Also, significant scale-type errors of both the within- and between-laboratory varieties were observed. The scale-type errors were possibly caused by stabilometer readings, since these instruments, if not properly calibrated, can give high results in one range and low results in another. Intricate sample fabrication procedures probably contributed to a large portion of the residual error measured.

ACKNOWLEDGMENTS

Without the hard work of the numerous laboratory technicians throughout the state, this report would not have been possible. The authors wish to thank the Pavement Branch of the Transportation Laboratory, particularly Thomas Whitney, for performing the cumbersome and tedious job of splitting the bulky samples into manageable and homogeneous subsamples. Also, special thanks go to the laboratory personnel for diligently carrying out the extensive testing program in each of the 11 transportation districts.

This project was conducted in cooperation with the Federal Highway Administration. The contents of this report reflect the views of the Transportation Laboratory, which is responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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FLOATING BEADS: BROAD OR NARROW GRADATION?

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Through the use of theory and in road service tests the 2 most common types of floating beads, narrow and broad gradation, were evaluated. These tests, which included several control lines using a broad-gradation non-floating bead, showed that the broad-gradation floating bead performed best under all conditions. The narrow-gradation floating beads gave good reflectivity under dry conditions but invariably demonstrated poor reflectivity under the slightest rainfall conditions. These effects are demonstrated through the use of wet and dry night photographs of dual centerline test sections.

•ONE of the major developments in the use of drop-on glass spheres in recent years has been the introduction of floating beads. Along with this development came a suggested change in the size range or gradation of drop-on beads.

Before the introduction of floating beads, virtually all specifications for drop-on beads required a broad-range gradation, which can be described as a 20-80 gradation. When floating beads were introduced, many of the specifications required a narrower size range, commonly termed a 40-80 gradation.

Since floating beads can be obtained in either size range, a question is raised as to which gradation should be specified. This paper attempts to outline the advantages and disadvantages of each gradation, both from a theoretical and practical point of view, with a view toward helping those concerned to make an intelligent choice between the 2 systems.

DESIGN THEORY OF THE 2 GRADATIONS

In order to discuss the 2 general size ranges it is helpful to know the physical characteristics of each gradation that determined their development.

The 20-80 gradation was originally a nonfloating drop-on bead. This gradation is correctly sized for optimum reflectivity and durability in a standard, 0.015 ±0.001-in. (0.38 ±0.025-mm) wet paint film thickness.

Traffic paints normally have a solids content between 50 and 60 percent. It is this property that determines the final dry film thickness. Thus, taking the range of wet film thicknesses given, 0.014 to 0.016 in. (0.36 to 0.41 mm), one obtains a dry paint film thickness range of 0.007 to 0.010 in. (0.18 to 0.25 mm).

Since it was known that the optimum embedment of a glass sphere is between 50 and 60 percent of its diameter, the optimum diameter of beads for the range of dry film thicknesses would be from 0.012 to 0.020 in. (0.30 to 0.51 mm). This range of bead diameters corresponds approximately to a U. S. sieve range between No. 35 and No. 50.

This range, however, is calculated on a rather close wet paint film tolerance. In reality, the wet paint film thickness will often vary considerably. Variations in machine speed, ambient temperature, paint viscosity, tank pressure, and amount of thinner used can cause rather large variations in film thickness. These variations can occur not only from day to day but also from mile to mile on any one day's application.

For these reasons it is desirable to have a broader range of bead sizes than the No. 35 to No. 50 mesh range calculated.

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Figures 1-11 demonstrate the effectiveness of the broad-range gradation over wide variations in application thicknesses. Figure 1 shows the various sizes of beads present in a broad-gradation specification. The full range of glass bead sizes at the bottom of the figure is represented by various hollow glass balls.

The diameters of these "beads" were carefully selected to correspond to the diameters represented by the U.S. sieve sizes between No. 20 and No. 120. The graph above the "beads" in Figure 1 is the weight distribution common to most 20-80 (broad range) gradation beads. From the graph one can see that a majority of the beads are between No. 35 and No. 50 mesh, with a smaller quantity above and below this range.

Figure 2 shows how a full range of standard bead sizes would appear when dropped into a wet film of paint 0.015 in. (0.38 mm) thick. Figure 3 is the same size range as it would appear when the 0.015-in. (0.38-mm) paint film dries to a film thickness of 0.008 in. (0.20 mm). From this, one can see that the beads between No. 35 and No. 50 mesh are embedded to approximately one-half their diameters. Also, those beads below No. 50 mesh that are not immediately effective will eventually become exposed as the film wears, giving long-term reflectivity.

From the size distribution shown previously, one can see that more than 80 percent of the beads in a 20-80 gradation are embedded to one-half their diameter or greater. Thus, in a 0.015-in. (0.38-mm) wet film of paint that dries to a thickness of 0.008 in. (0.20 mm), a 20-80 broad-range gradation is very efficient. A majority of the beads (No. 35 to No. 50 mesh) are embedded securely and effectively, and the beads smaller than No. 50 mesh are available for future use as the paint wears.

Figures 4 and 5 show how a 20-80 gradation gives good initial reflectivity when the paint is applied at a somewhat thick wet film thickness of 0.020 in. (0.51 mm). Figure 4 is the wet film at 0.020 in. (0.51 mm), and Figure 5 is the same film when dried to a film thickness of 0.010 in. (0.25 mm). From these one can see that there are still a sufficient number of beads exposed, even in a dry film thickness of 0.010 in. (0.25 mm), to give good initial brightness. Also, the beads below No. 35 mesh, which are completely covered, are available for long-term reflectivity as the paint film wears.

If one were to view the same Figure 5 and consider it to be a film of wet paint 0.010 in. (0.25 mm) thick, one can visualize how 20-80 gradation beads would appear when dropped on a 0.010-in. (0.25-mm) wet film of paint. This wet film will dry to approximately 0.005 in. (0.13 mm), and the resulting effect can be seen in Figure 6. In this case, one can see that the large beads will be more readily removed, but there are still a significant number of beads embedded securely for optimum initial and long-term reflectivity.

Thus, a broad-range gradation bead with a majority of beads sized for a wet film thickness of 0.015 in. (0.38 mm) will give optimum reflectivity and durability, even under wide variations in the final dry film thickness.

