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FOREWORD

In his two papers, Stratfull continues to provide information from the extensive and intensive studies of half-cell potential measurement as an indicator of existing or impending corrosion. In recent years Stratfull's work has received increasing attention and evaluation by several agencies.

The data from these most recent reports emphasize that the technique, when properly interpreted, is a valuable tool in the laboratory experiment on the influence of various materials factors on the protection of concrete from the damaging effect of corrosion as well as in the early detection of corrosion in the field.

Wrbas, Ledbetter, and Meyer present data from a sweep of slabs variously cured in artificially created environments intended to simulate those encountered in the field. The results confirm the detrimental effects of high curing temperature on surface properties. Although adequate strengths were obtained in all cases, there are differences attributable to several curing techniques. Procedures previously developed and promoted for estimating evaporation from surfaces are called into question by the data from this work.

Weed uses data from a series of experiments to develop a rule-of-thumb for estimating a reduction of stress that accompanies increase in entrained air for the range of air content application to highway construction. These data are also used to develop criteria by which rejection rates can be predicted for concrete with various levels of air entrainment.

DiCocco describes the use of statistical control charts and process controls for acceptance of the product. The important distinction between these two functions, not always clearly understood, is emphasized. The conclusion drawn is that control charts are valuable and assist in process control but that judgments as to acceptance are better made by using acceptance sampling plans based on lots, rather than the process, which is the focus of control charts.

The papers in this RECORD will be of interest to those involved in the production and use of concrete and to those research workers studying such activities. As is often the case with research on a material as complex as concrete, it is subject to local variations in materials and environment. In some cases these papers confirm previously published works reporting on similar studies, whereas in others they call into question past results. This is to be expected, and emphasis is indicated for continued diligence in both the conduct and implementation of research findings.

-Howard H. Newlon, Jr.

CORROSION AUTOPSY OF A STRUCTURALLY UNSOUND BRIDGE DECK

Richard F. Stratfull, California Division of Highways

An investigation was performed on a 12-year old salt-contaminated, reinforced concrete bridge deck that had to be replaced because of its deteriorated condition. In this investigation, the electrical half-cell potential measurements and effect of chlorides present seem to be related to some threshold amount that changes the steel from a passive to an active state. Beyond this point, the amount of salt present has little or no effect except as it might influence the area of corrosion involved. The chaining or sounding of the deck to locate delaminated concrete performed the function very well but did not necessarily locate the corroded steel. From the observation of the type of cracking, it appeared that the final mode of distress was concrete fatigue. An investigation of actual concrete cover disclosed that there was reinforcing steel corrosion at depths greater than 3 in. It was determined that estimating the pit depth of steel by visually estimating the thickness of rust is not a very useful inspection technique. In this highly salt-contaminated bridge deck, no relation was found between variations in the chloride content of the concrete and the relative severity of the corrosion of the steel.

•AFTER approximately 12 years of service, a highway bridge deck needed replacement because of the corrosion of reinforcing steel. The corrosion was associated with other factors, such as thickness and quality of cover and presence of salt and moisture, as will be discussed. Concrete cracking had advanced to the degree that falsework was necessary to prevent structural failure. As a result, it was decided to make a comprehensive investigation of the condition. This consisted of making electrical potential measurements (1, 2), chaining (3) or sounding (4) the concrete for delamination, measuring the chloride content of the concrete, determining the relation between amount of rusty steel and metal loss, and measuring the water absorption and strength of the concrete.

Although there are many reports in the literature (5) concerning bridge deck deterioration, there are no known reports that describe a comprehensive corrosion evaluation of a bridge that became structurally unsound and required deck replacement.

STUDY SUMMARY

Electrical Potential Measurements

The electrical potential measurements were effective in indicating that the corrosion of the steel was of far greater extent than that which would be indicated by the measured area of concrete delamination. This type of measurement should always be included in any investigation of the corrosion of steel in a bridge deck.

The electrical potential measurements are not considered a reliable indicator of the rate or amount of corrosion of the steel. However, in general terms, the more extensive the area of active potentials, the more probable is the greater amount of corrosion.

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This is because it takes time for a large area of corrosion to develop. Therefore, with a longer period of time for corrosion to be active, it is reasonable to assume that the corrosion loss would be greater. The half-cell potentials can be plotted on statistical distribution paper and used to "log" the condition of the bridge according to the percentage of active potentials. For example, for the bridge deck replaced, 94 percent of the potentials were measured to be active, whereas, for an adjacent bridge in which about 2 percent of the deck was repaired, only 30 percent of the values were found to be active.

Concrete Delamination

The chain (3) proved to be an effective and workable sounding device for locating undersurface fractures or concrete delaminations. However, care must be exercised so as not to misinterpret the hollow sound as always representing corrosion-caused concrete delaminations. In this investigation, one location that was thought to be a corrosion-caused concrete delamination actually turned out to be a location of disbonded epoxy membrane overlay. Delamination might also be the result of freeze-thaw action or separation of a grout layer from the underlying concrete due to poor bond. In this bridge, just prior to replacement, approximately 20 percent of the total concrete deck surface was found to be delaminated.

Probable Cause of Deck Unsoundness

Although the cause of the structurally unsound deck is considered to have been accelerated by a somewhat porous concrete, it appears that the principal factors leading to the unsoundness are as follows:

1. De-icing salts were absorbed by the concrete, which resulted in the corrosion of the steel.

2. The concrete was highly absorptive, which apparently resulted in some degradation of concrete strength, perhaps by a combined effect of salt crystallization within the concrete and freeze-thaw action. High water-cement ratios can greatly increase absorption and decrease strength of otherwise good concrete.

3. Corrosion of the steel caused concrete delamination or undersurface fractures and thus reduced the structural section of the deck to a point below that of the top mat of reinforcing steel.

4. As evidenced by the "alligator" type of cracking on the underside or soffit of the deck, the final stage of deterioration appeared to be fatigue failure of the slab, which is a result of "live" loading.

Rust Rating and Pit Depth

An attempt was made to determine how well the actual corrosion pit depth would correlate with a visual observation of the quantity of rust on the steel. On a scale of 0 (no rust) to 5 (Table 1), based on visual observation of rust scale and metal loss of the deformations on the reinforcing steel, only a poor correlation could be made. Therefore, metal loss should be measured, not estimated by rust thickness.

Chloride Content

The chloride content of the concrete in the range encountered was not a positive indicator of deterioration as evidenced by the comparison of the similar amount of deterioration in the two spans that had different salt contents. The significance in a chloride analysis seems to lie within the spectrum of simply determining that its concentration is sufficiently great so as to cause steel to change from a passive to an active state and thus be susceptible to corrosion. Beyond this point of chloride concentration, the control on the incidence of rust or corrosion rate will depend mostly on the moisture content of the concrete (1). Even if there are massive quantities of salt in the concrete, the steel will not actively corrode unless sufficient moisture is present. However, it is assumed that the extent of repairs would depend on the overall level of the chloride content. For example, in this structure, the chloride content was as great as 21 lb of chloride ion per cubic yard of concrete. Therefore, the use of a membrane to inhibit further intrusion of the salt into an already highly contaminated concrete would be useless.

Concrete Cover

The specified concrete cover over the steel for this bridge was $1\frac{1}{2}$ in. By actual measurement, it was found that the average concrete cover was about 2.0 in. with a range of 1.5 to 3.0 in. Corroding deck reinforcing steel was found not only in the top mat of steel where the concrete cover was as great as 3 in. but also in the lower mat of steel where the concrete cover was greater than 6 in. This could occur as the result of salt water reaching the steel by flowing downward through cracks.

Concrete Compressive Strength and Absorption

As given in Table 2, the compressive strength of the concrete was somewhat less than the strength level normally expected for 12-year-old concrete, and it was variable. The relatively low concrete strength values coupled with the relatively high volumetric absorption of the concrete indicate that the deterioration of the structure was accelerated by a concrete quality initially inadequate for a freeze-thaw and salting exposure. This mechanism for the deterioration seems to be confirmed by the fact that span 3, with its lower strength concrete, showed distress; however, falsework prevented actual failure. Span 1, with an equal amount of concrete delamination, did not show distress. However, it is not necessarily true that, even with a better concrete quality, the service life could be greatly extended.

General Conclusions

No one investigative technique was found to be adequate to answer all questions concerning the cause or extent of the deterioration. In specifically dealing with the corrosion phenomenon, the electrical half-cell potentials and the chloride analysis of the concrete appear to be the most important for determining the extent and level of corrosion activity. The electrical potential measurements have been previously found to be useful for predicting the locations of new concrete spalls (1). Conversely, for determining the area and location of advanced structural damage resulting from a concrete delamination, the drum-like sound of the chain when dragged over delaminated concrete was found to be the easiest and most rapid technique for locating undersurface fractures or spalls.

Because corrosion was found in all previous repairs, the epoxy mortar repairs did not inhibit further corrosion of the steel. Because corrosion was found when the concrete cover was as great as 3 in., it is apparent that an even greater depth of cover would be required for the concrete quality found in this bridge for the protection of the steel from salt.

BRIDGE HISTORY

The Truckee River Bridge is a three-span bridge of approximately 100 ft per span with a traveled way of 28 ft curb to curb. It carries two lanes of eastbound traffic, and the deck slopes to the right. It is a welded steel composite girder structure with four girders per span and reinforced concrete pier and wing abutments. All of the re-inforced six-sack concrete was designed to contain 4 to 6 percent entrained air. The concrete aggregate was blended from a single source; the coarse aggregate $(1\frac{1}{2})$ -in. maximum size) had a water absorption of about 2.8 percent by weight, whereas the fine aggregate had a water absorption of about 4 percent by weight.

The average 28-day concrete strength for the 11 bridges being constructed at the same time was 3,800 psi. There was only one test report for one concrete cylinder that could be positively identified as coming from the Truckee River Bridge. The 28-day compressive strength for the one concrete cylinder was 2,940 psi.

The cement used was an ASTM Type II of low-alkali content meeting California highway specifications, which in various shipments either was ground to ordinary range of fineness or was finely ground to provide high early strength. The records are not clear as to when the types of cement were used, but it is assumed that the fine type was used during periods of low ambient temperatures to accelerate early strength gain.

The bridge is located in the Sierra Nevada Mountains at an elevation of approximately 5,500 ft above mean sea level. The average rainfall is reported as about 24 in. Most of the precipitation is in the form of snow; the average annual snowfall is about 170 in. The temperature range in the area is from about +95 F to -41 F as measured in 1949. The frost penetration in the soil was anticipated to be at a 4-ft depth. As a consequence, the bridge and roadway are heavily salted during the winter season due to snowfall and frost. This bridge was completed in 1959 and was inspected at least annually by an engineer as part of a regular inspection program. Additional inspections were made as warranted. A review of the reports of inspections draws an interesting picture of the progressive deterioration of the deck of the structure.

In the report dated September 1960, it was observed that several large transverse cracks have opened up in all three spans as well as numerous other smaller cracks in the deck. In September 1961, it was observed that the transverse cracks in the three spans were of medium size in span 3 and of small size in the other two spans. Random pattern cracking was of medium size in all spans.

In September 1963, it was reported that, after only four winters, there was considerable pattern cracking of the soffit or the bottom of the deck of span 3 between the third and fourth girders. The following summer, the bridge was overlaid with an epoxy-sand seal. In August 1967, the soffit cracking in span 3 was described as severe. Leaking of water through cracks in the deck extended for more than 20 ft, and cracking was spreading to the deck areas between the second and third girders. The engineer estimated that this area of the deck would require replacement in 1 to 2 years. It was also noted in the 1967 report that seven areas of the deck had spalling and that the soffit of span 1 was starting to show cracking.

In May 1968, the soffit cracking in span 3 had increased to the point that its structural soundness was in question, and the placement of timber supports was recommended. The placing of the timber supports as well as the filling of 35 new deck spalls with an epoxy mortar and the placing of a new epoxy-sand seal in the slow traffic lane were completed during the summer of 1968. The epoxy mortar was made with a ratio of 5 parts pea gravel to 1 part epoxy by volume (1).

Additional deck spalling occurred and was repaired prior to the October 1969 report, in which it was recommended that the 11 additional spalls be repaired and the bridge deck be scheduled for replacement.

FIELD WORK

Initially, a reference grid was laid out on the deck on a 4-ft square pattern. These points were spotted on the deck using spray paint. The deck was then chained to delineate the areas of unsound or delaminated concrete. A solid ringing sound is normally heard as the chain is dragged on sound concrete, but there will be a dead or drum-like sound when delaminated areas are encountered. The delaminated areas were outlined on the deck using spray paint, and then they were plotted on cross-section paper for correlation with other operations. The existing deck patches of epoxy mortar and asphalt concrete, as well as the areas of the deck being supported by timber falsework, were also plotted on this sheet. Figures 1 and 2 show portions of spans 1 and 3 with delaminated areas being shaded and patched areas crosshatched.

Electrical potential measurements using a saturated copper-copper sulfate half-cell were taken on the 4-ft grid pattern on the top of the deck with additional readings at anomalies. These readings were reduced to contours and overlaid on the plot showing the delaminated areas. Figures 1 and 2 show these contours as a dashed line.

Twenty-two 4-in. cores were taken through the deck at various locations. The core locations were chosen to include all stages of deterioration and all ages of epoxy mortar patches. These locations were further chosen to include deck reinforcing. All cores were identified and their locations recorded. They were then analyzed for compressive strength, absorption, and chloride content.

The first phase of the deck removal consisted of removing the concrete from both sides of both exterior girders by striking the concrete with a pneumatic-mounted hydro-hammer. This operation exposed the reinforcing steel from the curb lines to about 2 ft out into the traveled way. It was observed that a fairly large percentage of this steel showed from minor to extensive corrosion. However, the chaining had indicated very little concrete delamination along these areas even though the half-cell potential readings indicated active corrosion.

During the concrete removal, the actual amount of concrete cover over the steel was measured. Along the left (facing the direction of traffic movement) edge of the deck, it was found that the average concrete cover over the steel was 2.36 in. with a range of 2.0 to 3.0 in. The concrete cover over the steel along the right edge (lower) of the deck averaged 2.25 in. with a range of 1.5 to 2.75 in. These measurements were made with a ruler.

Along the middle of the deck, a "pachometer" was used for measurement, and the indicated average amount of concrete cover over the steel was 1.78 in. with a range of 1.63 to 2.0 in.

Three selected slabs, approximately 7 by 12 ft, from specific areas of the deck being removed were set aside for recovery of the reinforcing steel. The slabs were from (a) an area of little delamination or cracking of the concrete, (b) an area that showed severe cracking on the soffit and epoxy patches of different ages on the top of the deck, and (c) the timber-supported area that was structurally unsound. Upon removal, the location of the reinforcing steel was identified. A detailed corrosion evaluation of the steel was made. The locations of these three test slabs are shown in Figures 1 and 2.

The abutments and piers were examined visually for evidence of corrosion-related deterioration. Pier 3 showed the only visual evidences of corrosion in that there was corrosion-caused spalling of concrete on the top and bottom of the pier cap. The top of the pier cap sloped with the cross slope of the deck, and corrosion-caused deterioration was found only on the low end of the cap, where salt-contaminated deck drainage water would flow. Electrical potentials were taken on the top of the cap. At one location the average of four readings was -0.24 V, and there was no visible evidence of corrosion. At another location, the average potential was -0.53 V, and corrosion of the steel and concrete spalling were observed.

Samples of concrete were taken in the area of high potentials for chloride analysis. The results of this analysis showed that the concrete at the top of the cap contained the equivalent of 31 lb/yd^3 of chloride ions and at the bottom contained 10 lb/yd^3 .

CONCRETE QUALITY

Twenty-two 4-in. diameter cores were taken through the deck for laboratory analysis, 10 cores from span 1 and 12 from span 3. Ten cores were in areas of deck spalling that had been repaired with epoxy mortar. All 10 cores in patched areas and 8 cores in unpatched areas were taken so as to include deck reinforcing. The reinforcing was found to be corroding at all epoxy patches. When making epoxy repairs, it is required that the steel be sandblasted; therefore, it is assumed that any corrosion products present must have been generated as a result of corrosion occurring after the time of repair.

