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Air Sampling, Quality Control, and Concrete

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Contents

Design for Air Monitoring Surveys Near Highways

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Because it is required by the National Environmental Policy Act of 1970, monitoring of air pollution near highways has been recently undertaken by many state transportation departments, private highway builders, and transportation planning organizations. This paper discusses the design of surveys for air monitoring near highways and includes an overview of the systems approach to survey design and equipment requirements in addition to definitive methods for the selection of sampling sites and the numbers of samples required. The methods presented for site selection employ results from modeling the atmospheric diffusion of highways. In addition to modeling, survey design requires expertise in analytical instrumentation, data acquisition, meteorological data interpretation, statistics, and systems engineering. This paper shows how to integrate these various disciplines so that cost-effective surveys for air monitoring can be designed.

In recent years, the monitoring of ambient air has become an important part of many environmental impact statements because it is a requirement of the National Environmental Policy Act (1). The provisions of this act have also made air quality surveys more complex: They require comprehensive planning to ensure that prescribed objectives can be attained in the shortest possible time and at the least cost, as well as adequate planning and good survey design since the actual monitoring is an expensive and time-consuming undertaking that requires skilled personnel and sophisticated analytical equipment.

A well-designed air quality survey (2, 3) will

1. Set the objectives of the air monitoring investigation;
2. Determine the physical parameters to be measured;
3. Set the network specifications, including the location of air monitoring stations, the duration of the study and sampling schedules, and the air-sampling method to be used;
4. Set the specifications for the individual stations in the network, including the equipment needed to conduct the study, the method and frequency for equipment calibration, and the methods for recording data; and, finally,
5. Determine the type of data analysis to be performed and the method for reporting data.

It should be recognized that these steps are interdependent and that a properly designed survey would consider each step, in order, in terms of its effect on the other parts of the survey design.

One of the most critical elements of a survey design for air monitoring is the location of sampling sites. Criteria for site selection have not been well defined in the literature; they consist mostly of recommendations that are subjective and general. Experience and technical judgment have been essential for determining the number and location of sampling sites.

It is the objective of this paper to present methods for applying available mathematical dispersion models to assist in site-selection decisions. This paper provides a discussion of the role of air monitoring in assessment of environmental quality and of the planning and management process. A detailed methodology for the selection of air monitoring sites near highways is then provided.

Most of the available information concerning the location of an individual sample or a continuous monitoring station is directed toward ensuring that a representative sample, free of undue influence from the immediate surroundings, is obtained (4, 5, 6, 7). The guidelines consider (a) the uniformity of site locations in terms of height above ground level; (b) avoiding constraints to airflow from any direction; (c) the selection of surrounding areas that are free of stacks, chimneys, or other local emission points; and (d) the most suitable elevation for a representative sample, especially in residential areas—3 to 6 m (9.8 to 19.7 ft) is suggested.

The need for nationally standardized criteria for selecting locations for monitoring stations has been suggested (8) since there have been indications that air monitoring stations located near city streets give drastically different results when moved only a short distance. Because of the complex nature of the spatial variations in urban carbon monoxide (CO) concentrations, a dual monitoring system has been proposed that consists of a background exposure station location outside the range of influence of nearby traffic and a pedestrian exposure station that has a sampling probe above the sidewalk. The

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pedestrian exposure station provides a means for measuring the individual's exposure to CO levels on downtown streets. The background exposure station measures the CO concentrations that occur over a large physical area of the city.

The design of an air monitoring network involves a trade-off between what is considered desirable from a strictly technical point of view and what is feasible with the available resources. The following guidelines suggested by the Environmental Protection Agency (EPA) (4) are important criteria for selecting network sites.

1. The priority area is the zone of highest pollutant concentration within the region. One or more stations should be located in this priority area.
2. Close attention should be given to densely populated areas within the region, especially in the vicinity of heavy pollution.
3. The quality of air entering the region must be assessed; therefore, stations must also be situated on the periphery of the region. Meteorological factors such as frequencies of wind direction are of primary importance for determining the locations of these stations.
4. The effects that future development will have on the environment should be considered; therefore, sampling should be undertaken in areas of projected growth.
5. A major objective of surveillance is to evaluate the progress made in attaining the desired air quality; therefore, sampling stations should be strategically situated to facilitate the evaluation of the implemented control tactics.
6. Air quality information that represents all portions of the region should be available.