Floating beads, when properly manufactured, will embed themselves approximately 50 percent in a wet paint film and remain embedded at this level as the paint film dries and shrinks. In this way the brightness of the line is somewhat, although not completely, independent of film thickness. If one were to place a broad-range gradation floating bead on a 0.015-in. (0.38 mm) wet film of paint, all the beads would be supported at approximately one-half their diameters, as shown in Figure 7. However, as the paint film shrinks to a final dry film thickness of 0.008 in. (0.20 mm), the beads larger than 0.016 in. (0.41 mm) in diameter would "bottom out". This is shown in Figure 8, where the beads over 0.016 in. (0.41 mm) in diameter (approximately No. 40 mesh) are no longer embedded at one-half their diameters. The same "bottoming" effect, but to a lesser degree, is observable in a representation of 20-80 mesh floating beads in 0.020-in. (0.51-mm) wet and 0.010-in. (0.25 mm) dry paint film (Figures 9 and 10). All those beads larger than 0.020 in. (0.51 mm) in diameter (approximately No. 35 mesh) bottom out in a dry film 0.010 in. (0.25 mm) thick. In a 10-mil wet paint film (Figure 10), all the beads are not embedded to one-half their diameter initially, and as the paint film shrinks those beads more than 0.010 in. (0.25 mm) in diameter (approximately No. 60 mesh) will bottom out, as shown in Figure 11.

Thus, at a theoretical dry paint film of 0.008 in. (0.20 mm), it would seem that those beads above 0.016 in. (0.41 mm) in diameter (approximately No. 40 mesh) are unnec-

Figure 1. Size distribution for 20-80 gradation beads.

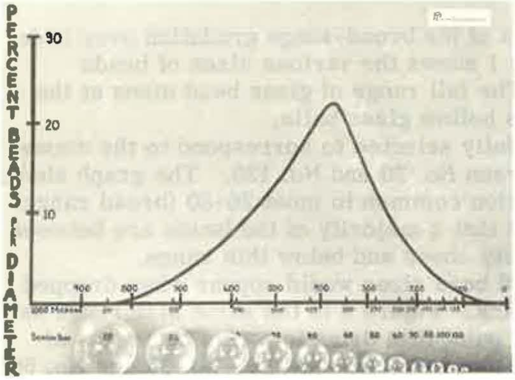


Figure 2. Embedment of 20-80 gradation beads in a wet paint film 0.015 in. thick.

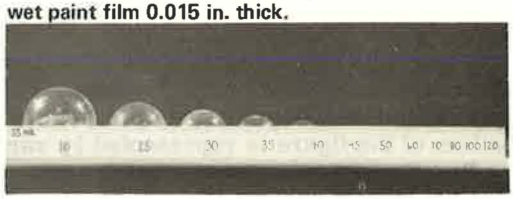


Figure 3. Embedment of 20-80 gradation beads in a dry paint film 0.008 in. thick.

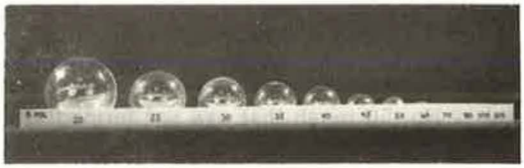


Figure 4. Embedment of 20-80 gradation beads in a wet paint film 0.020 in. thick.

Figure 5. Embedment of 20-80 gradation beads in a wet or dry paint film 0.010 in. thick.

Figure 6. Embedment of 20-80 gradation beads in a dry paint film 0.005 in. thick.

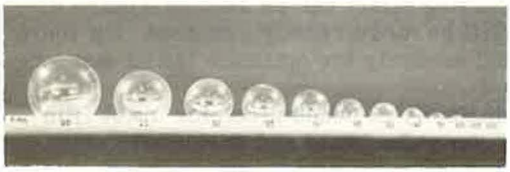


Figure 7. Embedment of 20-80 floating beads in a wet paint film 0.015 in. thick.

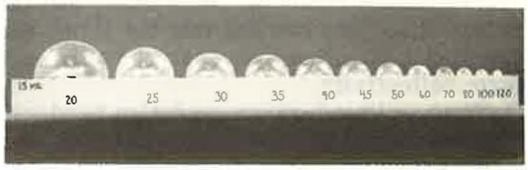


Figure 8. Embedment of 20-80 floating beads in a dry paint film 0.008 in. thick.

Figure 9. Embedment of 20-80 floating beads in a wet paint film 0.020 in. thick.

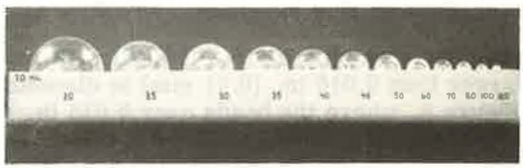


Figure 10. Embedment of 20-80 floating beads in a wet or dry paint film 0.010 in. thick.

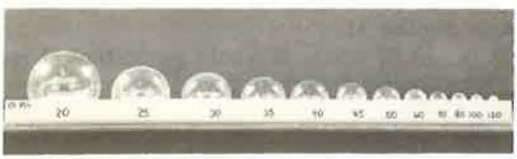
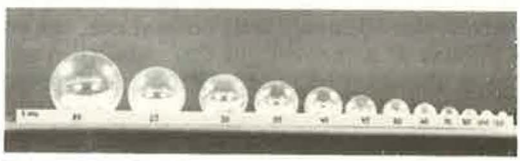


Figure 11. Embedment of 20-80 floating beads in a dry paint film 0.005 in thick.



essary in a floating system. However, because of other considerations not evident in a theoretical discussion, the beads larger than No. 40 mesh are necessary for full effectiveness under all conditions of use. One of these conditions is wet-weather visibility. From road service tests conducted on both the 20-80 and 40-80 gradation floating beads, the distinct advantage of a broad size distribution is evident.

ROAD SERVICE TESTS

Since 1971 Potters Industries, Inc., has been conducting dual centerline road service tests on various paint and bead systems. The test program was described in a paper by Ritter (1).

Most of the test sections compare a test line having various combinations and quantities of paint and beads with a standard line consisting of a 0.015-in. (0.38-mm) film of yellow alkyd traffic paint with 6 lb/gal (0.72 kg/liter) of 20-80 gradation beads dropped on. These 2 lines form a dual yellow centerline test section that is typically ½ to 1 mile (0.8 to 1.6 km) long. The sections are evaluated monthly by impartial observers from automobiles at night and rated on a scale of 10 (brightest) to 0 (least bright). In addition to the evaluations, night still photographs and motion pictures are taken for documentation purposes.