Four cores, two each from spans 1 and 3, contained neither epoxy patches nor reinforcing steel. They were tested for 28-day absorption using California Test Method 538-A. These four cores were then checked for compressive strengths (Table 2), which appear to be lower than would be expected for 12-year-old concrete. Table 2 gives the 28-day volumetric absorption of the concrete between 14.09 and 16.61 percent. In previous testing, the author has generally observed that, in similar concrete mixes, the volumetric absorption would be in the range of 13.5 to 15.0 percent. The greater absorption of the bridge deck concrete is probably influenced by the absorptive aggregates. The 22 cores were then analyzed for chloride ion intrusion. Each core was cut into 1-in. thick disks that were pulverized and analyzed using a "wet" analysis for control and X-ray diffraction (Table 3).

The data given in Table 3 show that a significant quantity of chloride ion has been absorbed by the concrete. Even though the chloride content was greater in span 1, there was no difference in the area of delamination between this span and span 3, which was structurally unsound.

In previous work (1), it was observed that, as long as the salt content was sufficient to support corrosion, then the presence or absence of corrosion depended on the moisture content of the concrete.

The threshold value of salt as measured for previous work was found to be about 1.5 lb of chloride per cubic yard of concrete. Localized distribution can of course vary greatly.

It seems clear that, once the minimum level of salt has been reached to cause corrosion, even greater concentrations play no further significant role in the corrosion of the steel. This is emphasized by comparing the chloride concentrations in the deck of this bridge, about 13 lb, to a previously reported (1) bridge that was corroding and eventually removed and only contained an average of about 1.5 lb of chloride ion per cubic yard at the level of the steel.

HALF-CELL POTENTIALS, CONCRETE DELAMINATION, AND RUSTING STEEL

In previous work (6), it was shown that, for half-cell potential values more negative than about -0.35 V to the copper-copper sulfate half-cell (Cu-CuSO₄), the steel in concrete appears to be active as a result of salt intrusion. The value range of -0.30 to -0.35 V seems to be an inconclusive area, whereas for values less negative than -0.30 V (Cu-CuSO₄) the steel is passive or chemically inhibited from corrosion.

To determine the relation among the half-cell potentials, rusting steel, and concrete spalls or delamination, the bridge was completely surveyed by electrical potential measurements, chained to find loose or delaminated concrete, and visually observed to locate any evidence of rusty steel.

Figures 1 and 2 show that the high potential measurements on nearly the whole deck surface indicated a massive area of active corrosion of the steel. Although not shown in Figures 1 and 2, visual observations of the steel during the concrete removal show significant areas of rusting steel even though the concrete in these areas showed no indication of spalling or delamination.

For the entire bridge deck about 94 percent of all of the half-cell potentials of the steel were in the active or corroding range. In contrast, approximately 20 percent of the deck area was found to be delaminated or to have undersurface concrete fractures at the top layer of the steel.

Figure 3 shows the electrical potentials and the concrete delamination that were measured on the relatively good slab. All of the half-cell potentials are in the active range of -0.35 to -0.55 V and denote the likelihood of significant corrosion of the steel. In Figures 3, 4, and 5, the locations of steel that had visible rust were plotted as solid lines. If the top mat of reinforcing had no rust, it was not shown.

Figure 3 shows an undersurface delamination that apparently is in a location of unrusted steel. Because this area of concrete still had the epoxy-aggregate membrane on the surface, it is believed that the hollow sound that caused a recording of a concrete delamination might have been the result of disbonding between the epoxy membrane and the underlying concrete surface.

As shown in Figures 3, 4, and 5, the relation between rusted steel and concrete delaminations can be poor. This is not to say that the corroding steel does not cause delaminations, but it does indicate that the amount of rust that forms on steel causing a spall can be highly variable. It is obvious that the sounding of concrete only relates to the condition of the concrete and not necessarily to the condition of the steel.

Figure 6 relates the half-cell potential measurements to the observed condition of the bridge by showing the cumulative frequency distribution on three bridges. The upper

Table 1. Rusting in test deck slabs.

	Rust Rating (percentage of length of steel)					
Slab Number	1	2	3	4	5ª	No Rust
1 (good condition)						
Top transverse bars	13	3	4	5	4	71
Top longitudinal bars	<1	1	-	-	-	99
Bottom longitudinal bars	-	-	-	-	-	100
Bottom transverse bars	<1	1	-		-	99
2						
Top transverse bars	12	3	4	3	37	41
Top longitudinal bars	8	2	1	1	7	81
Bottom longitudinal bars	1	<1	1	1	3	94
Bottom transverse bars	9	3	2	<1	<1	86
3 (unsound)						
Top transverse bars	7	2	6	4	62	19
Top longitudinal bars	5	3	5	8	32	48
Bottom longitudinal bars	1	1	2	3	14	79
Bottom transverse bars	18	5	9	11	7	50

*Rust rating 5 represents heavy rusting.

Figure 1. Part of span 1.



Crosshatched area: epoxy patches Dashed line: electrical potential contours

Figure 2. Part of span 2.



Table 3. Average chloride-ion distribution as a function of sla	slab depth.	unction of	as a	distribution	de-ion	chloride	Average	3.	Table
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0	Depth (lb chloride-ion/yd ³ concrete)									
Span Number	0 to 1 In.	1 to 2 In.	2 to 3 In.	3 to 4 In.	4 to 5 In.	5 to 6 In.	6 to 7 In.	7 to 8 In.		
1	23.5	13.2	4.4	1.3	1.4	0.9	1.3	0.5		
3	18.3	12.4	2.9	0.9	0.6	0.7	0.3	-		

Table 2. 28-day absorption-compressive strength of 4-in. concrete cores.

Core Location Number		Compres- sive Strength (psi)	28-Day Absorption (percent/ volume)
Span 1	2	3,690	16.61
Span 1	3	4,080	16.38
Span 3	11	3,020	14.42
Span 3	20	2,695	14.09

Figure 3. Slab showing little delamination.



Figure 5. Slab from area of timber supports.



Crosshatched area: epoxy patches Dashed line: electrical potential contours Solid line: corroded top reinforcing steel

Table 4. Visual rusting and actual pitting of steel and concrete.

	Pit Depth (mil)		
Rating	Range	Avg.	Visual Condition
1	0 to 46	15	Trace to light rust
2	0 to 57	17	Medium to heavy rust
3	3 to 58	18	Verv heavy rust
4	5 to 57	22	Light pitting and corrosion of deformations
5	21 to 97	52	Heavy pitting and corrosion of deformations

Figure 4. Slab showing severe cracking of soffit and epoxy patches.



Figure 6. Distribution of half-cell potentials.



Table 5. Average potential values found in test slabs.

	Slab				
Characteristic	1	2	3		
x (v)	-0.44	-0.47	-0.53		
Active potential (percent)*	91	98	100		
Concrete delamination (percentage of total area)	4	34	58		
Top bar rust (percentage of total length)	29	50	81		
Rust of all bars (percentage of total length)	19	28	55		

^aPercentage of total measurements that exceed -0.35 V, Cu⁻CuSO₄.

distribution curve shown in Figure 6 is for the bridge under investigation; the middle distribution curve is for another bridge. For the bridge deck that required replacement (lower curve, Fig. 6), about 94 percent of all of the potential measurements made showed that the steel was active (more negative than -0.35 V, Cu·CuSO₄) or corroding. For the deck that was repaired, only 30 percent of the measurements showed active half-cell potentials, and the new deck had no measured active potentials.

The data shown in Figure 6 may reflect a means for "logging" the condition of a bridge according to the percentage of active or passive potentials. However, the distribution curve can have a break; therefore, one must exercise caution in mathematically calculating a mean or standard deviation for the half-cell potentials.

REINFORCEMENT

The reinforcing steel removed from the three slabs recovered from the deck was evaluated as to degree of corrosion on a visual rating of 1 to 5 (Table 1). The slabs contained four layers of steel, two layers of top steel, the upper layer being transverse and the bottom being longitudinal, and two layers of bottom steel, the upper being longitudinal and the bottom being transverse. Truss bars were considered top steel when in the top plane and bottom steel when in the bottom plane.

The data given in Table 1 show that the amount and severity of rusting increased with the severity of the original physical condition of the test slabs. Slab 1 was from the area in better physical condition, whereas test slab 3 was from the area supported by falsework. In test slabs 2 and 3, there is a significant length of bottom reinforcing that was found to have significant rusting.

The relation of a visual rating of rust to the actual amount of metal loss was determined by sandblasting random pieces representing each visual rust rating and measuring maximum pit depths. It become obvious that estimating and logging degrees or amounts of rust can be misleading. Pit depths for 20 samples of each visual rating varied as given in Table 4.

The amount of metal loss to produce a visual volume of rust is highly variable as given in Table 4. This further illustrates the lack of a significant relation between delaminated concrete and rusted steel. In concrete of low absorption where small amounts of rust will cause high disruptive pressure (no absorption of the rust products by the adjacent concrete), the relation between rusty steel and concrete delamination should be of a high order.

Table 5 gives the average potential, \overline{X} , the percentage of the potentials that were active, the area of concrete delamination, and the percentage of rusted steel, which all increased in value commensurate with the increase in proportion to the distress observed in the slabs. The half-cell potentials were a better indicator of the area of rusted steel than was the area of concrete delamination. This is emphasized in the data from slab 1, where 91 percent of the half-cell potentials measured were found to be in the active range, whereas only 4 percent of the concrete was delaminated by rusting of the top steel, which was coated with rust for 29 percent of its length.

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The contents of this report reflect the views of the author, who is responsible for the facts and the accuracy of the data presented herein. 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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DISCUSSION

Bernard Erlin and W. G. Hime, Erlin, Hime Associates

The author has presented an excellent paper on the use of electrical measurements for the determination of hidden corrosion. Our comments are initiated by the title of the paper, which appears to indicate a more complete "postmortem" than was undertaken.

We feel that, although the efforts to detect those concrete areas where corrosion had occurred were probably complete, the determination of the basic underlying cause namely, the initial character of the concrete and its subsequent condition—is mandatory to an autopsy if a full understanding of cause and effect is to be attained. A more detailed study of the concrete would thus have been warranted.

Such a study, centered around a complete petrographic examination of the concrete, could have provided a wealth of information that might resolve the following questions:

1. Did freeze-thaw damage occur, and, if so, was it a contributor to, or a cause of, steel corrosion (through initiation of microcracks and consequent rapid migration of chloride ion)? Was the concrete actually air-entrained, as had been required by specifications?

2. Was the water-cement ratio of the concrete mix high? Were there features that would indicate the reason for the deep penetration of chloride and the reported variations of chloride concentration with depth?

3. Were there differences in concrete properties that would explain the relatively dissimilar performances of spans 1 and 3?

4. Because the high pH of concrete ordinarily protects steel except in the presence of extremely large amounts of chloride, could the observed corrosion be related to carbonation and/or chloride levels?

It would be of value for the author to elaborate on the chloride analyses because of the varieties of methods available. For example, if the chlorides were extracted with water, the amount of chloride extracted might be up to 60 percent incomplete, depending on the age of the concrete. We note that the author stated that he also used X-ray diffraction for such analyses. Current X-ray diffraction methods are entirely unsuitable for quantizing chlorides in concrete. On the other hand, X-ray spectrographic methods may be suitable. We believe, therefore, that, if an X-ray method was used, then it was X-ray spectrography.

AUTHOR'S CLOSURE

The comments by Erlin and Hime are appreciated; however, they appear to believe that the corrosion of reinforcing steel is an unusual circumstance that is prompted by either an initially poor or subsequently deteriorated concrete. Such is not necessarily the case.

The reason that a petrographic analysis was not performed is that the results are not directly applicable to the corrosion behavior of steel in concrete. This is not to say that steel will not corrode in concrete that has suffered distress due to an environmental attack, but steel can and does corrode in the best quality concrete without any degradation of the concrete itself. We have ample evidence of corrosion of reinforcing steel where there is no possibility of freeze-thaw damage to the concrete.

The following are the answers to Erlin and Hime's specific questions:

1. Yes, test procedures on the construction project verified that the concrete was air-entrained as required by the specifications. Visually, there was no evidence to indicate any significant freeze-thaw damage of the concrete. Microcracking by any cause is not necessary for the penetration of chloride ion into concrete.

2. It is regretted that construction records cannot be located that confirm the actual water-cement ratio of the concrete mix. However, based on an average 28-day concrete compressive strength of 3,800 psi for the total of 11 bridges built at the same time, it would seem that the water-cement ratio was not above normal limits. A review of the literature will show that the chloride ion readily penetrates all normal concrete and the concentration decreases with depth. Even in well-controlled concrete, the absorption characteristics to concrete vary greatly from point to point, especially on decks, which could result in highly variable saltwater penetration with time.

3. The only significant difference between spans 1 and 3 was that the lower strength concrete in span 3 resulted in apparent earlier fatigue failure in the latter span.

4. I have never observed carbonation of the concrete at a depth of 2 in. below the surface at the location of the reinforcing steel. Moreover, I am not aware of any reports by others claiming to find evidence of carbonation at such depth with the type of concrete used on this project.

The relation between chloride ion and corrosion that was found in this study was simply that a sufficiently great concentration of chloride ion penetrated to the surface of the steel and caused the steel to corrode. No relation was found between various concentrations of chloride and the degree of corrosion.

In addition, let me add that a modified Volhard method with an acid extraction was used. Also, I do not use X-ray diffraction, but the X-ray secondary fluorescence method, which is commonly called X-ray emission or X-ray spectrography.

HALF-CELL POTENTIALS AND THE CORROSION OF STEEL IN CONCRETE

Richard F. Stratfull, California Division of Highways

The half-cell potential of steel embedded in concrete specimens in laboratory tests was periodically measured and related to the visual observation of concrete cracking. It was observed that, when the half-cell potential values were more negative than -0.45 V to the saturated calomel electrode, 60 percent of the reinforced concrete blocks were cracked from the corrosion of the steel. At values between -0.27 and -0.42 V, the steel was corroding but not always enough to cause concrete cracking. In cracked concrete, the maximum half-cell potential of the steel was measured to be -0.59 V. In addition to the laboratory tests on small specimens, a prototype-simulated bridge deck was exposed outdoors to periodic wetting and drying of a chloride salt solution, and half-cell potentials were measured by using various techniques. It is shown that, once corrosion begins, the measurements will show the potential gradients of the resulting corrosion currents irrespective of the technique used to obtain them. However, there was a significant difference in the level of the potentials, and that level was clearly associated with the method of electrical measurement.

• PREVIOUS work $(\underline{1-6})$ has demonstrated that the half-cell potential of steel in concrete is a valid indicator of corrosion activity. In effect, measurements $(\underline{2}, \underline{4})$ of halfcell potentials have identified steel that is noncorroding (passive) when a measured value is numerically less than -0.22 V relative to the saturated calomel electrode (SCE) and corroding (active) when the value is numerically greater than -0.27 V (SCE). Between -0.22 and -0.27 V, the condition may be either active or passive.

Although an active potential of the steel does not correlate with a rate of corrosion, it is known (2, 4) that, with an increasing amount of corrosion, the numerical value of the potential also increases. Therefore, because there is concern (7) about cracking of concrete caused by rusting steel, an attempt was made to find a half-cell potential value that is indicative of the amount of steel corrosion that can cause concrete to crack and to explore some of the various techniques used to obtain half-cell potentials.

In this regard, data are given from two different tests. One test measured a single half-cell potential value for reinforced concrete that is partially immersed in a saturated solution of sodium chloride. The value in this type of test is that the half-cell potentials clearly show the noncorroding or passive state and conversely the active or corroding state of the steel.