From the photographic documentation it is possible to illustrate the consistently poor wet-weather performance of the 40-80 floating beads and in the same way observe the relatively good performance of the 20-80 floating beads.

Figure 12 was taken on a dry night 6 days after the test lines were applied. The left line is a standard line as described previously and the right line consists of the same thickness of paint with 4 lb/gal (0.48 kg/liter) of the 40-80 gradation floating beads dropped on. From this one can see that the line with the floating beads is somewhat brighter initially. Figure 13 was taken 20 days later under moderate rainfall conditions. Here, the substantially poorer wet-weather performance of the narrow gradation is evident. The standard line, although somewhat less bright under dry conditions, is more visible under wet-weather conditions.

Approximately 6 months later the same section shows the right line to be still somewhat brighter in dry weather (Figure 14). However, again 20 days later, under moderate rainfall conditions, the 40-80 gradation line (right) demonstrates very poor performance (Figure 15).

To further study this effect, another test section of 40-80 floating beads was placed later that year. Section 22A was identical to section 8-1 (standard line versus 4 lb/gal of 40-80 floating beads. Much the same effect was observed in this section. The 40-80 floating beads, right line in Figures 16 and 17, again displayed noticeably poor wet night visibility.

In the next year's test program 2 sections were striped to compare the performance of a line having 4 lb/gal (0.48 kg/liter) of 40-80 floating beads and a line having 6 lb/gal (0.72 kg/liter) of 20-80 floating beads to a standard line. Both sections were placed in May 1972. The first section consisted of a standard line on the left next to the line with 40-80 floating beads at 4 lb/gal (0.48 kg/liter) on the right. Figure 18 shows the improved brightness of a floating-bead line over a standard line under dry road conditions. Figure 19, taken 2 days later on a rainy night, shows the same reversal of performance observed in the 1971 test sections.

The next section contained a standard line on the left versus the 6-lb/gal (0.72-kg/liter), 20-80 floating-bead line on the right. Figure 20, taken on a dry night, again shows the improved brightness of a floating-bead line over a standard line. Figure 21, taken under moderate rainfall conditions, shows the 20-80 floating-bead line to be noticeably brighter than the standard line. Because the dry photographs of both of these sections (Figures 18 and 20) were taken on the same night and the wet photographs (Figures 19 and 21) were both taken 2 days later, an effective comparison is possible. The 20-80 floating-bead line was noticeably brighter than the standard line under both wet and dry road conditions, while the 40-80 floating-bead line gave poor performance under wet conditions.

Figure 12. Test section 8-1, dry, April 23, 1971.

Figure 13. Test section 8-1, wet, May 13, 1971.

Figure 14. Test section 8-1, dry, October 4, 1971.

Figure 15. Test section 8-1, wet, October 24, 1971.

Figure 16. Test section 22A, dry, October 4, 1971.

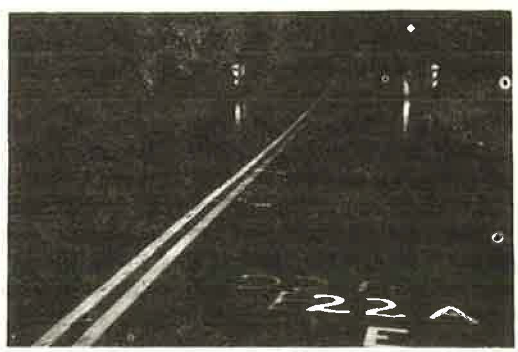


Figure 17. Test section 22A, wet, October 24, 1971.

Figure 18. Test section 11, dry, November 6, 1972.

Figure 19. Test section 11, wet, November 8, 1972.

Figure 20. Test section 13, dry, November 6, 1972.

Figure 21. Test section 13, wet, November 8, 1972.

Figure 22. Test section 38, dry, April 27, 1974.

Figure 23. Test section 38, wet, April 30, 1974.

To obtain a closer comparison of the performance of the two types of gradations of floating beads under both wet and dry conditions, section 38 was applied in April 1974. In Figure 22, taken April 27, 1974, under dry conditions, the left line contains 6 lb/gal (0.72 kg/liter) of 20-80 floating beads while the right line contains 4 lb/gal (0.48 kg/liter) of 40-80 floating beads. The film thickness is the same for both lines. From this dry-night photograph one can see that the brightness of both lines is approximately equal. However, Figure 23, taken just 3 days later, directly demonstrates the dramatic difference between the 2 systems in wet-night performance. The 20-80 line (left) is noticeably brighter and more visible than the 40-80 line.

CONCLUSIONS

In the section on design theory it was demonstrated how a broad-gradation nonfloating bead is correctly sized for optimum embedment in a dry paint film that can vary in thickness over a relatively wide range.

Under the same general conditions, it was also shown how a broad-gradation floating type of bead is effective. From the illustrations of the floating beads in a 0.015-in. (0.38-mm) wet, 0.008-in. (0.20-mm) dry paint film, one of the reasons for specifying a narrow gradation with floating beads was shown.

In road service tests of the various types of floating-bead lines currently applied it was shown that

1. A 40-80 gradation floating-bead line was brighter than a standard nonfloating-bead line only under dry conditions.
2. The same 40-80 gradation floating-bead line under wet road conditions invariably showed substantially poorer performance.
3. A 20-80 gradation floating-bead line was also brighter than a 20-80 gradation nonfloating-bead line under dry conditions.
4. The 20-80 gradation floating-bead line was far more visible under wet road conditions than the 40-80 gradation floating-bead line.

Thus, from a standpoint of effectiveness under all conditions of use, the broad-range gradation floating beads would seem to be the choice if floating beads were to be specified. This choice would give optimum performance under dry conditions and improved, not decreased, visibility under wet conditions.

REFERENCE

1. James R. Ritter. A Unique Approach to Evaluating Road Stripe Material on Two-Lane Rural Roads. Highway Research Record 447, 1973, pp. 1-7.

PAINTS AND GLASS BEADS USED FOR TRAFFIC DELINEATION MARKINGS

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This investigation was designed to study the effectiveness of Missouri's traffic delineation system on both concrete and asphalt concrete surfaces and to determine if safe and economical improvements could be made. The investigation was organized into three phases so that elimination of variables could be accomplished with a minimum of samples. Phase 1, a field evaluation of the Missouri standard dispersion resin-varnish paint and a chlorinated rubber-alkyd paint in a transverse stripe, clearly indicated the superiority of the chlorinated rubber-alkyd paint to resist wear. Phase 2, a field evaluation of several proprietary high-heat paints in a transverse stripe, indicated a wide range of life expectancies to exist. Phase 3, a field evaluation of traffic delineation stripes in the proper longitudinal configuration with various types of glass beads applied at 3, 4, and 5 lb/gal (360, 480, and 600 kg/m^3), showed the Missouri type 2 floating bead to consistently rank high in performance. This investigation has produced a superior delineation system that provides economic savings over the system previously used.