The second test measured the half-cell potential and the potential gradients on the surface of a corroding simulated bridge deck. The half-cell potentials that are obtained on the simulated bridge deck are similar to those that would be obtained on an actual field structure. Four different techniques were used to measure the electrical potentials on the simulated bridge deck. These measurements show how the level of the measured potential can be affected by the reference electrical "ground."

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The laboratory test data indicate that, when the measured half-cell potential of steel in concrete test blocks was numerically greater than -0.42 V (SCE), approximately 60 percent of 137 blocks were cracked by the rusting steel. Of the concrete blocks that were cracked by the rusting steel, the maximum measured half-cell potential was -0.59 V (SCE).

A trend in the data indicated that the measured half-cell potential of the steel that caused concrete cracking decreased as the cement factor of the concrete was increased. The variation in the potential may possibly be the result of higher electrical resistance due to an increase in the cement factor of the concrete.

The most common cement factors used in structural concrete are between 6 and 8 sacks of cement per cubic yard. The data for concrete mixes within this range indicate that a significant percentage (about 50 percent) of the concrete blocks were observed to be cracked by corrosion when the half-cell potential was numerically greater than about -0.42 V (SCE).

However, based on 95 percent of the observations of the half-cell potential of steel, the least negative potential associated with corrosion-caused concrete cracking was -0.31 V (SCE).

The results of this laboratory work indicate that a nondestructive means for measuring or detecting corrosion activity of steel can be useful for evaluating reinforcing concrete structures exposed to aggressive environments. For example, half-cell potential values that are numerically less than -0.22 V (SCE) indicate passive or noncorroding steel; for values that are numerically greater than -0.27 V (SCE) but less than -0.42 V (SCE), the steel is active but would not, on the average, be expected to have rusted enough to cause visible concrete distress except at a lower level of probability. For potential values that are numerically greater than -0.42 V (SCE), the laboratory tests showed that there is better than a 50 percent probability that goodquality concrete cover of about 1-in. thickness will have been cracked by the rusting steel, whereas concrete cracking can be expected when the potential is at least as negative as -0.31 V (SCE). However, it is important to recognize that the following must always be considered when relating concrete cracking and the half-cell potential of the steel:

1. The half-cell potential of steel can only be empirically related on a statistical basis to concrete cracking under specific conditions,

2. The half-cell potential of steel does not measure the physical or structural condition of concrete, and

3. The cracking of concrete due to the corrosion of steel is related to concrete strength, absorption, moisture content, stresses, and its thickness over the steel.

Therefore, as these latter variables change, so will the empirical relations between concrete cracking and the half-cell potential of the steel. Although it may be hazardous to use empirical relations, this laboratory work confirms that of Kliethermes (7) when he reported on the inspection of 120 exposed concrete decks from 33 states. Kliethermes found bridges to be in good condition when 90 percent of all potential readings were less negative than -0.22 V (SCE); other studies (2, 4) have shown that, in more than 99 percent of the tests, the steel was noncorroding or passive when the potential was less negative than -0.22 V (SCE). Also, Kliethermes reported that the bridge decks were in poor condition, i.e., spalled and cracked, when the potential readings were more negative than -0.32 V (SCE), whereas in this study approximately 97 percent of the cracked concrete blocks had a potential more negative than -0.32 V (SCE).

Therefore, although the empirical relation between the potentials and concrete distress can have a degree of accuracy, it must be looked on as a guide that should be tempered by more specific tests and, most important, by the judgment of the engineer. Previous laboratory work has shown (2) that, at the time of concrete cracking, the average pit depth in a $\frac{1}{2}$ -in.-diameter embedded steel bar that had corroded enough to cause cracking was approximately 10 mils (0.01 in.). Because the measurements of the half-cell potential depend on an electrical circuit, variable measurements may be obtained due to the conductivity of the concrete. For example, with a surface dry concrete, the contact resistance of the half-cell to the concrete may be so great that an erroneously low potential value may be measured.

As shown by the various equipotential contours on the bridge deck test slab, care must be exercised both in obtaining the half-cell potentials and in interpreting them.

In order to evaluate the condition of a concrete structure in which all of the reinforcing may not be electrically interconnected, half-cell potentials were made on a simulated bridge deck slab where in the proper lead to the voltmeter was connected to electrically and nonelectrically connected steel. In addition, a test was made where the voltmeter was not connected to the steel but to another stationary half-cell on the surface of the concrete so as to demonstrate the feasibility of making measurements without an electrical connection to the steel.

The results of these tests show that, for all measurement techniques used in this study, the same corrosion-caused voltage gradients were measured on the concrete surface whether or not a direct electrical connection was made to the reinforcing steel. However, there was a significant difference in the level of the potentials that was clearly associated with the referenced electrical "ground."

CONCLUSIONS

In light of the foregoing, the following conclusions can be drawn:

1. Electrical potential measurements can indicate active or passive steel condition.

2. Differences in the electrical half-cell potentials are associated with the "solution potential" of the steel as well as the voltage gradients resulting from current flow.

3. In a voltage gradient, the measured half-cell may not necessarily reflect the true half-cell potential of the most proximate steel because the voltmeter can only indicate the highest voltage at that point. For example, two pieces of steel may be in close proximity to the point of measurement, one corroding and the other not corroding. The voltmeter will only indicate the highest voltage present, and thus there will be no indication of the presence of noncorroding steel.

4. To detect corrosion-caused electrical current flow, it is not always necessary to electrically connect the voltmeter to the reinforcing steel.

5. The best measure of the electrical half-cell potential is a direct electrical connection to the steel under consideration.

6. Under the condition of electrical current flow, all half-cell potential measurements will be distorted by the arithmetic difference of the associated voltage gradients.

FABRICATION OF TEST BLOCKS

The variables of concrete manufacture used in this series are given in Table 1. The river-run aggregate was $\frac{3}{4}$ in. maximum size, and the gradation complied with the 1964 standard specifications of the California Division of Highways. The cement used was ASTM Type II, modified, low-alkali, which also complied with the 1964 standard specifications.

The reinforced concrete specimens were $4\frac{1}{2}$ in. wide, $2\frac{1}{2}$ in. thick, and 15 in. long (Fig. 1). The $\frac{1}{2}$ -in.-diameter steel bar was cast in the concrete to provide a nominal 1 in. of concrete cover at any point. A binding post was used to make an electrical connection from the steel bar to a recording voltmeter (Fig. 1).

Out of each batch of concrete specimens fabricated, one-half were steam-cured for approximately 16 hours at 138 ± 5 F and then post-cured in water for 28 days. All steam-cured specimens were held in their molds for a minimum of 4 hours prior to steam-curing. In all cases, the concrete was steam-cured on the day that it was mixed. The other half of the batch of specimens was cured by being completely immersed in water at a temperature of approximately 72 F for 28 days after 1 day of curing in the molds.

After the concrete curing period, the still wet specimens were partially immersed for a depth of $3^{1}/_{2}$ in. into a saturated solution of sodium chloride.

Figure 2 shows the typical testing layout for the concrete blocks. Note the multiconductor plug and cable arrangement for making electrical connections to the voltage recorder.

Electrical half-cell potentials of the steel as referenced to an SCE were made and recorded thrice weekly.

On approximately a 10-day cycle, the concrete blocks were removed from the testing tanks and visually inspected for evidence of concrete cracking. No concrete block was out of the test tank for more than 1 hour at a time.

TEST RESULTS

Figure 3 shows a typical potential record. As indicated, the half-cell potential assumes a low or passive value soon after being partly submerged in the tank and remains low until the chloride ion permeates the concrete and reaches the surface of the steel. It then causes the steel to become active. The initially high potential of the steel is caused by the film of water at the interface of the concrete where the steel projects into the atmosphere. Drying of this surface stops the active corrosion at that point, and the measurements then reflect those values for concrete-embedded steel.

The measured half-cell potential values for the 5-, 6-, and 8-sack concrete specimens are given in Tables 2, 3, and 4. The active potential values given were first measured after the salt was assumed to have reached the steel (a large jump in value). Also given are the measured potential values at the time cracking was first observed by inspection, which was usually some time later.

As shown in Figure 4, the half-cell potential of steel at the time of active potential, and the time when the concrete has been observed to have cracked, tends to numerically decrease with increasing cement factor. Also, for concrete of the same cement factor, the half-cell potentials are numerically greater in concrete that has been steam-cured. For a given cement factor there is a significant increase in the half-cell potential of the steel between the time when it is first measured to be active and the time when a crack in the concrete is first observed.

As shown in Figure 5, the average half-cell potential of the steel when it was first observed to be active was -0.36 V (SCE). The average half-cell potential of the steel in these same specimens, when cracking of the concrete was observed, was -0.47 V (SCE). Although there is a considerable overlap in the potentials, it is apparent that there is an upper range of potentials, which indicates a significant possibility of concrete cracking and significant rusting of the steel before cracking is observed. The possibility of detecting relatively large amounts of corrosion before cracking occurs is most significant.

Also, as shown in Figure 5, approximately 62 percent of the cracked concrete specimens had a potential that ranged between -0.45 and -0.59 V. Therefore, at a potential of -0.45 V or greater, rust-caused concrete cracking was observed more often than not. However, as shown in Figure 4, the half-cell potential of the steel at the time of concrete cracking, and also at the time of initial activity, significantly decreases in numerical value when the cement factor is increased from 5 to 6 sacks of cement per cubic yard. Because structural bridge concrete in California contains at least 6 sacks of cement per cubic yard, only those data for the 6- and 8-sack mixtures were used in preparing Figure 6. For 6- and 8-sack concrete, as indicated in this figure, at a half-cell potential of -0.42 V (SCE), approximately 95 percent of the first measured active potentials had a lower numerical value. At a later date when the concrete was observed to be cracked, approximately 50 percent of the rust-caused cracked concrete specimens had a half-cell potential greater than -0.42 V (SCE).

In this regard, Kliethermes (7) recently reported on the half-cell potentials and physical condition of 120 bridge decks in 33 different states. A half-cell potential of -0.42 V (SCE) is about equal to -0.50 V (CuSO₄) used by Kliethermes (7). He reported

Figure 2. Partial immersion testing of steel in concrete.

Figure 1. Test specimen.





Table 1. Concrete mix variables.

Comont	Nominal	Ain	Unit	Mixing Water (lb/ft ³)	
Factor	(in.)	(percent)	(lb/ft ³)	Gross	Net
5.03	2 ¹ / ₄	1.5	152.1	325	280
6.02	31/4	1.8	151.0	368	321
8.24	31/2	1.6	152.5	374	331

Table 2. Half-cell potential values for 8-sack concrete specimen.

	Moist-Cur	е	Steam-Cure	
Factor	Active Potential (mV)	Concrete Crack Potential (mV)	Active Potential (mV)	Concrete Crack Potential (mV)
θ sacks of cement per cubic yard	290	330	360	390
	440	540	385	440
	405	305	385	470
	450	460	330	420
	305	435	340	325
	310	465	385	415
	435	425	385	485
	395	490	365	410
	315	445	340	490
	300	400	395	505
	330	320	305	385
	355	355	340	430
	_	-	315	385
	-	-	290	400
	270	360	330	540
	300	350	290	400
	365	405	295	445
	305	450	295	350
	-	-	345	515
	350	465	285	395
Potential in millivolts (SCE)	348	412	338	430
Coefficient of variation (percent)	17	17	11	13
95 percent confidence limits, $\overline{\mathbf{X}}$ = mean	30	34	17	26



Figure 4. Cement factor and half-cell potential.



Steam-Cure

Table 3. Half-cell potential values for 6-sack concrete specimen.

Factor	Active Potential (mV)	Concrete Crack Potential (mV)	Active Potential (mV)	Concrete Crack Potential (mV)
6 sacks of cement per cubic yard	340	310	320	400
an proteined a traditional and a part and the court	350	400	340	530
	300	405	375	545
	265	460	360	560
	330	390	410	450
	375	430	360	380
	390	440	435	535
	350	345	355	435
	330	360	380	420
	375	405	400	550
	280	510	330	510
	400	500	320	530
	370	500	270	470
	295	435	340	505
	330	450	290	475
	270	345	370	390
	320	540	380	500
	360	505	320	460
	330	520	370	550
	300	530	370	400
Potential in millivolts (SCE)	333	439	355	480
Coefficient of variation (percent)	12	16	11	13
95 percent confidence limits, $\overline{\mathbf{X}}$ = mean	18	32	19	28

Moist-Cure

Table 4. Half-cellpotential values for 5-sackconcrete specimen.

	Moist-Cure	е	Steam-Cure	
Factor	Active Potential (mV)	Concrete Crack Potential (mV)	Active Potential (mV)	Concrete Crack Potential (mV)
5 sacks of cement per cubic yard	381	569	410	533
	466	560	356	441
	472	568	416	502
	396	532	371	467
	395	510	391	548
	440	577	385	467
	455	479	401	496
	334	580	380	541
	447	585	433	501
	469	572	396	574
	390	578	340	586
	420	581	407	585
	370	579	445	524
	351	504	399	529
	419	455	419	570
	388	570	415	567
	359	557	426	588
	404	557	353	558
	419	503	476	522
	317	448	432	576
	311	501	383	536
	348	518	403	506
	344	399	428	519
	387	433	346	559
	287	477	369	481
Potential in millivolts (SCE)	391	528	399	531
Coefficient of variation (percent)	13	10	8.2	7.7
95 percent confidence limits, $\overline{\mathbf{X}}$ = mean	21	22	14	17

that, for the 50 bridge decks that were in good condition with only minor cracking, the measured potential value of the steel did not have a numerical value greater than -0.50 V (CuSO₄). For 31 additional bridge decks that were rated "fair-cracked," approximately 5 percent of the decks had a measured potential that numerically exceeded -0.50 V (CuSO₄). For the 39 bridge decks that were rated "poor-spalled-cracked," about 50 percent of the bridges had potential values that exceeded -0.50 V (CuSO₄).

From this field report, it is obvious that the half-cell potential of the steel is a meaningful nondestructive technique that can be used to evaluate the condition of a reinforced concrete bridge structure with regard to active corrosion of the steel and corrosion-caused cracking of the concrete.

FABRICATION OF SIMULATED BRIDGE DECK SLAB

A reinforced concrete slab, 6 ft long and 7 ft wide, was cast containing the normal amount and position of reinforcing steel as found in bridge decks. Although two layers of steel were used, for simplicity, only the top mat of reinforcing steel is shown in Figures 7, 8, 9, and 10. The transverse reinforcing bars, No. 5's, are on 11-in. centers, and the longitudinal reinforcing steel bars, No. 4's, are on 18-in. centers. In addition, truss reinforcing bars were placed in the transverse direction on 11-in. centers. The lower mat of reinforcing steel is the same as the top mat; however, it was spaced a little off-center as normally found in bridge design.

One electrically isolated reinforcing bar is located in the top mat. The location of this one bar not touching the other steel in the mat is noted in Figure 8 by the word "ground." Also included in this test slab is a $\frac{1}{4}$ -in. round by approximately 2-in. long steel probe that is embedded into the concrete with 4 in. of concrete cover. This steel probe has a wire connected to it that is brought out through the concrete to the surface to facilitate electrical measurements. The location of this probe is shown in Figure 9. It also is not connected electrically to the other embedded steel.

The concrete contained $\frac{3}{4}$ -in. maximum-sized aggregate and the gradation was in accordance with the 1970 California standard specifications. The thickness of the slab is slightly less than a typical bridge deck slab, $5\frac{5}{4}$ in. The penetration of the fresh concrete was $1\frac{1}{4}$ in., and the regular cone slump was about $2\frac{1}{2}$ in. The concrete had a designed cement factor of 6 sacks of cement per cubic yard and was vibrated into place with normal vibration techniques.

The concrete cover over the top mat of steel was 1 in. After the slab was cast, it was wet-cured for 28 days. After wet-curing, it was allowed to dry for a period of approximately 6 months. A berm to hold ponded water was then constructed of wood and plastic. Thereafter, 8.6 lb (about 2 lb/yd^2) of sodium chloride were spread on top of the slab, and water was added. No further additions of salt were made and only periodic applications of water. Loss of salt during rainy weather was prevented by use of a waterproof cover.

Essentially, part of the water on the surface of the slab either evaporated into the air or was absorbed by the concrete. Once the slab surface became dry, it was reflooded with water, and cycling continued. In general, the slab would be flooded for about 1 week and dry for about 2 weeks. The first potential measurements were made prior to the first application of salt and water. Active potentials began to appear about 10 days after the salt was applied.

TESTING PROCEDURE

The slab surface was marked off at 6-in. intervals longitudinally and transversely. On this grid pattern of 6 in., center-to-center, potential measurements were made using a high-impedance voltmeter. One electrical connection was made to the SCE, which was touched to the surface of the concrete at the grid intersections. The other electrical connection to the voltmeter was made in four different ways:

1. The potential measurements were referenced to all of the reinforcing steel (Fig. 7);

2. The potential measurements were referenced to a single, long, electrically insulated reinforcing bar (Fig. 8); Figure 5. Distribution of half-cell potentials, all concrete.



Figure 7. Equipotential contours referenced to all reinforcing steel.



Figure 9. Equipotential contours referenced to isolated probe.



Figure 6. Distribution of half-cell potentials, 6- and 8-sack mixtures.



Figure 8. Equipotential contours referenced to insulated rebar.







3. The potential measurements were referenced to an electrically isolated steel probe that is embedded 4 in. down in the concrete (Fig. 9); and

4. The potential measurements were referenced to an SCE that was left in place on top of the slab (Fig. 10).

Using voltage readings from each series of measurements, equipotential contours were drawn at 50-mV intervals.

TEST RESULTS

Figures 7, 8, 9, and 10 are equipotential contour maps for the simulated bridge deck obtained by using the four different methods of measurements described.

For simplicity, the word "ground" is used to denote the location of one electrical connection. However, the equipotential contours shown in Figures 7, 8, 9, and 10 are actually referenced to the electrical connection of the reinforcing mat, the rebar, etc. Therefore, it will be noted on the four figures that the level of the potential contours varies in accordance to the technique for making the measurements, but the general location of the contour lines is the same for each method of measurement.

Figure 7 shows the contour intervals when the half-cell was referenced to embedded reinforcing steel mats. As shown by the contour intervals, there are variations of the half-cell potentials of the steel caused by the corrosion of the steel and the associated flow of electrical current.

In Figure 8, the half-cell potentials were obtained by grounding the voltmeter or referencing the half-cell to a single, long, electrically isolated piece of reinforcing steel in the concrete. As will be noted, there is a great similarity between potential contours that were obtained even though two methods for grounding were used.

The contours shown in Figure 9 were obtained from measurements made when one terminal of the voltmeter was connected or grounded to the small, electrically insulated probe embedded in the slab. The equipotential contours were then drawn using this probe as a reference for the measurements. It will be noted that again there is a great similarity among the equipotential contours obtained by the three methods of grounding as shown in Figures 7, 8, and 9.

In order to clearly demonstrate the feasibility of obtaining potential measurements without an electrical connection to the steel, two SCE's appropriately connected were used to obtain electrical potential measurements. If there were no corrosion-caused current flow in the concrete, the measured voltage between these two half-cells would always be essentially zero irrespective of where either half-cell was placed on the concrete. The location of the stationary half-cell is shown by the arrow in Figure 10. This half-cell was left in place and never moved during the entire period of the measurements. The other half-cell was moved about to the 6-in. grid locations. The measurements were then used to draw the contours shown in Figure 10. As will be noted, the contours are similar in Figures 7, 8, and 9. Although the contours are similar, the potential values are not. Therefore, it is obvious that care must be exercised in making electrical connections to determine the true value of the electrical potential of the steel. This is particularly true if the measurements are to be used as an indicator of the presence of active or passive corrosion conditions.

The general procedures and techniques described can be adapted to field survey methods (7). Grid spacing is increased to keep the number of measurements to a practical level, but smaller spacing is necessary to delineate small active areas.

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EFFECTIVENESS OF MEMBRANE CURING ON CONCRETE SURFACES

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The data obtained for this research were taken from 19 sidewalk-sized test slabs (26 by 24 by 8 in. thick). Variables investigated included three environments (73 F and 25 percent RH, 100 F and 30 percent RH, and 140 F and 25 percent RH), four curing methods (white pigmented curing compound, monomolecular film followed by white pigmented curing compound, watersoluble linseed oil, and no curing compound), and three wind velocities (0, 8 to 10, and 18 to 20 mph). Constants included mix design (5 sacks per cubic yard of concrete), mixing temperature, mixing procedure, placement, finish, and cure time. In all cases adequate strengths were obtained, but curing temperatures of more than 100 F resulted in a significant reduction in the strength of the top portion of all concrete slabs, even though adequate curing methods were used. At temperatures of more than 100 F, the surfaces cured with the combination monomolecular film (one application before final finish) followed by white pigmented compound showed a high abrasion loss compared to the surfaces cured with either water-soluble linseed oil or white pigmented compound by itself. Thus, there appear to be no surface strength benefits from the one application of the film before finishing. Evaporation of water from the surface of the slabs was significantly retarded with the use of any of the curing compounds. Evaporation rates measured experimentally in this study did not agree with the values predicted by the PCA chart, especially the rates when wind was present. Thus, the validity of a portion of the PCA chart is questioned.

•THE surface properties of portland cement concrete pavement are affected by the combined effects of wind velocity, air temperature and relative humidity, concrete temperature, and type of curing compound. Properties of concrete, such as resistance to freezing and thawing, strength, water tightness, wear resistance, and volume stability, improve with age so long as conditions are favorable for continued hydration of the cement. The improvement is generally rapid at early ages but continues at a slower rate for an indefinite period. For proper cement hydration, there must be the continued presence of moisture and a favorable temperature (1). Hydration virtually ceases when concrete dries below a relative vapor pressure (relative humidity) of about 0.80 (2). At this pressure the water-filled capillaries begin to empty. Because hydration occurs only in these water-filled spaces, hydration ceases when the capillaries begin to empty; therefore, the effective curing time is confined to that period during which the relative humidity in concrete remains above 80 percent. If saturated concrete is placed in saturated air, it will not lose weight; however, if it is placed in air in which the vapor pressure is even slightly below that of saturated air, the concrete will lose water by evaporation. When the vapor pressure of the atmosphere changes, the moisture content of the concrete changes also; it rises with a rise in humidity and vice versa. Concrete sealed against evaporation must initially contain

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about 0.5 gram of water per gram of cement to ensure full hydration, though complete hydration is seldom if ever achieved because self-desiccation progressively reduces the space available for hydration products (2).

Because of increased labor costs and rapid construction pace, an increasing number of concrete highway pavements are being cured with membrane-forming curing compounds (ASTM Designation C 309). Although it is recognized that the moist-curing methods (ponding, sprinkling, and wet coverings) best ensure continued cement hydration, membrane-cured concretes have given creditable performance (3, 4). Field and laboratory tests have been conducted to evaluate several combinations of curing and protective treatments for concrete. A study conducted by the Virginia Highway Research Council evaluated concrete panels cured with white pigmented liquid membrane and white polyethylene sheet, both with and without subsequent treatments using linseed oil (5). On some concrete panels a monomolecular film was used to reduce evaporation prior to regular curing. Results showed linseed oil (in mineral spirits) to be the most satisfactory of the several alternatives practically available for improved durability. Application of the linseed oil following curing with a white pigmented resin-based compound of the type specified by the Virginia Department of Highways was again shown to be satisfactory. Finally, procedures for the use of monomolecular film were initiated on days when there was high evaporation potential, when there was delayed application of curing, or when there were equipment breakdowns.

Reports by Pennsylvania Department of Transportation and Kansas State University have also found, during evaluation of concrete protective sealants and curing compounds, that linseed oil proved superior to all other products as a concrete protective material (6, 7).

Field and laboratory tests conducted at Utah State University demonstrated that a monomolecular film served as a suitable evaporation retarder on the surface of the concrete, and it can be applied before finishing. A typical material that will serve as a suitable evaporation retarder is composed of molecules having a long hydrocarbon chain, which is hydrophobic, attached to a hydrophillic alcohol terminal group. The long hydrophobic chain orients itself vertically above the surface of the bleed water (8). If sufficient molecules are present, they form a tightly compressed, effective film. Water molecules may not possess sufficient energy to escape through this long chain film; hence, the evaporation is significantly retarded.

A concrete problem that is affected by surface properties of the concrete involves plastic shrinkage. Plastic shrinkage cracking is usually associated with hot-weather concreting and may develop whenever the rate of evaporation is greater than the rate at which water rises to the surface of the recently placed concrete (bleeding) (4). Although plastic shrinkage cracking is normally associated with hot-weather concreting, experience in Virginia has shown that spring and fall are more critical periods because of the occurrence of higher winds and lower humidities than are common in the summer (5). This evaporation causes the concrete to shrink, thus creating tensile stresses at the drying surface. Liquid-membrane curing compounds are utilized to retard or prevent evaporation of moisture from the concrete. Without the application of these curing compounds, stresses will develop before the concrete has attained adequate strength, and surface cracking may result (1, 6, 7). Plastic shrinkage cracks vary in length from a few inches (2 to 3) to a few feet (3 to 7) and are often almost straight, without any definite pattern (8). With regard to their depth, the term "surface cracks" is misleading; in fact widespread plastic cracking in pavement, extending to a depth of 4 in., has been observed (6). Therefore, unless the cracks are quite shallow and narrow, they can weaken the pavement, permit penetration of moisture, and render the reinforcement vulnerable to corrosion (9).

If the rate of evaporation exceeds the rate at which bleeding water rises to the surface, then plastic shrinkage and plastic shrinkage cracking are likely to occur. This has been shown experimentally (10). However, field investigations have shown that characteristics of the concrete do not have a major influence on plastic shrinkage or plastic shrinkage cracking (6). This has led to the preparation, by the Portland Cement Association (PCA), of a chart indicating the interrelation among air temperature, relative humidity, concrete temperature, wind velocity, and rate of evaporation of surface moisture (11). PCA states that evaporation rates above about 0.2 $lb/ft^2/hour$ may cause plastic shrinkage cracking and that at rates below 0.1 plastic shrinkage cracking will probably not occur (11).

From the foregoing it can be seen that proper curing is very important, especially where high winds, elevated temperatures, and low humidities occur simultaneously. To more definitely assess the interrelations among wind, temperature, humidity, and curing method on concrete pavement surfaces, we conducted a laboratory study.

EXPERIMENTAL PROGRAM

The data obtained for this research were taken from 19 sidewalk-sized test slabs (26 by 24 by 8 in. thick). Variables investigated included three environments (73 F and 25 percent RH, 100 F and 30 percent RH, and 140 F and 25 percent RH), four curing methods [white pigmented curing compound (WPC), monomolecular film followed by white pigmented curing compound (MMF plus WPC), water-soluble linseed oil (LO), and no curing compound (Table 1)], and three wind velocities (0, 8 to 10, and 18 to 20 mph). Constants included mix design (5 sacks per cubic yard of concrete), mixing temperature, mixing procedure, placement, finish, and cure time (Table 2).

For control purposes, flexural strength tests were performed on two 6- by 6- by 36-in. beams from each batch of concrete (ASTM Designation C78-64). The specimens were moist-cured prior to testing. Also, compressive strength tests were made on two 6- by 12-in. cylinders from each batch. The specimens were moist-cured and tested in accordance with ASTM Designation C 39-64. Analysis of the strength results indicates a coefficient of variation of 6 percent for flexural strength and 11 percent for compressive strength.

After 28 days of curing, a minimum of three cores [4 in. (diameter) by 8 in.] were taken from each slab and subjected to diagnostic analyses including the following:

1. Dynamic modulus of elasticity (ASTM Designation C215-60)—the torsional sonic modulus was determined for each case,

2. Bulk density by absolute volume of the top 3 in. and bottom 3 in. of each core (ASTM Designation D1188-68),

3. Abrasion coefficient of the finished surface of the cores (ASTM Designation C418-68), and

4. Splitting tensile strengths on the top 3 in. and bottom 3 in. of each core (ASTM Designation C 496-66).

The steel forms (72 by 27 by 8 in.) for the slabs had two wooden dividers to separate the concrete into three slabs. These three slabs were in turn each subdivided into halves, one-half being covered with a curing compound and the other half without curing compound. In each of these halves, as shown in Figure 1, there was a smaller metal box (6 by 4 by 4 in.) that was inside a wooden box. These smaller metal boxes were designed to be lifted out and weighed periodically on a 10,000-gram balance. Concrete was placed in the forms in three layers, vibrated in place, and finished, and the proper cure was applied to the surface. The wind generator was turned on prior to the placement of the third layer and turned off only during the application of the curing compounds. The weighings of the boxes continued until the water loss became negligible, which was normally after 50 hours. As a method of control, weighings were taken from identical boxes under steady conditions (0 mph). The wind was channeled across the surface of the blocks to prevent the wind from spreading. No significant change in wind velocity could be measured between the first and last block. As a check to ensure that the relative humidity did not vary along the slab lengths, three control sections were monitored along each slab row. Evaporation data from each control section were virtually identical. It should be noted that this test is not designed to duplicate the ASTM water retention test; rather it is an attempt to measure total water loss from the time of placement.

Table 1. Curing methods.

Curing Method	Description
WPC	A white pigmented curing compound, resin-based (ASTM Designation C 309), was sprayed on the test slabs after finishing and after the water sheen had disappeared from the surface (180 ft ² /gal).
MMF plus WPC	A monomolecular film was sprayed on the surface of the test slabs prior to finishing (200 ft ² /gal). After finishing, as soon as the water sheen had disappeared from the surface, the same white pig- mented curing compound as used in the WPC curing method was sprayed on the surface.
LO	A water-soluble linseed oil applied to the surface upon completion of the finishing operation (200 ft ² /gal).
Control (no cure)	The slab surface was allowed to cure without application of any curing compound.

Table 2. Concrete mix designs and strengths.

Batch Code	Percent Absolute Volume						Initial	7-Day Compressive Strength	7-Day Flexural Strength
	Cement	Water	Fine Aggregate [®]	Coarse Aggregate⁵	Air	Slump (in.)	Weight $(1b/ft^3)$	Designation C 39	Designation C 78
36	8.9	13.9	22.5	51.7	3.0	3/4	150	3,760	546
37	8.8	13.9	22.3	51.3	3.7	3/4	144	3,940	674
38	8.8	13.9	22.3	51.3	3.7	3/8	144	4,330	674
39	8.8	13.9	22.3	51.3	3.7	$1^{3}/_{8}$	144	3,120	674
40	8.8	14.0	22.6	51.8	2.8	11/4	148	4,060	645
41	8.8	14.0	22.6	51.8	2.8	1	148	4,220	645
42	8.8	14.0	22.6	51.8	2.8	11/2	148	3,440	645
43	8.8	14.0	22.4	51.5	3.3	1	148	3,840	679
44	8.8	14.0	22.4	51.5	3.3	1	148	3,920	679
45	8.8	14.0	22.4	51.5	3.3	2	148	3,440	679
46	8.8	14.0	22.4	51.6	3.2	7/8	147	3,960	701
47	8.8	14.0	22.4	51.6	3.2	13/4	147	4,140	701
48	8.8	14.0	22.4	51.6	3.2	13/4	147	3,520	701
49°	8.8	13.9	22.4	51.4	3.5	$1^{1}/_{2}$	149	3,280	°
50	8.9	13.9	22.5	51.7	3.0	$2^{3}/_{4}$	148	3,220	663
51	8.9	13.9	22.5	51.7	3.0	11/1	148	3,540	663
52	8.9	13.9	22.5	51.7	3.0	1 ³ /8	148	3.200	663
53	8.9	13.9	22.5	51.7	3.0	3/4	148	3,080	646
54	8.9	13.9	22.5	51.7	3.0	11/4	148	3,900	646
55	8.9	13.9	22.5	51.7	3.0	21/4	148	3,250	646
56°	8.8	14.0	22.6	51.8	2.8	15/8	147	3,010	c

^aSiliceous river-run fine aggregate with a fineness modulus of 2.86. ^bSiliceous river-run coarse aggregate with a fineness modulus of 6.90 (maximum size 1 in.). ^cOnly cylinders made for these batches.