•THIS study determined the effectiveness of the Missouri State Highway Commission standards pertaining to types of paint and glass beads and application rates used in paint and traffic delineation markings. The 3-phase investigation was completed with the minimum of samples by eliminating many variables. The optimum rate of application was the first major variable to stabilize, while, at the same time, a newly formulated cold-applied paint was tested. Next, various proprietary high-heat, fast-drying paints were tested for their resistance to wear. The wear tests were based on transverse stripes. The various types and application rates of glass beads were studied by placing longitudinal centerline stripes on divided highways having loadings of either 18,000 or 30,000 average daily traffic (ADT). This investigation has produced a superior delineation system that provides economic savings over the system previously used.

TEST LAYOUT

In phase 1, test stripes placed transverse to the flow of traffic in the driving lane were used to study the optimum wet film thickness of cold-applied paints that would provide a good service life. The 4-in. (46-cm) standard stripes were randomly placed at approximately 18-in. (0.50-m) intervals. The wet film thickness of each stripe was checked with an interchemical wet film thickness gauge as described in ASTM D1212-70. Drying time was checked with the discontinued ASTM dry-time apparatus assembly using a weighted rubber wheel as described in ASTM D711-55. Stripes were eliminated from the test if the quality of the line did not conform to a wet film thickness range within ±1 mil (±0.025 mm) or a specified width.

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Phase 2 used high-heat paints with essentially the same test procedures as phase 1. However, because of the configuration of the small striper, the bead flow could not be obstructed or interrupted to obtain wet film thickness values for each test stripe. The placement of the stripes required nonbeaded dummy stripes to calibrate the striper so that immediately thereafter the beaded test stripes could be placed with observation acceptance. These stripes were placed side by side, rather than randomly as in phase 1, in the same relative location within the test deck.

Phase 3 was designed to test the best paints applied at optimum wet film thickness in the actual centerline position. This phase also provided the opportunity to determine what type of glass bead and concentration would give the best night reflectivity relative to each other. The test sections were located on divided highways with 2 duplicate 1,200-ft (366-m) longitudinal centerline test sections per variable placed directly adjacent to each other in the opposing travel lanes. Missouri's standard stripe sequence is 15 ft (4.6 m) of white and 25 ft (7.6 m) of black or skip for divided highways. The stripes were placed to the left of the longitudinal joints or the old stripe to eliminate the variable of the condition of the old paint film. Wet film thickness measurements were taken for each test section by manually diverting the beads from one 15-ft (4.6-m) dash mark.

In all the phases, paints were placed on both portland cement concrete and asphalt concrete pavements.

EVALUATION PROCEDURES

Evaluation of the paint stripes was based on observations by a panel of 3 or 4 men, chosen for their experience with paint or paint operations. ASTM D713-69 covers basically the general modes of failure used for these evaluations: appearance, durability, and night visibility. Specific evaluation for durability as related to chipping and abrasion was done according to ASTM D913-51 and ASTM D821-47 respectively. In the initial testing to determine the optimum wet film thickness and the relative durability of the cold- and hot-applied paints, the night visibility observations were omitted. Basically, the performance tests were used to evaluate the portion of a transverse stripe within the limits of the inner and outer wheel paths. Even though these stripes were beaded, a meaningful reflectivity reading is almost impossible. Bead loss, however, was considered as an extra observation and was taken into account as a function of abrasion. Naturally, the glass beads provide some protection for the paint; therefore, the rate of loss of the glass beads will influence the rate of wear of the paint film.

Night visibility was incorporated into the evaluation procedures in phase 3. The effectiveness of the white paints, determined as being the most durable from phases 1 and 2, was studied to determine if they gave satisfactory delineation in daytime and at night. The test stripes were placed in the normal centerline condition on divided highways. Observation of the stripes in daylight involved the same procedures as previously mentioned for the transverse stripes. Observation at night was basically the same as that described in ASTM D713-69. However, by knowledge gained from a previous study of several paint stripes, the ASTM rating system of these test stripes for night visibility was augmented by a point-count survey method.

TEST RESULTS

Phase 1—Cold-Applied Paints and Optimum Wet Film Thickness

Phase 1 evaluated white and yellow Missouri standard dispersion resin-varnish paint (MS) and a chlorinated rubber-alkyd paint (CR) applied at 10, 15, and 20 mils (0.25, 0.38, and 0.50 mm) of wet film thickness. Both paints were purchased in accordance with specifications of the Missouri State Highway Commission.

Field evaluation was made approximately once every 2 weeks during the initial 9 weeks, then once every 3 weeks for the remainder of the test. The observers rated both the inner and outer wheel path portion of each stripe. Each test stripe was one of a set of four replicates. Observation of the original data showed no significant difference between the rating values for wheel paths; therefore, only the data obtained for the outer wheel path were analyzed. The different modes of failure considered were hard to distinguish at times. The angle between the sun, stripe, and observer was critical. Some of the deteriorations, especially abrasion, could not be detected because of the glare or washout of the finer worn spots. The failure due to chipping was easier to evaluate early in the test, but, when the abrasion failures became more predominant (i.e., when the substrate was exposed because of full-depth abrasion), a considerable amount of interference was encountered in making the evaluation. General appearance was the easiest mode of failure to evaluate and included both the abrasion and chipping losses as well as the discoloration and dirt retention of the stripe.

The complete statistical evaluations made on each of the modes of failure show that the general appearance rating was inclusive and contained basically the same significance levels as the abrasion and chipping contained. Data from the chipping and general appearance ratings best predict the actual life expectancy of the stripes. The abrasion ratings projected the life expectancy approximately 40 percent higher than actual. Therefore, the following discussion will be based mainly on data on general appearance.

Analysis of variance indicated no significant difference between observers; however, kind of paint, wet film thickness, age, color, and pavement type were significant. Significance above the 0.1 percent level was indicated. Reorganization of the data by successive elimination of specific variables showed that the kind of paint was significant. The CR had the best wear characteristics. Table 1, which shows the magnitude of this difference, gives the life expectancies in weeks for an ASTM failure rating of 3.0. The data also show the magnitude of the significant difference between pavement types.