Figure 1. Concrete slab setup.



To determine the effect of moisture loss, measurements of water loss from the surface of test slabs were recorded by the periodic weighing of the boxes during the initial curing period (first 50 hours).

Figures 2 through 7 show the water loss measurements versus log time. The figures are representative of each compound's ability to retain water when exposed to the various temperatures and wind velocities. By comparison, it is clearly shown that the portion of the slab without any curing compound (no cure), exhibits considerably higher water loss than any of those with a curing compound. At the early stages of concrete curing, all curing compounds appear to retard water evaporation at nearly the same rate, but a noticeable difference is seen in the later stages of curing. Further comparison of the six figures shows that it is clearly evident that no significant benefits were obtained by adding MMF, in the particular manner employed in this research, before WPC. Though this statement seems inconsistent with other reports, it should be noted that a single application was used, at the heaviest rate recommended by the manufacturer. Perhaps two or more applications would produce better results, as it is hypothesized that insufficient film was present to form an effective barrier. Consistently, the LO and WPC retarded the rate of evaporation better than MMF plus WPC (except at 73 F and 18 to 20 mph). In fact, at least a 15 percent reduction in water loss was noted with the use of LO and WPC over MMF plus WPC at 73 F and 8 to 10 mph (Fig. 2), and this reduction was 30 percent at 140 F and 8 to 10 mph (Fig. 6). Therefore, on the type of concrete used in this study, having little or no observable bleeding, these laboratory results do not show any reason to use a single application of MMF prior to finishing as an evaporation retarder.

There is another interesting finding relating to the effects of wind. In Figures 2, 4, and 6 the evaporation rates for specimens cured without any compound (no cure) at the specified temperature—but without wind—are shown by dashed lines. Note the effect of wind when compared with no wind in the same environment. Without wind the evaporation during the first several hours is considerably reduced, and with wind none of the curing compounds reduced the evaporation rate to that experienced without cure at 0 mph (Fig. 6). This dramatizes the strong influence of wind on evaporation.

Values were obtained from the PCA chart for the conditions employed in this study. In order to compare the PCA values with the experimental values determined in this research, water loss values for the first few hours were plotted against time, and evaporation rates were determined. Figures 8 and 9 show typical results obtained. PCA values (11) and the experimental values are given in Table 3. As can be seen, the values determined from this research are not quite as high as the values obtained from the PCA chart (except at 140 F). Because the values at 140 F from the PCA chart had to be extrapolated, no real comparison could be made here. Note that the values obtained from the PCA chart for steady conditions (0 mph) are nearly the same as those obtained experimentally in this study. However, significant differences were found at increased wind velocities, which casts doubt on the validity of the PCA chart.

The effects of wind on the curing of the top of the concrete are shown in Figure 10. Top splitting tensile strength is plotted against curing temperature for the four curing methods. These four methods were grouped at each particular wind velocity because no distinguishable difference was statistically established for any of the curing methods. Because review of the strength data indicates that the concretes used were of similar strengths, it is concluded that wind apparently affects the strength of the concrete at 73 F and 25 percent RH. The increased wind (18 to 20 mph) must reduce the available water (evaporation from the surface) so as not to allow the concrete to obtain as high a surface strength (compared to 8- to 10-mph conditions).

Figures 11 (8 to 10 mph) and 12 (18 to 20 mph) show a comparison of abrasion loss and temperature under wind conditions. As can be seen, an increase in abrasion loss is noted with increased wind conditions. Also, no apparent abrasion benefit is obtained with the use of these curing compounds because the no-cure specimens had essentially the same losses as the cured specimens. A comparison of the four methods of cure (MMF plus WPC, WPC, LO, and no cure) indicates that, at 140 F, MMF plus WPC



Figure 2. Effect of curing method on evaporation of water from surface of concrete slabs (73 F and 8 to 10 mph).

Figure 3. Effect of curing method on evaporation of water from surface of concrete slabs (73 F and 18 to 20 mph).





Figure 4. Effect of curing method on evaporation of water from surface of concrete slabs (100 F and 8 to 10 mph).

Figure 5. Effect of curing method on evaporation of water from surface of concrete slabs (100 F and 18 to 20 mph).



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Figure 6. Effect of curing method on evaporation of water from surface of concrete slabs (140 F and 8 to 10 mph).

Figure 7. Effect of curing method on evaporation of water from surface of concrete slabs (140 F and 18 to 20 mph).





Figure 8. Determination of evaporation rate (73 F and 8 to 10 mph).

Figure 9. Determination of evaporation rate (140 F and 8 to 10 mph).



Table 3. Comparison of evaporation rates.

Air	Relative	Wind	0	Evaporation Rate (lb/ft ² /hour)	
(deg F)	(percent)	(mph)	Compound	Experimental	PCA*
73	25	0	None	0.047	0.040
73	25	10	None MMF plus WPC WPC LO	0.158 0.058 0.031 0.044	0.22
73	25	20	None MMF plus WPC WPC LO	0.186 -0.062 0.058 0.029	0.38
100	30	0	None	0.037	0.03
100	30	10	None MMF plus WPC WPC LO	0.136 0.085 0.065 0.045	0.14 - - -
100	30	20	None MMF plus WPC WPC LO	0.182 0.111 0.107 0.057	0.24
140	25	0	None	0.054	0.00
140	25	10	None MMF plus WPC WPC LO	0.206 0.198 0.099 0.085	0.00°
140	25	20	None MMF plus WPC WPC LO	0.235 0.128 0.031 0.048	0.00 ^b

*Concrete temperature of 80 F was used in all predicted values.

^bExtrapolated values.







Figure 11. Effect of curing temperature and curing compounds on abrasion coefficient (0 and 8 to 10 mph).

Figure 12. Effect of curing temperature and curing compounds on abrasion coefficient (0 and 18 to 20 mph).



statistically exhibited a significantly larger abrasion loss. Apparently, the MMF plus WPC did not seal the surface against water loss so well as the other methods at this elevated temperature and wind condition. As indicated by the water loss measurements, there must be sufficient water loss to cause a reduction in surface strength, hence an increase in abrasion loss.

There were no significant differences in the dynamic moduli of all the slabs, and similarly there were no significant differences in bulk densities of the top and bottom of the cores taken.

CONCLUSIONS

The study data support the following conclusions:

1. In all cases adequate strengths were obtained. But, curing temperatures in excess of 100 F resulted in a significant reduction in the strength of the top portion of all concrete slabs, even though adequate curing methods were used. With the simulated wind conditions of 8 to 10 mph and 18 to 20 mph, this reduction in strength was even more pronounced.

2. At temperatures in excess of 100 F, the surfaces cured with the MMF plus WPC showed a high abrasion loss compared to the surfaces cured with either LO or WPC by itself. Thus, there appear to be no surface strength benefits from the one application of the MMF before finishing.

3. Evaporation of water from the surface of the slabs was significantly retarded with the use of any of the curing compounds (MMF plus WPC, LO, and WPC). LO and WPC tended to retard the evaporation more than MMF plus WPC. Thus, the laboratory results do not show any advantage to the particular way in which MMF was used as an evaporation retarder. Conversely, the use of an adequate curing compound was shown to be advantageous.

4. Evaporation rates measured experimentally in this study did not agree with the values predicted by the PCA chart, especially the rates when wind was present. Thus, the validity of a portion of the PCA chart is questioned.

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STATISTICAL ANALYSIS OF CONCRETE STRENGTH VERSUS AIR ENTRAINMENT

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A series of structural-grade concrete test cylinders was cast and tested in the laboratory to determine the decrease in compressive strength to be expected with increasing levels of air entrainment. Regression analysis of these data shows the decrease of strength to be about 400 psi for each 1 percent increase of air entrainment within the range of interest between 3 and 10 percent. Additional techniques are applied to construct a curve to predict probable rejection rates with various air entrainment levels that might be specified in the future. These results, combined with engineering judgment, led to the adoption of a new air specification of 6.0 ± 1.5 percent. Operating characteristic curves are developed to predict probable rejection rates for different lot sizes with the new specification. A logarithmic transformation of the data is used in a second regression analysis to further explore the fundamental relation between air entrainment and strength.

•BECAUSE of a demonstrated scaling problem on concrete bridge structures in New Jersey, a research project was undertaken to test various protective materials and methods. The preliminary test results, supported by a thorough literature review, indicate that the most effective means to prevent scaling continues to be adequate air entrainment. The literature search also produced the rule-of-thumb that increased air entrainment is accompanied by a loss of compressive strength of approximately 5 percent for each additional 1 percent of entrained air. It was desired to determine the maximum level of air entrainment that could be specified without risking too great a loss of compressive strength.

Because certain construction practices are believed to reduce the amount of air entrainment at the surface of the concrete where it is most needed, an increase from the current specification of 4.5 percent up to levels as high as 8 percent were being considered to provide a measure of safety. However, because of the known strength loss with increasing air content, it was suspected that this might result in too great a decrease in strength.

It was decided to cast a series of test cylinders using normal structural concrete with varying amounts of entrained air to determine the loss of strength to be expected. Several laboratory batches of structural grade concrete were prepared with a cement factor of 6.7 sacks per cubic yard and the water-cement ratio controlled as closely as possible at 5.25 gallons per sack. The air content of the fresh concrete was measured by the pressure method and was varied from 3 to 12 percent. This resulted in a total of 67 cylinders being used for the analysis.

Because these batches were carefully controlled and the cylinders were cured in an optimum manner, the resulting strengths represent the potential of field concrete under the best conditions. Because this is rarely the case, the strengths actually obtained in the field would be expected to be somewhat lower.

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The American Concrete Institute (ACI 318-63) has suggested that adequate control of a concrete project is achieved if not more than 20 percent of the strength tests fall below the design strength when working stress design is used. Although this indicates that it would be perfectly normal for up to 20 percent of the cylinders to be below the design strength, for the purposes of this paper these cylinders will be called rejects, and the actual percentage of cylinders below design strength will be referred to as the reject rate. In our case, the design strength is 3,000 psi. The overall reject rate ran about 2 percent with the current air specification of 4.5 percent so there appeared to be considerable latitude to increase the air level.

The recommendation that 20 percent of the total (large) number of tests may be allowed to be rejects is the same as saying that the probability that a single cylinder will be a reject may be as high as 20 percent. (If the overall population of test cylinders has 20 percent rejects, any one cylinder taken at random from this population would have a 20 percent chance of being a reject.) In order to use this recommendation as a guideline, it was necessary to find a method to predict expected rejection rates for the higher levels of air entrainment under consideration.

The method to be used involves the calculation of the regression line for compressive strength versus level of air entrainment. This line is then plotted with its associated confidence bands for a single, future predicted value. Because we are concerned only with strengths that fall below the 3,000-psi minimum, one-tailed student-t values are used in the calculation of these confidence lines, and they are plotted only on the lower side of the regression line as shown in Figure 1.

These confidence lines directly provide the probability of rejection for a single, future laboratory test cylinder at any particular air level. For example, a future cylinder with an air level of 8 percent has a 95 percent chance of being above 3,000 psi because the point (X = 8 percent, Y = 3,000 psi) in Figure 1 is crossed by the 95 percent confidence line. The probability of rejection (being below 3,000 psi) is 5 percent (100 percent minus 95 percent).

Generally speaking, it is not statistically correct to make multiple predictions from these confidence lines because they are intended only for the prediction of a single, future value. However, these lines will be used in a unique manner, one that will permit their application to any number of future values. Briefly, this involves accepting the regression line and its associated confidence lines as a true and correct description of the physical situation. As such, these lines would not change no matter how many additional data points might be obtained. As would be expected, the validity of the conclusions will be dependent on the accuracy with which the regression line has been established.

First, Figure 1 must be plotted in a more useful form to avoid the necessity of interpolating among the various confidence lines. This is done by taking those points that require no interpolation (such as the X = 8 percent, Y = 3,000 psi, and 95 percent confidence line intersection) and using them to construct the curve shown in Figure 2.

All that is needed to predict the rejection rate for a whole future population of cylinders is to know the probability of rejection for a single, future cylinder that falls anywhere within the range included by this population. To obtain this, it must first be assumed that air levels will be distributed normally about the specified target value. (Our own data and those of others tend to confirm this. A typical standard deviation is approximately $\sigma = 0.67$ percent.) This hypothetical normal curve is then imagined to be centered on the level of air to be studied and extends three standard deviations above and below this value as shown in Figure 3. It is then divided into 10 increments of equal area, and a table of the cumulative normal distribution is used to find the zvalue at the centroid of each of these areas. From these, the air level at each centroid is determined, the corresponding rejection rates are read from the Figure 2 curve, and an averaging process is employed to arrive at a reject rate that will apply to a whole population of cylinders spread out over the range of $\pm 3\sigma$.

A simple analogy is helpful in understanding this averaging process. The equal area increments represent boxes, each of which contains an equal quantity of some product. The total production run consists of 10 of these boxes, and each one of the boxes contains a different quantity of rejects. To determine the reject rate for the



Figure 1. Regression analysis of compressive strength versus air content.

Figure 2. Probability of rejection versus air content for a single future laboratory cylinder.



A Design

entire production run, we add up the number of rejects in each box and divide by the total number of items in all 10 boxes.

Table 1 illustrates this procedure for an air level of 6.5 ± 2.0 percent. The 10 equal area increments are given in column 1, the z-values at the centroids of these increments are given in column 2, and the products of the z-values multiplied by the standard deviation are given in column 3. Although the calculation given in this table will be repeated for different levels of air, this is not as laborious as it might seem because these first three columns always remain the same. If this procedure were to be used for some other study, columns 1 and 2 would still remain the same.

The values given in column 4 are obtained by adding the column 3 values to the particular air specification for which the calculation is being made. The column 4 values then represent the actual air levels at the centroids of the 10 increments.

The values in column 5 are obtained from the curve shown in Figure 2. The reject rates are determined at each of the column 4 air levels and then entered in column 5. All that remains is to sum and average column 5 to arrive at an average rejection probability. This rejection probability can be thought of as the expected "percent defective" of the entire future population of laboratory cylinders that would result from this particular air specification.

This procedure is repeated for several different possible specified air levels, and the results are plotted as the right-hand (laboratory) curve shown in Figure 4. It is now necessary to make an approximation to construct a curve in Figure 4 that will apply to field concrete. This is accomplished by plotting the one known field point (the current reject rate of 2 percent with the current air specification of 4.5 ± 1.5 percent) and drawing a curve through this point parallel to the laboratory curve. At a later date, after sufficient data have been obtained with a new specification so that another known field point can be plotted, it will be possible to determine this curve more accurately.

Two minor assumptions should be noted. Although the current air specification requires a tolerance of ± 1.5 percent, it is assumed that the variability actually achieved in the field is ± 2.0 percent ($\pm 3\sigma$) as was used in the calculations for the laboratory cylinders. Also, the laboratory cylinders were cast in groups of four, whereas field concrete cylinders are taken in groups of three. Because the overall variability of field concrete consists primarily of batch-to-batch variation and is influenced to a much lesser degree by sampling, testing, and within-batch variability, this difference would have little effect on the overall variance. Because the effects of these two assumptions are small, they have been ignored.

Remembering that the ACI recommended maximum limit for rejection probability is 20 percent, we shall now use this curve to predict the probable result of different air specifications. For example, a specified level of 8 percent would have an overall expected rejection rate well in excess of the 20 percent limit. Even a 7 percent specification would be slightly above the 20 percent rejection level. At the other extreme, a specification of 5 percent would have an expected rejection rate of about 4 percent, which is unnecessarily conservative.