The effect of thickness when studied in analyses of both types of paint was significant. This does not indicate the relationship between the thickness of each paint separately. Therefore, by evaluating the 2 paints separately on each pavement type for the 3 thickness ranges indicated, significance remained at the 0.1 percent level. The thickness variables for each paint type for each pavement were subdivided into linear and quadratic components to further evaluate the nature of the thickness relationship. This analysis showed that generally there is a linear relationship with both the MS and the CR. This indicates that life expectancy does increase so that near the end of the expected life of the stripes the 20-mil (0.50-mm) thickness is significant. However, the advantage of the 20-mil (0.50-mm) thickness does seem small (Table 1), particularly for the white paints, and this was of major concern for the striping standards at that time.

This study indicates the life expectancy of the white and yellow MS was increased by 6 and 12 percent respectively on concrete and by 12 and 9 percent respectively on asphalt when the wet film thickness was increased from 10 to 20 mils (0.25 to 0.50 mm). This was an overall increase of approximately 9 percent for both colors on both test pavements. The life expectancy of the white and yellow CR was increased by 40 and 32 percent respectively on concrete and 18 and 30 percent respectively on asphalt when the wet film thickness was increased from 10 to 20 mils (0.25 to 0.50 mm). This was an overall increase of approximately 30 percent for both colors on both test pavements.

Similar comparisons were made reflecting the magnitude of the effect of paint type by the general appearance at each paint thickness. The life expectancy of the CR at 10-, 15-, and 20-mil (0.25-, 0.38-, and 0.50-mm) thickness was 16, 20, and 38 percent greater respectively than that of the MR, and there was a somewhat greater increase in life for the yellow paint than for the white.

The magnitude of the effect of pavement type was higher than expected. A comparison of pavement type regardless of paint type shows that the asphalt concrete pavement has approximately 41, 41, and 35 percent greater life expectancy than the concrete for the 10-, 15-, and 20-mil (0.25-, 0.38-, and 0.50-mm) application rates.

Phase 2—Hot-Applied Paints

Phase 2 was a field evaluation of several proprietary types of hot-applied paints and was confined to the application of paint stripes in a similar manner as described for phase 1. The test methods and evaluation of these stripes were exactly the same as those for phase 1.

The 2 cold-applied paints from phase 1 were included for standards from which durability of the heated paints could be evaluated. A newly formulated CR that incorporated methylene chloride rather than methyl ethyl ketone as part of the vehicle was included in phase 2 for exploratory results only.

The life expectancies of the paints based on general appearance were used to evaluate these results. The wear resistance of the high-heat paints was definitely influenced by the type of pavement surface. The aggregate was exposed in both test locations; however, protrusion of the aggregate particles was much more pronounced in the concrete test deck. Failure of the test stripes generally began on the protruding aggregate particles. Because of this difference in rates of wear per pavement type, the statistical analysis considered the data obtained from each pavement type separately.

Terminal rating values of 3.0 and 4.0 were included in the analysis to determine if the failure rate was sufficiently established at the 4.0 level to accurately establish significance of the same order as that given at the 3.0 level. The results of statistical analysis and the relative order of significance for the paints considered show that the same identical order persists within a pavement type. However, a color reversal that exists between the pavement type with white on asphalt and yellow on concrete gives the longer life expectancy. Tables 2 and 3 give these data by color and pavement type. Comparing the ordering of the kinds of paint shows that color has a definite bearing on the wear resistance of a particular brand of paint.

These results show that, among the various proprietary high-heat traffic paints, paint type G consistently ranked high regardless of color or pavement type. The CR retained its superiority over the MS.

The high-heat paints conformed to all the required properties as shown in their specifications; however, further tests were made for information only. Laboratory analysis of the high-heat paints was conducted by using a Beckman IR-12 spectrophotometer for the infrared spectra for the vehicle and solvent portions. An Ortec nondispersive X-ray spectrometer and an ARL microprobe were also used to determine the properties of the pigment portion of the paint. Results of this study are given in Table 4.

The infrared spectra of all the paints except type O revealed that the vehicles were all modified phthalic alkyds. The short, medium, or long oil are based on comparison with known alkyds and are considered to be more probable designations. The major and minor components of the pigments are transcribed into the most probable chemical components by the results of the X-ray and microprobe analyses.

Phase 3—Glass Beads

Phases 1 and 2 provided the data for phase 3 in which the best of the cold- and hot-applied paints tested were used to determine the quality of various glass bead types and application rates.

Each test section consisted of 1,200 ft (366 m) of centerline stripe. All systems on the striper were checked and calibrated prior to application, and the wet film thickness and dry times of each paint were taken at the time of application.

Evaluation of the paint stripes was based on the same criteria as those for phase 1 and 2 stripes; however, the night visibility test was added to test the reflectivity of the glass beads. In this test, an automobile was positioned on the shoulder of the highway so that the left tires just touched the edge of the concrete pavement or just touched the white edge stripe on the asphalt concrete pavement. The front bumper was even with the leading end of a 15-ft (4.6-m) white dash mark. This dash mark was used as the beginning of the test section and as a reference point for all observations. A point-count

Table 1. Life expectancy, in weeks, of stripes calculated from regression slope values.

Paint Type	Asphalt 10 mil	Asphalt 15 mil	Asphalt 20 mil	Concrete 10 mil	Concrete 15 mil	Concrete 20 mil
General Appearance						
MS white	13.2	13.9	14.8	9.7	9.7	10.3
CR white	13.7	14.6	16.1	10.5	12.3	14.7
MS yellow	13.7	15.2	14.9	9.2	9.0	10.0
CR yellow	17.3	17.9	22.4	11.8	12.5	15.6
Abrasion						
MS white	19.2	21.0	24.3	19.8	19.2	22.6
CR white	20.5	21.9	28.5	19.3	25.4	28.7
MS yellow	21.6	24.4	25.1	16.8	18.5	25.4
CR yellow	25.9	33.3	37.0	25.0	23.9	35.9
Chipping						
MS white	17.6	18.9	20.6	11.1	11.2	11.9
CR white	17.7	19.5	23.8	12.6	15.4	17.8
MS yellow	17.5	18.8	18.5	10.5	10.2	11.9
CR yellow	23.6	26.7	32.7	14.8	15.9	20.2

Note: 1 mil = 0.025 mm.

Table 2. Analysis of variance for white and yellow paint on concrete and asphalt pavements for general appearance rating of 3.0.

Source	Degrees of Freedom	Sum of Squares	Mean Square	F-Value	Significance Level	Significance	$k_{0.01}$
Asphalt pavement White paint							
K	6	99.17	16.53	65.07	0.001	—[a]	1.1[b]
R	35	8.88	0.25				
Total	41	108.05	2.64				
Asphalt pavement Yellow paint							
K	6	863.59	143.93	2,767.90	0.001	—[a]	0.5[b]
R	35	1.80	0.05				
Total	41	865.39	21.11				
Concrete pavement White paint							
K	6	402.63	67.11	113.55	0.001	—[a]	1.7[b]
R	35	20.69	0.59				
Total	41	423.32	10.32				
Concrete pavement Yellow paint							
K	6	370.98	61.83	792.69	0.001	—[a]	0.6[b]
R	35	2.74	0.08				
Total	41	373.72	9.11				

[a]Highly significant. [b]Difference in means required to indicate a significant difference.

Table 3. Ranking for various high-heat, yellow and white paints on concrete and asphalt pavements.

Pavement	Paint Color	Ranking Paint Type	Ranking Average	Pavement	Paint Color	Ranking Paint Type	Ranking Average
Asphalt	White	G	14.8	Concrete	White	G	10.6
		I	14.4			CR	8.4
		K	14.2			I	7.9
		CR	13.6			K	6.4
		M	12.6			MS	6.3
		MS	11.8			M	2.1
		O	10.2			O	1.4
Asphalt	Yellow	I	15.6	Concrete	Yellow	K	10.5
		G	15.3			G	9.6
		K	14.2			CR	9.2
		CR	14.0			M	7.2
		M	12.7			I	5.6
		MS	11.0			MS	4.8
		O	1.5			O	1.4

method of testing night visibility was devised and found to be much easier to justify than the ASTM visual rating system.

The life expectancies of each set of test stripes based on ASTM method are given in Table 5 for each of the observed modes of failure.

The relationship between observers and between pavement types was nonsignificant for observers and significant for pavement types.

The night visibility rating system (Table 5) represents the ASTM method with averages computed by two methods. These data, as a whole, indicate that night visibility is the major controlling source of failure of the three modes shown. The weighted rating does, in a few instances, have a lower rating value than the night visibility. Generally, the same ordering will occur regardless of which rating is used; however, the significance between the various test sections may change. The following analyses will be based on the data shown for night visibility calculated by averaging the interpolated values between observations per observer.

The failure values for the cold-applied paints with the type 1 glass bead applied at 5 lb/gal (600 kg/m^3) indicate the following to be significant. The CR that used methylene chloride as part of the vehicle is poor in its ability to maintain a satisfactory night reflectivity. CR did maintain a respectable daytime appearance and durability rating; therefore, the probable cause of failure was an excessive loss of glass beads.

The CR did show a greater life expectancy over the MS on the asphalt concrete pavement. However, on the concrete pavement there was no difference in these 2 paints. There was a considerable difference between the life expectancy of the same paint and pavement type. The asphalt concrete provided higher values than the concrete by approximately 2 to 1. These data indicate that the possibility of selective striping by pavement types could be considered.

The failure values for the high-heat paints with the type 1 glass bead applied at 5 lb/gal (600 kg/m^3) indicate the following to be significant. The high-heat paints are the same brand of paints as those in phase 2. This analysis shows the type G paint to be the best regardless of pavement type. However, the pavement surface does not seem to have the same magnitude or ordering effect as that shown for the cold-applied paints. The concrete shows the largest life expectancies for 3 of the 4 paints. The fact that these paints are located in 30,000 ADT areas as compared with the cold-applied paints in 18,000 ADT areas may indicate that the surface characteristics are less important as traffic volume increases. The lower life of the paints placed in the higher volume locations was attributable to the loss of glass beads.

The effects of type and concentration of glass beads were studied by using 4 bead types applied at 3, 4, and 5 lb/gal (360, 480, and 600 kg/m^3). The evaluation by type consisted of placing each bead type on asphalt and concrete pavements in the 18,000 ADT and 30,000 ADT areas. This allowed all types of beads to be tested with the best of the cold- and hot-applied paints previously tested. The type 1 and 4 beads were gradation drop-on beads; type 1 conformed to the standard Missouri type 1 specification, and type 4 conformed to a gradation suggested by several manufacturers. The type 2 and 3 beads were floating beads; type 2 conformed to the standard Missouri type 2 specification, and type 3 conformed to the same gradation without the floating compound added. The application rates were evaluated by using either the concrete or the asphalt pavement as given in Table 6.

The performance of type 1, 2, 3, and 4 beads with the cold-applied paints (Table 6) for the 5-lb/gal (600-kg/m^3) application rate on US-50 and US-54 definitely indicates that types 3 and 4 are not significantly different on either pavement. Bead type 2 is significantly better relative to the other beads on concrete; however, on asphalt, the type 1 bead is best.

The performance of type 1, 2, 3, and 4 beads with the hot-applied paints (Table 6) for the 5-lb/gal (600-kg/m^3) application rate on I-55 and I-70 shows that, for concrete pavement, all types are significantly different from each other, and type 2 gives the best performance on concrete and a performance as good as any of the others on asphalt.

These results indicate that the type 2 bead, when applied in concentrations equal to the other types of beads, consistently ranks higher in performance. The type 2 bead shows the best performance on the concrete pavements and is significantly better than

Table 4. Generic composition of high-heat paints.

Paint Type	Vehicle	Solvents	Pigments: Major Components	Pigments: Minor Components
G yellow	Oil-modified phthalic alkyd, short to medium oil	Trichloroethylene, commercial mineral spirits	Si, Pb, Ca, Cr	Al, Fe
G white	Oil-modified phthalic alkyd, medium to long oil	Trichloroethylene, commercial mineral spirits	Ti, Ca	Al, Si
I yellow	Oil-modified phthalic alkyd, medium to long oil	Methylene chloride, commercial mineral spirits	Pb, Ca, Cr, Al, Si	Fe
I white	Oil-modified phthalic alkyd, medium to long oil	Methylene chloride, commercial mineral spirits	Ca, Ti, Si	Al, Fe
K yellow	Oil-modified phthalic alkyd, medium to long oil	Trichloroethylene	Ca, Pb, Cr	Si, Zn, Fe
K white	Oil-modified phthalic alkyd, medium to long oil	Trichloroethylene	Ca, Ti, Si, Zn	Fe, Cr, Al
M yellow	Oil-modified phthalic alkyd, short to medium oil	Toluene, xylene	Ca, Si, Pb, Cr	
M white	Oil-modified phthalic alkyd, medium to long oil	Toluene, xylene	Ca, Si	Ti, Cr, Fe
O yellow	Oil-modified polyamide	Toluene	Ca, Pb, Cr, Si	Al, Fe
O white	Oil-modified polyamide	Toluene	Ca, Ti	Si, Al, Cr, Fe

Table 5. Failure values for each test section.