At this point, engineering judgment had to be exercised. The literature survey had shown that nearly half the states were using an air specification of 6 percent or more for bridge decks. Furthermore, our current mix design was producing consistently high strengths with the mean strength above 4,700 psi. Therefore, it was decided to adopt a specification of 6 percent, for which the predicted rejection rate is 14 percent, somewhat high but well within the limits prescribed by the ACI. This specification is just now being adopted, and the resulting strengths will be watched carefully to determine whether further changes are warranted.

Figure 5 shows the possible consequences of this rejection probability of 14 percent when applied to various sample sizes. Although test cylinders are customarily taken in groups of three, the curves in this graph are for sample sizes of 5, 20, 100, and ∞ to show the general trend. For example, the N = 5 curve indicates that, if a small job were to require only 5 test cylinders, there is about an 85 percent chance that 20 percent or less (one cylinder in this case) will be rejected. Stated the other way, there is a 15 percent chance that two or more cylinders will fail even when the concrete is in Figure 3. Hypothetical normal curve that is centered on the particular air specification under study.



Figure 4. Expected population "percent defective" versus specified air level.



Table 1. Determination of average "percent defective" for air range of 6.5 ± 2.0 percent.

Area Increment (1)	z-Value at Centroid (2)	Product of z-Value Multiplied by Standard Deviation (3)	Air Level at Centroid (6.5 ± 2σ) (4)	Probability of Rejection at Centroid (5)
1	-1.645	-1.10	5.40	0.00040
2	-1.037	-0.69	5.81	0,00075
3	-0.675	-0.45	6.05	0.00110
4	-0.385	-0.26	6.24	0.00150
5	-0.126	-0.08	6.42	0.00190
6	+0.126	+0.08	6.58	0.00275
7	+0.385	+0.26	6.76	0.00425
8	+0.675	+0.45	6.95	0.00725
9	+1.037	+0.69	7,19	0.01200
10	+1.645	+1.10	7.60	0.02400

Note: o from historical data is taken to be 0.67 percent. Average rejection probability = average "percent defective" = 0.05590/10 = 0.0056.



Figure 5. Operating characteristic curves for samples taken from a population with a percent defective of 0.14 (14 percent).

Figure 6. Regression lines compared to 5 percent factor.



perfect control. What may occasionally seem like bad concrete could very well be within specification. The curves shown in Figure 5 do not tell us which is actually the case, but they do provide insight as to what sample reject rates may be expected.

Another interesting outcome of this study is the determination of the rate of decrease of strength as air entrainment is increased. The rule-of-thumb that the strength could be expected to decrease by about 5 percent for each additional 1 percent of air is an exponential expression of the form $y = 0.95^{x}Y_{o}$. In this expression, y stands for the predicted strength, the constant 0.95 pertains to the 5 percent factor, x is the level of air, and Y_{o} is the theoretical strength that would be achieved if there were absolutely no air in the concrete.

Unlike the regression line used thus far in the study, an exponential curve of this type would tend to level off as higher levels of air are approached (dashed lines in Fig. 6). From a fundamental standpoint, it is quite logical that the true relation would be described by such a curve because (a) additional air should have a diminishing effect as more and more air is added and (b) zero strength theoretically should not be reached until 100 percent air is approached.

Because $y = K^Y_{\circ}$ can be expressed as $\log y = x \log K + \log Y_{\circ}$, the regression program was run with a logarithmic transformation of y to fit a curve of this type to the data. The first-order regression constants (slope and intercept) provide the values for log K and log Y_o, from which K and Y_o can be found. This relation is then plotted (Fig. 6). From this, the following three conclusions can be drawn:

1. Throughout the range of interest, between 3 and 10 percent air content, the two regression lines almost coincide. This serves to justify the use of the linear model in this study.

2. The value of K obtained from the computer for the exponential regression line is 0.90. This indicates a decrease of strength of 10 percent for each 1 percent of air, exactly double the rule-of-thumb value.

3. Whichever regression line is used, the decrease in strength predicted by the experiment is substantially greater than that predicted by the rule-of-thumb.

There may be an explanation for the difference noted in the third conclusion. The water-cement ratio is a critical factor that has an inverse effect on the strength of concrete. In the laboratory tests, the water-cement ratio was held as constant as possible at 5.25 gallons per sack. The 5 percent rule-of-thumb may allow for the fact that higher levels of air entrainment provide greater workability of the mix, thereby lowering the water requirement and regaining some of the lost strength. If this is so, then the curves developed for this study will be somewhat conservative and would predict rejection rates somewhat higher than would actually be experienced in the field. Therefore, any error in this particular case can be expected to be on the safe side.

The particular combination of statistical techniques employed in this paper was developed in an attempt to make the best possible choice for a new air-entrainment specification. To the best of the author's knowledge, this is an original approach that provides a valid method for estimating a large quantity of future values when the regression line is well established. With a computerized regression program and the appropriate statistical tables, these steps can be performed quite rapidly and could readily be applied to other studies concerning the relation between two variables.

APPLICABILITY OF CONTROL CHARTS TO CONCRETE PRODUCTION AND INSPECTION

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The concepts of process control and their applicability to concrete production are discussed. The commonly used types of process control charts are reviewed, and concurrent use of R- and \overline{X} -charts is recommended. Particular attention is given to selection of producer's risk, rational subgroup, and subgroup size and to correct procedures for determining the variation needed to set control limits. In addition, operating characteristic curves are presented to show how the number of rational subgroups used in setting up these charts affects the probability of falsely declaring a process in control. Other operating characteristic curves are presented to show the effect of rational subgroup size on the ability of R- and \overline{X} -charts to detect respectively changes in process variation and slippages in process means. Guidelines for initial use are offered for producers who wish to institute formalized process control in concrete plants but who lack the necessary data. The paper also discusses the applicability of acceptance control charts to concrete inspection. Their concepts and assumptions are considered in light of concrete properties, and it is concluded that they are not appropriate for concrete inspection. Specifically, it is shown that assumptions necessary for proper use of acceptance control charts are not satisfied by concrete properties and that, even if they were, there would be no advantage in using them instead of equivalent acceptance sampling plans because the amounts of sampling and testing required would be the same.

•DURING the last decade, highway engineers have sought better ways to ensure that construction materials comply with specification requirements. In the process, some confusion has resulted. Some have attempted to use process control charts instead of acceptance sampling plans, perhaps in the mistaken belief that there is no difference between acceptance control charts and process control charts and that either type can be substituted for acceptance sampling plans. In fact, this is not the case.

As Juran and Gryna (1) see it, "process control or 'correction' refers to the sequence of events by which a process is kept free of sporadic troubles, i.e., the means by which the status quo is maintained." This is in contrast to acceptance sampling, which is defined by the American Society for Quality Control (in ASQC Standard A2-1970) as the "sampling inspection in which decisions are made to accept or reject a product." Similarly, acceptance control charts are used to judge the acceptability of a process and are similar to acceptance sampling plans. If specific conditions are met, acceptance control charts can be used instead of acceptance sampling plans, but they can never replace process control charts. The function of process control charts is to keep a process in a state of statistical control, whereas the function of acceptance sampling plans and acceptance control charts is to judge the acceptability of either a product or a process, and no pretense is made that the process judged is in a state of statistical control.

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The differences between acceptance sampling and process control are well known. The necessary conditions for proper use of acceptance control charts as substitutes for acceptance sampling plans are also well understood. But unfortunately, the literature dealing with these subjects seldom reaches practicing highway engineers, and therefore valuable information goes unused. Consequently, engineers responsible for ensuring quality concrete hold basic misconceptions, which although not specifically stated are implied in the literature available on attempts made to ensure concrete quality. These misconceptions can be summarized as follows:

1. Process control charts can be substituted for acceptance sampling plans or acceptance control charts and sampling can thus be reduced,

2. Acceptance control charts are interchangeable with process control charts, and 3. Buyers can successfully perform process control.

A review of basic process control concepts and the conditions necessary for proper use of acceptance control charts is usually enough to show that process control has nothing in common with inspection or acceptance control charts and that process control can be successfully performed only by producers. It also shows that the amount of sampling for process control could exceed that necessary for inspection and that, in practice, process control is a difficult task.

Thus, in hope of dispelling some of these misconceptions, the major objective of this paper is briefly to review, in a form easily available to highway engineers, the basic concepts and types of control charts for their applicability to concrete production, in order to prepare for the eventual, almost certain, introduction of formalized process control by the concrete industry. A secondary objective is to suggest guidelines for initial use to those producers who wish to initiate process control but who lack the appropriate data because public agencies and other buyers have assumed responsibility for testing all output from their plants.

No new theory is presented—rather, well-established concepts are reviewed to see how they can be applied to concrete. In this respect, the references given are important and should be consulted wherever the paper dwells only briefly on any of the topics discussed.

PROCESS CONTROL CHARTS

Concrete is a manufactured product. As such, its production must be systematically controlled if compliance with specification requirements is not to be left to chance. As for any other product, acceptance sampling will reject most or all concrete produced if its production cannot be controlled to attain the desired properties and property levels. Thus, prospective sellers must ensure that their concrete can consistently and economically satisfy market requirements.

Ensuring that concrete can be economically manufactured involves capability studies; ensuring that it constantly meets buyer demands requires process control. Both capability studies and process control involve physical manipulation of machines and materials, and for that reason they are the responsibility of the manufacturer. The buyer can observe these functions and use the resulting information, but seldom can he perform either for two reasons. First, the buyer and his inspectors may be removed from the manufacturing process. Second, inspectors rarely have the many skills necessary for effective process control, which involves such tasks as sharpening tools, calibrating measuring devices, blending materials, and operating machinery as well as a knowledge of applied statistics. Most inspectors are not that knowledgeable, and, even if men with the necessary skills could be found, producers might prevent their interfering with production. This, however, does not mean buyers should ignore process control. Buyers should encourage it and understand it well to analyze process control data and make informed decisions. Process control data are usually available in the form of control charts, and buyers or their representatives must be familiar with the types of control charts and concepts involved to take advantage of the information they provide.

Concepts and Nomenclature

In simplest terms, control charts are graphical tests of hypotheses. These charts are based on the idea that in a manufacturing process variations are inevitable but can be minimized by eliminating causes of large variations. According to control chart theory, total variation in process output is composed of two parts: variation due to assignable causes and random or inherent variation. The former is the variation that can be identified and eliminated by removing the assignable causes. The latter is that variation that cannot be attributed to any single factor and cannot be economically eliminated. When all variation due to assignable causes is removed and only random variation remains, the process is said to be in control. Control charts are used to test graphically the hypothesis that differences in properties of the process output are due only to random variation, i.e., that the process is in control.

When the process is in statistical control, variation is at a minimum, and computed statistics of the output properties assume predictable patterns, which in most cases can be characterized closely by known frequency distributions. Control charts make use of these facts in a simple, systematic way. To set up a control chart, variation due to assignable causes is eliminated, the magnitude of the random variation is computed, and the frequency distributions of the properties of the process output are determined. Then, the sample size and statistic to be used in testing the hypothesis of control are chosen along with a confidence interval. Once these parameters are known, the critical values for the hypothesis that only chance variation exists can be computed and plotted. The result is a control chart in which the limiting lines correspond to critical values that the controlled statistics cannot exceed in order not to reject the hypothesis of control.

Control charts can be used for different purposes and based on a number of statistics. Control charts are commonly used for process control, for process acceptance, and for analysis of past data. These uses give rise to the nomenclature of process control charts, acceptance control charts, and control charts to analyze past data. Besides taking their names from their intended function, control charts are also named for the statistics used. The process control charts most commonly used, which take their names from the statistics used, are the following:

- 1. Control charts for fraction defective (p-charts);
- 2. Control charts for number of defects (c-charts);
- 3. Difference control charts;
- 4. Cumulative sum control charts ("cusum" charts);
- 5. Standard deviation control charts (σ -charts);
- 6. Control charts for sample ranges (R-charts); and
- 7. Control charts for sample means $(\overline{\mathbf{X}}$ -charts).

Charts Applicable to Concrete

The choice of the statistic to be used in a control chart depends on the nature of the product and process to be controlled, nature and ease of testing, reproducibility of test methods, and expertise of the control chart users. Thus, to decide which statistic is most appropriate, the advantages and shortcomings of each must be viewed in light of the product's properties eligible for control.

In the case of concrete, producers could choose to control the same properties that concrete buyers measure for acceptance sampling: slump, air content, and cylinder strength. But their choice is not and should not be limited to these properties. Among other variables eligible for control are the amounts and quality of ingredients used in making concrete. The choice depends on the producer and on his knowledge of the relations between chosen control variables and desired properties of the final product. For concrete, it is possible to control slump and strength by controlling the water-cement ratio and air content by controlling the amount of air-entraining agent. Thus, the properties likely to be chosen for control can all be measured on a continuous scale, and statistics such as mean, standard deviation, and range, as well as fraction defective, can be computed for each property. This means that X-charts, σ -charts,

R-charts and p-charts as well as others are all theoretically applicable to concrete production. Although applicable, all of these charts do not offer the same advantages.

<u>p-Charts</u>—Charts to control the fraction defective (p-charts) are desirable from a management point of view. They provide a continuous record of quality for economic studies and management decisions. However, they are not very helpful to the quality control engineer because a production unit can be out-of-specification for more than one property and because, by controlling the total fraction defective, he does not know which property is causing defects or in what proportion. Thus, information essential to prevent defects is not readily available. Moreover, p-charts require large sample sizes, and, unless testing is relatively inexpensive and nondestructive, they are economically undesirable. Because concrete testing is time-consuming and expensive and because concrete can be defective for more than one property, p-charts are not the most appropriate for control of its production.

<u>c-Charts</u>—Number-of-defects-per-unit control charts are used when one single production unit can have a large number of defects that are not necessarily detrimental to performance of the production unit but are nevertheless undesirable, for example, numbers of blemishes per square yard of cloth, scratches on a refrigerator, or minor defects in a car. This type of control chart requires that the testing be by attributes. It is not the most appropriate to control such variables as are encountered in concrete production.

<u>Difference Control Charts</u>—Difference control charts are used to test the hypothesis that a process output is no different than material in a standard lot kept under the same environmental conditions as the process output being judged. They are employed when test results are sensitive to such conditions. The process is said to be in control if the control statistic of an output sample does not differ from the corresponding statistic computed from a sample taken from the standard lot by more than the difference expected due to sampling variations. For concrete, no standard lot can be kept because it hardens, and thus this type of control chart is not appropriate.

<u>Cusum Charts</u>—Cumulative sum control charts can be used for control of both the process average and fraction defective. These charts are statistically more discriminating than the corresponding Shewhart charts, or \overline{X} - and p-charts. But their limits depend on the average run length (ARL) and are difficult to compute without the aid of a computer or such monographs and tables as those given by Kemp (2, 3). Although these charts, in principle, are applicable to concrete production, it is believed that they will not be well received by the concrete industry. What is gained in statistical efficiency with cusum charts does not compensate for the simplicity and clear graphical display of the process operation lost by not using the corresponding classical Shewhart charts.

<u> σ -Charts</u>—Standard deviation control charts (σ -charts) are adaptable for control of the variability of output properties that can be measured on a continuous scale, such as those of concrete, and thus they could be used. However, they require large sample sizes. If the sample is less than 10, the range is preferable as a measure of variability. Because in concrete testing it is very difficult to sample 10 or more consecutive production units, σ -charts are not the most appropriate. The range control chart is more effective because it allows judging variability at more frequent intervals.

<u>**R-Charts-Control charts for sample ranges (R-charts) are widely used to control the variability of process output.</u> They are applicable to concrete properties and are considered the most appropriate for control of concrete variability.</u>**

<u>X-Charts</u>—Perhaps the most widely used and misused of all control charts are those for sample means. These are simple to construct and provide self-explanatory displays of process conditions with time. Because of their simplicity and because the theory for these charts is widely published, \overline{X} -charts are considered the most appropriate to control the levels of concrete properties.

R- and X-charts then emerge as the most logical choices for control of concrete production. Next, it must be determined whether they should be used separately or concurrently. The ultimate goal of process control is the elimination of defective production units. In the case of concrete properties, defectives can be caused both by shifts in means and by increases in variability. Thus, for effective control of the process, R- and $\overline{\mathbf{X}}$ -charts must be used concurrently. To illustrate this point, consider air content. If mean air content coincides with that desired, but its variability as measured by the standard deviation is larger than allowed, some concrete batches will have air contents outside the desired limits. This is shown in Figure 1a, where σ_0 is the desired standard deviation and $\overline{\mathbf{X}}_0$ is the desired mean. If the process mean and standard deviation coincide with those desired, no results will exceed the tolerances. However, if $\overline{\mathbf{X}}_0$ approaches the desired but σ_0 increases to σ_1 , some results will exceed the limits as represented by the shaded areas. Similarly, if the mean of the process shifts to $\overline{\mathbf{X}}_0 \pm \delta$ while the process standard deviation remains approximately equal to that desired, results will again fall outside the limits as shown in Figure 1b. If the mean increases to $\overline{\mathbf{X}}_0 + \delta$, some results will exceed the upper limit as represented by the area A_2 . If the mean shifts to $\overline{\mathbf{X}}_0 - \delta$, a fraction of the results represented by area A_1 will exceed the lower limit. Thus, to ensure that the process output meets specification tolerances, both the process mean and the variability must be controlled, and R- and $\overline{\mathbf{X}}$ -charts must be used concurrently.

Information Required for R- and \overline{X} -Charts

Choosing the types of control charts to be used is only the first step. Next comes the more difficult task of gathering the necessary information. Constructing R- and \overline{X} -charts requires knowing the frequency distribution of sample means and sample ranges, the frequency distribution of the control properties, the desired process mean, the standard deviation of the control properties when the process is operating at the level of control desired, the probability of falsely looking for trouble in the process when none exists, and the size of the rational subgroup to be used.

The frequency distributions of sample means and sample ranges are well known. Their parameters are extensively tabulated in the quality control literature and present no problem. The literature also indicates that the frequency distributions of concrete properties are approximately normal (4). The desired process mean is usually set in the specifications and needs no attention at this stage, but the remaining parameters are not so readily obtainable. The selections of subgroup size and the probability of falsely looking for process trouble depend on costs, whereas standard deviations to be used must either be determined from given standards or obtained through process capability studies.

The choice of producer's risk (the probability of falsely looking for assignable causes when none may exist) depends on the economic consequences of not discovering assignable causes in those instances where they do exist. To stop the process and look for trouble adds to production costs, but so does rejection of production units. In choosing the probability of falsely looking for trouble, the cost of looking for assignable causes and discovering none must be balanced against that of rejection resulting from assignable causes going undetected. If looking for assignable causes is inexpensive, whereas the cost of rejection is high, the probability of falsely looking for trouble should be relatively large, say, 5 to 10 percent. However, if the cost of looking for trouble in the process is high whereas the cost of rejections is low, this probability should be chosen to be low.

For concrete, the cost of rejections can be very high; for example, rejection of a $6-yd^3$ load of concrete means a loss of at least \$90. A few rejections can quickly dissipate a day's profit. But chasing nonexistent assignable causes on 5 percent of the occasions when a sample is recovered from the process can be more expensive yet, especially if work must stop. This suggests that initially setting the probability of falsely looking for trouble at approximately 1 percent and using the customary 3σ limits is still appropriate. This risk could then be changed, based on actual cost data.

The choice of the rational subgroup must be consistent with the objective of control charting and must be based on both economic and process considerations. The objective of the range chart is continuous testing of the hypothesis that process variation does not differ from its variation when in control by more than expected due to sampling variation alone. For proper testing of this hypothesis, R-chart limits must be based on random variation alone, i.e., on the variation representing control. If other than random variation were included, the resulting limits would be wider, with loss of sensitivity of the R-chart to changes in process variation (5, ch. 13, p. 42). Similarly, the intent of an \overline{X} -chart is to detect shifts in process average greater in magnitude than those expected due to random variation. This is accomplished by continuously testing the hypothesis that the process mean at any time does not differ from that of the process when in control by more than expected due to random variation. The limiting values of the expected shifts, which are the \overline{X} -chart limits, depend on the random variation. Again, if these limits were computed based on a standard deviation including other than random variation, they would be wider and would result in loss in sensitivity of the \overline{X} -chart to shifts in process average.

Thus, for control chart purposes, it will be necessary to obtain a sample reflecting only random variation. Such a sample is also known as the rational subgroup, and most quality control books offer guidelines for its proper selection. These guidelines can be succinctly summarized: Include in the subgroup only consecutive production units manufactured with the same materials and under essentially the same conditions. It is reasoned that such a sample is most likely to reflect random variation alone because the process mean and variation are likely to remain stable over short periods.

Unfortunately, this golden rule cannot always be easily applied because recovery of samples from consecutive production units can be physically difficult or even impossible. For concrete, testing consecutive production units is difficult because it takes about 20 minutes to sample and test one production unit. During that time, a plant in full production can mix at least 10 batches, and concrete testing cannot be postponed. Thus, sampling consecutive units is a remote possibility unless more than one tester is provided, or variables other than those inspected for acceptance sampling are used for control—probably an unlikely case. However, if production of nonconsecutive units occurred under essentially the same conditions and within a relatively short time, variation among them might approach that of consecutive units. Under these circumstances, a sample approaching the rational subgroup would still be obtainable with one tester. Thus, when sampling for process control, care should be taken to ensure that sampling is performed as quickly as possible, and during sampling that aggregates, cement, admixtures, and personnel remain unchanged.

A subgroup consisting of consecutive or nearly consecutive production units is also desirable from a practical point of view and preferable to a random sample. The practical importance is that such a sample facilitates the identification of assignable causes. For example, assume that a random sample of size n is recovered over 2 hours and has to be used for control chart purposes. Further assume that the mean computed from this sample shows a lack of control when plotted on the \overline{X} -chart, that individual test results are not available, and that the problem is to identify what caused the process mean to shift.

Under these circumstances, it is not known at what point during production the process first came under the influence of assignable causes. Thus, all factors present during the 2-hour period, but not before, must be suspected and investigated. In concrete production many things can change in 2 hours, and the list of suspects may be large, making it difficult to isolate the culprit if it has not disappeared meanwhile. But if the sample consisted of consecutive or nearly consecutive production units, the search would be limited to factors present during a very short time. The list of suspects would be smaller, and chances of the culprit disappearing would be minimized. Thus, random sampling that is essential for acceptance sampling is undesirable in sampling for process control.

Another factor influencing the control limits is sample size. The larger the sample size is, the tighter the limits and greater the sensitivity of the charts. Choice of subgroup size depends chiefly on economics. In choosing sample size, testing costs must be balanced against the consequences of producing defectives. Thus, optimum sample size and sampling frequency should be determined for each separate process. However, findings of theoretical studies can serve as a guide until enough information is accumulated from case studies. A. J. Duncan made one such study of sample sizes for \overline{X} - and R-charts (6, p. 398), summarizing his findings as follows: "1. The customary sample sizes of 4 or 5 are close to optimum if the shifts to be detected are relatively large, e.g., if the assignable cause produces a shift of 2σ ' or more in the process average. If it is the aim of the chart to detect shifts in the process average as small as 1σ ', sample sizes of 15 to 20 are more economical than sample sizes of 4 or 5.

"2. If a shift in the process average causes a high rate of loss, i.e., high relative to the cost of inspection, it is better to take small samples quite frequently than large samples less frequently. For example, when the rate of loss is high, samples of 4 or 5 taken every half hour are better than samples of 8 or 10 taken every hour.

"3. Under certain circumstances charts using 2σ or even 1.5σ limits are more economical than charts using the conventional 3σ limits. This is true if it is possible to decide very quickly and inexpensively that nothing is wrong with the process when a point (just by chance) happens to fall outside the control limits, i.e., when the cost of looking for trouble when none exists is low. Contrariwise, it will be more economical to use charts with 3.5σ to 4σ limits if the cost of looking for trouble is very high.

"4. If the unit cost of inspection is relatively high, the most economical design is one that takes small samples (say samples of 2) at relatively long intervals (say every 4 to 8 hours) with narrow control limits, say at $\pm 1.5\sigma$."

If the concrete properties conventionally tested were chosen for process control, which is likely to be the case, it is suspected that assignable causes would produce large shifts in process average. But this is only a conjecture, which cannot be substantiated with available data. The bulk of the data available to the author for the traditionally measured concrete properties were obtained under random sampling. They thus reflect random as well as assignable cause variation and preclude determining the magnitude of changes in process averages due to assignable causes. If for process control producers chose other than the conventionally tested properties, the magnitude of shifts in mean due to assignable causes would still be unknown because data on other than conventionally tested properties are almost nonexistent. This precludes choosing sample size on the basis of the expected magnitude of shifts in process mean, and the choice must be made on the basis of cost of rejection, which can be high. Thus, it appears desirable to test small subgroups at frequent intervals, and samples of four taken at least every hour are a good starting point in accumulating the data necessary for determining optimum sample size.

Regardless of the sample size ultimately chosen, its effect on sensitivity of $\overline{\mathbf{X}}$ - and R-charts can be shown with operating characteristic curves (6, p. 391). The OCcurves shown in Figure 2a show how sample size affects the $\overline{\mathbf{X}}$ -chart's ability to detect a shift in mean of a given magnitude. The OC-curves shown in Figure 2b show how sample size affects the R-chart's ability to detect changes in process variation. It can be seen that the probability of not catching a shift in process average of the same magnitude increases as sample size decreases. For example, the probability of not detecting a shift in mean of magnitude $2.0\sigma'$, on taking the first sample after the shift has occurred, varies approximately from 0.04 for n = 6 to 0.55 for n = 2. Similarly, Figure 3 shows that the probability of not detecting a change in process standard deviation when it doubles ($\lambda = 2$) varies from 0.52 for n = 6 to 0.82 for n = 2. The OCcurves also show that the probability of not catching large changes in process average and variation, upon taking the first sample after the changes have occurred, is relatively high with small subgroups. But the probability of not catching changes in process average and variation on the first and/or second sample after the changes have occurred is the product of the probability of not detecting the change for each individual sample, and generally the probability of not catching a change with any of g consecutive subgroups is $P^{\mathfrak{g}}$, where P is the probability of not catching the change with a single subgroup. Because P is a fraction, the probability of not detecting a change in either process average or process variation in any of g subgroups quickly becomes small even for moderate values of g. For this reason, sampling for process control should occur at frequent intervals.

Having chosen the producer's risk and rational subgroup size, the value of the standard deviation to be used must still be determined before R- and \overline{X} -charts can be Figure 1. Effects on fraction defective of changes in standard deviation and shifts in mean.

Figure 2. OC-curves for process control charts for 3σ limits.





Figure 3. OC-curves for control charts of sample mean.



 $[\]theta_j$ = size of shift in mean from the desired process mean in terms of the process standard deviation.

Figure 4. Example of process control chart when standards are not given.



(b) R-CHART



constructed. This variation may be known from past experience, derived from given standards, determined through process capability studies, or approximated from recent process output. In setting up R- and $\overline{\mathbf{X}}$ -charts, two situations can arise. In the first, with no standards given, the minimum achievable process variation is unknown, and the process must be brought into control with respect to itself. In the second, standards are given and the process must be controlled to meet the standards but need not necessarily be brought into control with respect to itself. When no standards are given, the process must be manipulated until all assignable cause variations are eliminated and the properties controlled assume predictable patterns. For correct determination of whether a process has reached a state of control, the decision must be based on test results from a large number of rational subgroups. But because testing is usually costly, it is customary to accept the hypothesis that a process has reached control solely on the basis of limited data from the recent past and to estimate the standard deviation to be used in setting control limits from these same data. This procedure, which involves some risks, consists of the following steps (discussed further in 5, ch. 13, pp. 46-63):

1. Test a predetermined number of subgroups of size n;

2. Compute the mean and range for each subgroup;

3. Using appropriate formulas based on the data collected and the subgroup size, calculate upper and lower limits for both the R- and \overline{X} -charts;

4. Plot the subgroup ranges and means respectively on R- and $\overline{X}\text{-charts};$ and

5. If all plotted points fall within the control limits, accept the hypothesis of control; otherwise reject it.

However, a process thus declared in control may in fact not be. The probability of the process not having reached a state of statistical control depends on the number of rational subgroups and the subgroup size. King (7, 8) has computed these probabilities for a number of subgroups of size five as a function of shifts in process average. The resulting OC-curves and OC-curve upper bounds are shown in Figure 3, in which a process whose mean has shifted from the true but unknown process mean by $1.0\sigma'$ would be accepted as being in control with respect to itself only 3 percent of the time, if the X-chart were based on 25 subgroups of 5. However, if the mean had shifted the same amount, the process would be accepted as in control approximately 90 percent of the time, if the mean control chart were based on two subgroups of five. Thus, of the OC-curves shown, that based on 25 subgroups of 5 gives the lowest probability of declaring a process in control with respect to itself when in fact it is not. This is one reason why most books on quality control recommend basing the X- and R-charts on at least 25 subgroups of 5. Another reason is that such control charts have been observed to work well in practice.

In some cases, particularly as producers accumulate data, the standard deviation is known, and control limits can be easily determined. More frequently, standards are given, and the standard deviation to be used for process control is derived from them. This is usually the case when the sole objective of process control is to meet the buyer's (or inspecting agency's) requirements, and producers choose the same properties for control that will be tested for acceptance sampling. In those situations, the standard deviation to be used in construction of \overline{X} - and R-charts is taken as onesixth the specification range for each control property. This approach assumes that the process is capable of producing outputs whose properties have standard deviations equal to or less than one-sixth the respective specification ranges.

For concrete properties, standards are given in the form of specifications, and bringing the process into control with respect to itself could appear to be superfluous work. It is desirable, however, to ensure that the process is capable of meeting the specifications. When a process is brought into control with respect to itself, the process mean may differ from that specified, but process variation is close to a minimum and represents that economically achievable. Thus, bringing the process into control with respect to itself will reveal whether it can meet specifications; if it cannot, defectives will result. Undertaking production under these circumstances is very risky if the buyer is using statistical acceptance sampling plans (as defined in ASQC Standard A2-1970) for inspection. Consequently, producers should bring the process into control with respect to itself whenever possible. However, because concrete testing is costly and difficult, it is suggested that initially producers use a minimum of five subgroups of five to establish control limits instead of customary 25 subgroups of 5. This gives a probability of falsely declaring the process in control of approximately 50 percent (Fig. 3) when the mean shifts by 1.0σ . But producers could revise the limits and reduce this probability by using results from subgroups tested subsequently to monitor the process and falling within the control limits.

Constructing R- and \overline{X} -Charts

Once necessary decisions have been made and essential parameters chosen, construction of R- and \overline{X} -charts is reduced to a simple step-by-step procedure. When no standard is given, R- and \overline{X} -charts must be based on observed data obtained from the process during the immediate or recent past. The necessary steps are as follows:

1. Take g rational subgroups of size n from the process, as close in time as possible, and compute the average range \overline{R} as follows:

$$\overline{\mathbf{R}} = \frac{\sum_{i=1}^{g} \mathbf{R}_{i}}{\frac{\mathbf{g}}{\mathbf{g}}}$$

where

 R_i = the range of each subgroup of size n, and

g = number of subgroups of size n.

Then the upper control limit UCL and lower control limit LCL are computed as follows and the R-chart plotted as shown in Figure 4b:

$$UCL = \overline{R} + k \frac{d_3}{d_2} \overline{R}$$
$$LCL = \overline{R} - k \frac{d_3}{d_2} \overline{R}$$

where

- k = the number of range standard deviations corresponding to one minus the probability of falsely looking for trouble (usually taken as two or three),
- d_2 = the mean of the distribution of relative range R/σ' for sample of size n, and
- d_3 = the standard deviation for the distribution of the relative range for sample size given (values of d_2 and d_3 are tabulated in most quality control books, but sometimes under different names, 6, p. 908).