Test Section	Paint Type	Bead Code	Bead Concentration (lb/gal)	Failure (weeks) at 3.0 Rating: Appearance[a]	Durability[a]	Night Visibility $\bar{X}$[a]	Night Visibility $\bar{X}$[b]	Failure (weeks) at 4.0 Weighted Rating
District 5, US-50, Concrete Pavement								
T-1	B	1	5	33.32	34.14	22.42	22.30	28.06
T-2	D	1	5	32.00	33.63	21.14	20.89	26.50
T-3	D	1	4	31.98	35.70	22.50	22.33	27.62
T-4	D	1	3	33.18	35.33	21.51	21.22	26.86
T-5	D	2	5	33.35	35.62	33.33	33.01	30.64
T-6	D	2	4	34.12	35.46	29.24	29.11	29.06
T-7	D	2	3	37.52	43.39[c]	21.30	21.33	25.40
T-8	D	3	5	31.22	39.21	19.73	21.78	24.11
T-9	D	4	5	30.91	37.25	20.00	19.71	23.66
T-10	F	1	5	28.27	34.76	10.00	11.37	19.98
District 5, US-54, Asphalt Concrete Pavement								
T-1	F	1	5	23.50	24.72	16.71	16.56	17.82
T-2	B	1	5	50.26[c]	50.25[c]	45.28[c]	45.28[c]	39.44
T-3	D	1	5	49.67[c]	49.13[c]	56.54[c]	56.54[c]	46.19[e]
T-4	D	2	5	35.92	38.51	34.50	33.67	33.80
T-5	D	3	5	26.44	33.77	25.14	25.17	22.40
T-6	D	4	5	23.90	29.42	25.79	26.31	20.86
District 6, I-55, Concrete Pavement								
T-1	B	1	5	26.87	28.22	15.36	15.33	19.78
T-2	O	1	5	28.22	29.25	25.81	25.67	25.67
T-3	K	1	5	32.84	35.97[d]	6.33	6.38	15.76
T-4	G	1	5	36.39[d]	40.78[d]	29.00	29.00	29.78
T-5	G	2	5	43.16[d]	49.29[d]	38.96[d]	38.96[d]	39.84[e]
T-7	G	3	5	48.92[d]	54.54[d]	32.08	32.06	32.71
T-8	G	4	5	27.06	28.41	21.57	21.53	21.22
District 6, I-70, Asphalt Concrete Pavement								
T-2	B	1	5	26.09	27.67	13.51	13.33	19.51
T-3	O	1	5	33.14	44.18[d]	11.67	10.10	25.62
T-4	G	2	5	33.65	48.37[d]	27.59	27.51	30.88
T-5	G	2	4	51.63[d]	56.41[d]	29.92	29.67	36.91[e]
T-6	G	2	3	53.92[d]	58.74[d]	27.54	27.40	33.84
T-7	G	1	5	45.94[d]	48.35[d]	27.67	27.67	31.12
T-8	K	1	5	29.21	34.66[d]	10.00	7.00	21.25
T-9	G	1	4	46.54[d]	47.83[d]	24.28	22.00	28.30
T-10	G	1	3	46.34[d]	42.81[d]	23.67	22.61	27.23
T-11	G	4	5	53.73[d]	52.21[d]	22.20	21.58	29.37
T-12	G	3	5	82.88[d]	59.92[d]	26.20	26.11	31.39

Note: 1 lb/gal = 120 kg/m^3.

[a]Determined by interpolation between average observer test section ratings of 2 or 3 observers/observation.
[b]Determined by averaging the interpolated values between observations per observer.
[c]After 40 weeks exposure, still rated higher than 3.0. Value extrapolated by regression analysis.
[d]After 34 weeks exposure, still rated higher than 3.0. Value extrapolated by regression analysis.
[e]Values computed as of termination of test did not fall below 4.0. Value extrapolated by regression analysis.

the next best bead. On the asphalt concrete pavement, the type 1 bead is significantly better than the type 2 bead when used with the cold-applied paints. However, considering the failure values, the type 2 bead shows the most consistent rate of failure.

The difference between the application rates of the type 1 bead with the cold-applied paints on US-50 (Table 6) was not significant. The maximum difference in mean failure was less than 2 weeks. The type 2 bead at 5 lb/gal (600 kg/m^3) had a significantly longer life than at 4 lb/gal (480 kg/m^3), and both of these were significantly better than at 3 lb/gal (360 kg/m^3).

The difference between the application rates of the type 1 bead with the hot-applied paints on I-70 (Table 6) shows that 5 lb/gal (600 kg/m^3) is significantly better than 4 or 3 lb/gal (480 or 360 kg/m^3), and there was no significant difference between the latter two. The type 2 bead, however, shows no significant difference based on application rates.

The results show that the type 1 bead at 5 lb/gal (600 kg/m^3) and the type 2 bead at either 5 or 4 lb/gal (600 or 480 kg/m^3) consistently rank high in significance. The most consistent results were obtained with the type 2 bead at 4 lb/gal (480 kg/m^3).

ALTERNATE METHOD FOR ESTABLISHING NIGHT VISIBILITY RATING

The method of rating traffic delineation stripes by night visibility, as stated in ASTM D713-69, may be done with the human eye and judgment or by photographic methods in which the rating is based on a factor of 10 for the highest reading and 0 for complete failure. It was difficult for the rating team to mentally adjust from observation to observation on this type of subjective rating. Therefore, another system of night visibility rating, which had been used previously in an exploratory test, was also included.