2. To set up the \overline{X} -chart from the g subgroups, compute the grand mean \overline{X} as follows:

$$\overline{\overline{X}} = \frac{\sum_{i=1}^{ng} X_i}{ng}$$

where

 X_1 = the individual test results,

g = the number of rational subgroups, and

n =the size of each rational subgroup.

Then compute the UCL and LCL as follows, and plot the X-chart as shown in Figure 4a:



Figure 5. Example of process control chart when standards are given.

Figure 6. Control limits for process and control charts related to specification limits.

FRACTION DEFECTIVE

Figure 7. Magnitude of shifts in mean necessary to produce defectives.



$$UCL = \overline{\overline{X}} + k \frac{\overline{R}}{d_2 \sqrt{n}}$$
$$LCL = \overline{\overline{X}} - k \frac{\overline{R}}{d_2 \sqrt{n}}$$

When the mean and standard deviation of control properties are known or derived from specification limits, the procedure for setting up R- and \overline{X} -charts for subgroups of size n becomes very simple. Letting σ' equal the known or derived standard deviation, the procedure for constructing R-charts reduces to the following steps:

1. Compute \overline{R} by $\overline{R} = d_2\sigma'$, where $\overline{R} =$ the average range, $d_2 =$ the mean of the relative range, and $\sigma' =$ the known or assumed process standard deviation.

2. Compute the R-chart UCL and LCL as follows-UCL = $d_2\sigma' + kd_3\sigma'$ and LCL = $d_2\sigma' - kd_3\sigma'$.

3. Plot the R-chart as shown in Figure 5b.

Similarly, letting the given or specification mean equal $\overline{\mathbf{X}}$, the procedure in setting up an $\overline{\mathbf{X}}$ -chart for standard given reduces to the following steps:

1. Compute the UCL and LCL as follows-

UCL =
$$\overline{\overline{X}} + k \frac{\sigma'}{\sqrt{n}}$$

LCL = $\overline{\overline{X}} - k \frac{\sigma'}{\sqrt{n}}$

2. Plot the $\overline{\mathbf{X}}$ -chart as shown in Figure 5a.

Both the procedure for no standard given and the procedure for standard given assume that the control properties are normally distributed. This assumption is usually of no great consequence unless the properties controlled follow distributions deviating markedly from normality. As already mentioned, concrete properties can be taken to be approximately normally distributed, and this assumption should not result in difficulties.

Operation of R- and $\overline{\mathbf{X}}$ -Charts

Consistent with the objective of using control charts graphically to test the hypothesis that the control statistic does not fall outside the allowed intervals, the operation of R- and \overline{X} -charts reduces to three steps:

1. Sampling and testing rational subgroups,

2. Computing subgroup means and ranges, and

3. Plotting subgroup means and ranges on the appropriate control charts to see whether they fall within the chosen confidence intervals (which are represented graphically by the control limits).

If the values of subgroup ranges and means fall within the corresponding control chart limits, the process is considered in control, and routine testing is continued. If either the mean or range of one or more subgroups plots outside the control limits, the process is taken to be out of control. When a point on either plot is out of control, assignable causes are sought, and if found they are identified and eliminated. During the search for assignable causes, testing frequencies are increased, and testing continues until there is reason to believe that the process is back in control and likely to remain there. When there is evidence that control has been restored, routine testing is resumed until another point again plots out of control, and the cycle then starts again.

Experienced quality control personnel have refined the basic rules for operation of control charts in attempts to prevent the process from going out of control at all. They have established criteria for action before assignable causes can cause trouble. For

example, 2σ limits are often used as a warning limit for action when too many points approach or exceed those limits. Other criteria are also used. Duncan summarizes the common action criteria as follows (6, p. 347):

"1. One or more points outside the control limits.

"2. One or more points in the vicinity of a warning limit. This suggests the need for immediately taking more data to check on the possibility of the process being out of control.

"3. A run [defined as successive items of the same class] of 7 or more points. This might be a run up or run down or simply a run above or below the central line on the control chart.

"4. Cycles or other nonrandom patterns in the data. Such patterns may be of great help to the experienced operator. Other criteria that are sometimes used are the following:

"5. A run of 2 or 3 points outside of 2σ limits.

"6. A run of 4 or 5 points outside of 1σ limits."

This multiplicity of criteria increases the chances of falsely looking for trouble, and the choice among action criteria should be based on economics.

R- and \overline{X} -Charts Suggested for Initiation of Formalized Process Control in Concrete Plants

The properties to be controlled, size of the rational subgroup, and probability of falsely looking for trouble are the responsibilities and choices of producers. They should also choose whether to control each process with respect to given standards or with respect to itself. To make these decisions, producers must rely on data systematically collected and properly analyzed. To the writer's knowledge, however, few concrete producers practice formalized statistical process control, as defined in ASQC Standard A3. For this reason, it seems appropriate for public agencies to suggest process control charts for producers to use until they can accumulate enough information to set up quality control systems properly on an individual plant basis. Such suggestions follow, based on the points discussed, which it is believed may provide a good starting point and yield good results:

1. Bring the process into control with respect to itself to ensure that specifications can be met;

2. When the process is in control with respect to itself and variation is consistent with the specification ranges, set up R- and \overline{X} -charts based on the standard deviation derived from the specifications, i.e., one-sixth the range for the property inspected;

3. Use a probability of falsely looking for assignable causes of approximately 1 percent by using $3\sigma \lim its$ control charts (k = 3); and

4. Use both R- and $\overline{\mathbf{X}}$ -charts to minimize rejections and a rational subgroup of size four.

Were these suggestions taken, most concrete plants could be controlled to meet specifications. In New York, control charts based on these guidelines were used in three case studies with good results (4). There is every reason to believe that results would be similar using these guidelines for other plants.

ACCEPTANCE CONTROL CHARTS

In some instances, the specification range is much greater than six process standard deviations, and the process output can meet the specification limits even when the process mean has shifted out of control. When this occurs, there is little chance of producing defectives, and it may be desirable to use the kinds of \overline{X} -charts known as acceptance control charts.

Their construction requires that the process standard deviation be known. Their control limits do not coincide with those of \overline{X} -charts for process control based on the same sample sizes and standard deviations. Unlike process control charts, acceptance control charts are not designed to detect lack of process control. Their only goal is to ensure with known risks that the percentage of defective output is limited

to preestablished levels. In this respect, acceptance control charts resemble acceptance sampling plans with standard deviation known. In fact, double-limit acceptance control charts and double-limit sampling plans by variables with standard deviation known both require approximately the same sample size to ensure the same quality levels with the same risks. But, although statistically identical to variables sampling plans with known standard deviation, acceptance control charts differ in concept. Acceptance control charts accept or reject a process, whereas acceptance sampling plans accept or reject lots. The course of action required to implement the decisions of acceptance control charts is to do something about the process. The action required to implement the decisions made with acceptance sampling plans is to reject or accept individual lots, which are limited amounts of production.

Another basic difference between acceptance control charts and acceptance sampling plans lies in the sampling. Acceptance sampling requires random sampling, whereas for acceptance control charts it is desirable to take the necessary sample all at once. This is because, if the sample is recovered over a particular period, a change in the process that has taken place during that time may be covered up by the averaging of sample results (6, p. 435).

Limits for Acceptance Control Charts

The limits for acceptance control charts depend on the following quantities: acceptable process level (APL), rejectable process level (RPL), producer's risk (α), consumer's risk (β), subgroup size (n), and process standard deviation (σ '). These quantities have meanings analogous to the parameters necessary to design sampling plans by variables. Specifically, APL is the process fraction defective that can be accepted with no adverse consequences. RPL is the process fraction defective that can barely be tolerated. Producer's risk is the probability of rejecting a process that is producing a fraction defective equal to the APL. Consumer's risk is the probability of accepting a process producing a fraction defective equal to the RPL. Subgroup size n is the number of consecutive units that should be tested to ensure meeting the conditions set by specifying the APL, RPL, α , and β . Process standard deviation is the standard deviation to be used to determine the limits; it should approximate the standard deviation of the process measured when no assignable causes are present.

Once α , β , APL, and RPL are chosen, n is set and must be calculated from these quantities before the limits for acceptance control charts can be derived. The value of n is independent of the magnitude of the standard deviation, which is assumed known and is computed as follows:

$$n = \left[\frac{Z_{\alpha} + Z_{\beta}}{Z_{\text{APL}} - Z_{\text{RPL}}} \right]^2$$

where

 Z_a = normal deviate corresponding to α ,

 Z_{β} = normal deviate corresponding to β ,

 Z_{APL} = normal deviate corresponding to the APL, and

 Z_{RPL} = normal deviate corresponding to the RPL.

Knowing n and σ' the control charts limits are obtained as follows:

$$UCL = U - Z_{\alpha}\sigma' - Z_{\beta}\frac{\sigma'}{\sqrt{n}}$$
$$LCL = L + Z_{\alpha}\sigma' + Z_{\beta}\frac{\sigma'}{\sqrt{n}}$$

where

UCL = upper control limit, LCL = lower control limit,

- U = upper specification limit,
- L = lower specification limit,

 \mathbf{Z}_a = normal deviate corresponding to $\boldsymbol{\alpha}$,

- Z_{β} = normal deviate corresponding to β ,
- σ' = known standard deviation, and
- n = sample size.

It should be emphasized that these limits are derived using the specification limits as reference points, whereas the reference point for process control chart limits is the design or specified mean, as shown in Figure 6 (9). This is not accidental. It is consistent with the assumptions that the specification range should be greater than $6\sigma'$ to use acceptance control charts and that the process mean can shift about the design mean without producing defectives so long as the standard deviation remains unchanged.

Applicability to Concrete

The objective of acceptance control charts is to reject processes whose output equals or exceeds the RPL. This objective limits their applicability to only those properties that can be measured immediately after manufacturing. If those properties cannot be measured immediately after production, use of these charts leads to two difficulties. First, if the process shifts to the rejectable level, defectives will be produced during the time lag between production and testing. Depending on the time elapsed, this can result in accepting substantial amounts of inferior product. Second, a process operating at a rejectable process level at the time the sample is produced can shift back to an acceptable level while waiting for test results. When this happens, rejecting the process on the basis of the last available data leads to rejecting an acceptable process and causes unnecessary manufacturing delays.

Concrete properties that can be measured immediately after mixing are slump and air content, and in principle acceptance control charts can be used for these properties provided that sampling and testing are performed at the plant site. But, although applicable in theory, the use of acceptance control charts for slump and air content is neither practical nor desirable. They are not practical because no saving in testing is realized, and they are not desirable because conditions for the use of acceptance control charts do not exist.

Acceptance control charts are desirable if the following conditions are satisfied:

1. The specification range is wide enough to accommodate shifts in process averages of considerable magnitude without resulting in defectives,

2. The process standard deviation is known and stable,

3. The production units included in the subgroup represent consecutive production, and

4. The decision of rejection can be enforced.

Neither slump nor air content meets these conditions, for reasons that will now be discussed.

Specification Range-Acceptance control charts are used to give producers of a uniform product an advantage when the specification range is considerably greater than six times the process standard deviation. If the specification range is very large, compared to the six standard deviations needed to meet the specifications, the process average can be allowed to shift considerably without resulting in defectives. Under these circumstances both acceptance sampling and process control can be relaxed (Fig. 7). The specification range is 12 process standard deviations. But for most properties the specification limits need provide only a range of six standard deviations to eliminate nearly all defectives. This means that the process average shown in Figure 7 can shift to $\pm 3\sigma'$ from the nominal design value while producing almost no defectives. Only when the process average moves outside the shaded area will defectives begin to be produced. But large shifts are not likely to occur, and thus the chances for defectives are almost nonexistent. Because production of defectives is unlikely, the producer need not be particularly meticulous about process control. He needs only to prevent very large shifts in the process average, which usually take very little effort to avoid. Similarly, the buyer is not likely to receive defectives and can afford to accept the material so long as the process is monitored to prevent large shifts in process level. To ensure this, he can rely on acceptance control charts, using his own data or the producer's data. But for slump and air content, the specification range usually approximates the needed six standard deviations (4). This means that small shifts in the process level are likely to result in large fractions defective. For this reason, to ensure that process control is pursued, concrete buyers should use acceptance sampling and rely completely on their own data, and acceptance control charts are inappropriate.

<u>Standard Deviation</u>—In the preceding discussion, it was tacitly assumed that the process standard deviation was known. In fact, the process standard deviation for slump and air content changes from plant to plant (4). Moreover, there is no impartial way to assume a safe value. If small standard deviations are assumed, producers of unacceptable quality are rewarded and buyers penalized. If large standard deviations are assumed, producers of uniform quality will suffer unnecessary and unfair rejection. These points are most important, and one may convince himself of their validity with a few simple sketches. This means that, to be fair in setting up acceptance control charts, the standard deviation should be determined for each separate concrete plant, and control chart limits would have to change from plant to plant. The result would be an administrative nightmare.

<u>Subgroup</u>—For acceptance control charts, the sample should consist of consecutive production units. Recovery of such a sample is a difficult task for concrete, even if the sample size is small. The sample size for acceptance control charts depends on α , β , APL, and RPL and can be relatively large. For example, for an APL of 0.003 and RPL of 0.036, the necessary sample size is 10 if $\alpha = 0.05$ and $\beta = 0.10$. If higher quality levels were required, the sample size would be larger. These relatively large sample sizes make recovery of samples consisting of consecutive or almost consecutive production units a difficult task. This is another drawback for acceptance control charts.

Enforcement of the Rejection Decision—As already discussed, acceptance control charts accept or reject a process and not a finite or tangible amount of material. If the buyer uses acceptance control charts, he can encounter difficulties in enforcing rejection. When a process is rejected, a producer can refuse to look for assignable causes. In such cases, the buyer cannot really enforce his decision. He can stop buying the product, but, if the producer has an alternative, less demanding market, he may not care. Because the decision does not involve material, but rather doing something totally under the producer's control, the buyer must depend on the producer's cooperation. Within the same company, acceptance control charts can work because the producer and those responsible for process acceptance report to the same manager. In such cases, disputes can be quickly resolved with no necessity for litigation. But in a vendor-vendee relationship, this arrangement can lead to problems.

<u>Amount of Sampling</u>—From the point of view of testing and sampling, there is no advantage in using acceptance control charts. If a point on the acceptance control chart represents the amount of material as a lot, then to ensure the same quality levels with the same risks, acceptance control charts and sampling plans by variables with standard deviation known require the same sample size. In fact, sample size is computed with the same formula. But for an acceptance sampling plan, sample size must consist of a random sample. This is an advantage because sampling of consecutive concrete production units is difficult, and acceptance sampling plans by variables with standard deviations known are preferable to acceptance control charts.

To summarize, then, acceptance control charts lose on all accounts, and their use in concrete inspection is not appropriate.

SUMMARY

It is hoped that this paper has served to stress that for concrete

1. Process control is a difficult task requiring (a) constant sampling and testing, (b) constant attention of plant managers, and (c) constant care by manufacturing personnel; and

2. Concrete buyers should avoid assuming responsibility for process control because (a) it requires interfering with management of the production process, (b) it requires skills that concrete inspectors cannot be expected to possess, (c) it requires decisions that are properly the responsibility of plant managers, and (d) it could require more sampling than acceptance sampling.

It is also hoped that the discussion of acceptance control charts makes it clear that process control charts and acceptance control charts cannot be used interchangeably and that acceptance control charts are not appropriate as a replacement for acceptance sampling in concrete inspection.

Finally, the author hopes that those responsible for buying concrete will read the literature referenced in this paper before deciding to use process control or acceptance sampling to ensure quality concrete. The author is confident that the informed buyer, except on rare occasions, will choose acceptance sampling.

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