The alternate (point-count) method of rating the stripes for night visibility was also included in this phase to provide for comparison of the 2 methods. These centerline stripes were evaluated by positioning an automobile on the shoulder of the highway so that the left tires just touched the edge of the concrete pavement or just touched the white edge stripe on the asphalt concrete pavements. The front bumper was even with the leading end of a 15-ft (4.6-m) white dash mark. This dash mark was used as the beginning of the point-count method and was established as stripe 1. However, for the point-count method, the observers positioned themselves immediately in front of the automobile, standing between the headlights rather than sitting in the front seat of the automobile. The first observation was made with the low-beam light, and the second observation was made with the high-beam light. The observer, for each light setting, made a point count of how many 15-ft (4.6-m) dashes, including the one used to place the automobile in position, could be clearly and distinctly seen. Each observer decided when to stop counting. It was accepted that this point would be when the stripe no longer gave a clear distinct white appearance and faded to dull gray. The observers consistently felt more comfortable with this rating system than that of the subjective 10 to 0 ASTM rating. Both ratings were made for all test sections in this phase of this investigation. From the number of observations made and the large number of test sections included in this phase, a large quantity of data were made available for analysis. Regression analyses made from the high-beam point-count ratings for each pavement type for the cold- and hot-applied paints versus the ASTM night visibility ratings showed good similarity. This suggested a complete regression analysis be made by using all 307 pairs of data. The resulting regression equation is shown in Figure 1. Because this equation was shown to have a standard error of estimate of only 0.59 and a coefficient of variation of only 12.5 percent, this alternate method does appear to have greater promise than was initially anticipated. From this analysis of the high-beam point-count ratings there does appear to be a less subjective method available whereby night visibility observations may be made by less experienced people who are not familiar with the ASTM method.

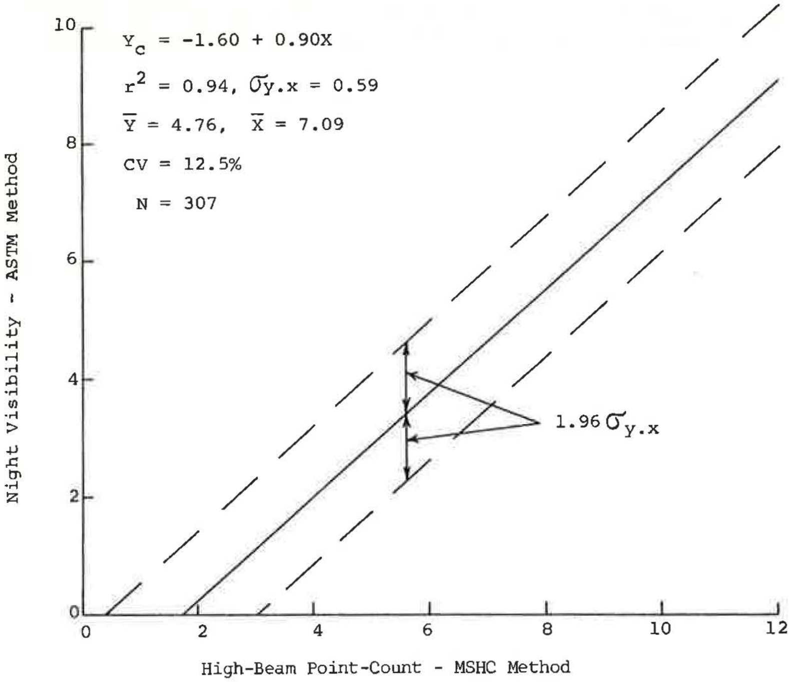
The low-beam point-count ratings were taken with each observation. It became apparent near the latter stages of the investigation, however, that a leveling off of the

Table 6. Ranking of beads and concentrations by failure in weeks and significant differences in mean failures.

Bead Type	Concentration (lb/gal)	Mean Failure (weeks)
US-50, Concrete Pavement, Paint D		
2	5	33.01
2	4	29.11
1	4	22.33
3	5	21.78
2	3	21.33
1	3	21.22
1	5	20.89
4	5	19.71
US-54, Asphalt Concrete Pavement, Paint D		
1	5	56.54
2	5	33.67
4	5	26.31
3	5	25.71
I-55, Concrete Pavement, Paint G		
2	5	38.96
3	5	32.06
1	5	29.00
4	5	21.53
I-70, Asphalt Concrete Pavement, Paint G		
2	4	29.67
1	5	27.67
2	5	27.51
2	3	27.40
3	5	26.11
1	4	24.47
1	3	22.61
4	5	21.58

Note: 1 lb/gal = 120 kg/m^3. Vertical lines connect variables for which no claim can be made for any difference in their behavior at 0.01 level.

Figure 1. Combined regression analysis of high-beam point-count and ASTM night visibility ratings.



ratings occurred around the third and fourth dash marks, and the high-beam point-count ratings continued to deteriorate. Because of this inconsistency, the low-beam point-count ratings were omitted and considered inadequate for use as a night visibility rating method.

CONCLUSIONS

Phase 1 proved that both white and yellow cold-applied CR were significantly better than MS when placed as transverse stripes on asphalt and concrete pavements. The stripes placed on the asphalt pavement had durability ratings significantly better than those placed on concrete pavement for both paints at all thickness ranges.

The significance between application rates of the paints indicates there is a linear relationship for durability for both the MS and CR. This indicates that as thickness increases the life expectancy of the stripe increases; however, the actual difference in life expectancy of the MS is only increased by 1 percent for each 1-mil (0.025-mm) increase in application rate. The CR showed a 3 percent increase in life expectancy for a 1-mil (0.025-mm) increase in application rate.

Transverse stripes placed on the asphalt surface had approximately a 40 percent increase in life expectancy over that for the concrete surface. The CR consistently showed greater life expectancy than the MS in all thickness ranges on all pavements.

Phase 2 proved that the relative durabilities of several proprietary high-heat paints were definitely a specific characteristic of the kind and color of paint and of the pavement type to which it was applied. However, one particular type of paint was consistently higher ranking than the other paints.

Phase 3 proved that the overall performance of the paint-bead system is best evaluated by the night visibility mode of failure. The cold- and hot-applied paints continued to substantiate the previous work in this study. The overall performance of the Missouri type 2 floating bead was better than the Missouri type 1 gradation bead. Glass bead types 3 and 4 had the lowest life expectancies. The type 2 bead at 4 lb/gal (480 kg/m^3) showed the most consistent results. The less subjective point-count method of rating the appearance of a stripe at night was included in phase 3 as a possible alternate method to the ASTM night visibility rating method. This method may be used by relatively inexperienced observers for determining when restriping is necessary.

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