MONITORING AND MODELING IN AIR QUALITY ASSESSMENT

To determine the environmental impact of a new or existing highway and to provide for routine source surveillance or the operation of a control program for intermittent air pollution, it is necessary to estimate air pollution concentrations by monitoring, by modeling, or by a combination of these methods. The interrelation between monitoring and modeling that can lead to an optimum design for an air quality survey is shown in Figure 1. As it indicates, the purpose of mathematical models is to quantitatively combine the effects of source strength and meteorology to describe the resulting ambient air pollution concentration. Source strength is affected by a number of variables that include the size of the source, variable emission rates, and the efficiency of equipment employed for air pollution control. The meteorological factors that affect air pollution control are wind speed and direction, atmospheric stability, inversion height, and terrain features. To be useful, mathematical models must be able to account for all these parameters.

Air pollution models vary in complexity from simple models that measure atmospheric dispersion on a microscale to sophisticated models that measure multisource factors on a mesoscale by describing transport, dispersion, and photochemical reactions of pollutants. Microscale models are used to estimate the ambient pollution levels near a single source or project ($C_{project}$). Mesoscale models are used to estimate the areawide impact of a proposed source or project or the background concentrations ($C_{background}$) due to other sources.

Ambient air pollution concentrations occurring downwind of a source consist of two components: pollution contributed directly by the source and the background. In most analyses, these components should be determined separately. The total air quality impact, represented in Figure 1 by the concentration C_x, is equal to the sum of the background plus the concentration contributed by the highway under study. Whenever other major highways are nearby, their contribution of pollution must also be added to the highway contribution. The objective of most air quality surveys is to determine the maximum value of C_x as accurately as possible and to compare that concentration with the National Ambient Air Quality Standards (9).

The role of monitoring is to measure ambient pollution concentrations, meteorological parameters, and source strength parameters. Air monitoring at carefully selected sites provides for direct measurement of background concentrations. Measurements of meteorological or source strength parameters can be used to verify model input data or to validate models that measure the atmospheric dispersion of air pollution. Measurements of source strength, meteorology, and both microscale and mesoscale (background) air pollution concentration are required to validate the microscale models. The measurements of source strength, meteorology, and air pollution must all be representative of areawide conditions to validate the mesoscale models.

A basic premise of the design approach presented here is that air quality measurements can best be used to supplement and verify air quality predictions from mathematical models since the spatial and temporal variations of air pollution concentration that occur in the environment are too complex to resolve by monitoring alone (the numbers of stations required approaches infinity). For this reason, the site-selection methods presented are designed to provide data for model validation. Validated models can then be used to determine spatial variations in pollution levels since they can be used to estimate concentrations even at sites where monitoring was not performed. Thus, a greater amount of air quality information can be derived from models than by monitoring alone.

OBJECTIVES AND PLANNING FOR AN AIR MONITORING SURVEY

Figure 2 shows an overview of the important decision points required in the initial phases of planning a survey. In general, the broad objectives of any study for monitoring air quality are to determine the extent of the existing air pollution problem and to validate any mathematical models or assumptions used to estimate the future impact on air quality of a new highway or change in emissions from an existing highway. More specific objectives may include

1. Checking for compliance with ambient air quality standards at critical locations,
2. Determining when and where the worst case background pollutant concentrations occur within the impact area,
3. Validating or calibrating a mesoscale model to accurately predict future background concentrations, and
4. Validating or calibrating a microscale model to accurately estimate the air quality impact of a proposed facility or change in emissions from an existing facility.

Consideration should also be given to the potential exposure of people to air pollutants. If sensitive receptors such as children, the elderly, or the sick are to be exposed to additional pollutant concentrations after the completion of a proposed project, it may be important to document the current levels of exposure of sensitive populations so that the future levels of exposure can be more accurately estimated.

Once the objectives of the air quality study are de-

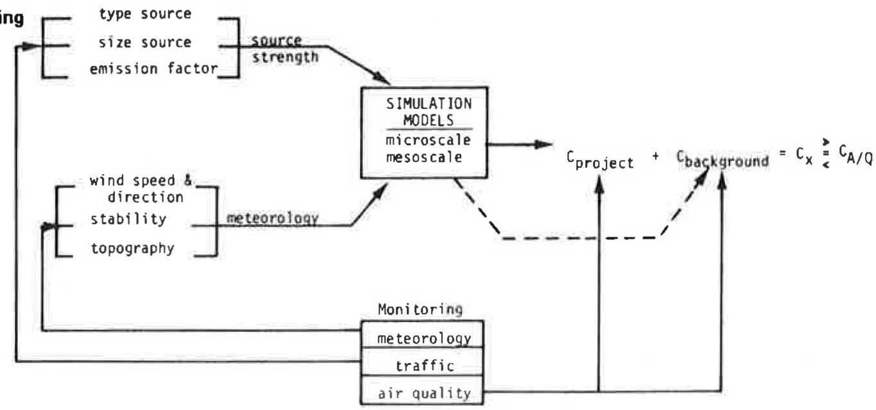


Figure 1. Interaction of modeling and monitoring to assess the air quality impact of a project.

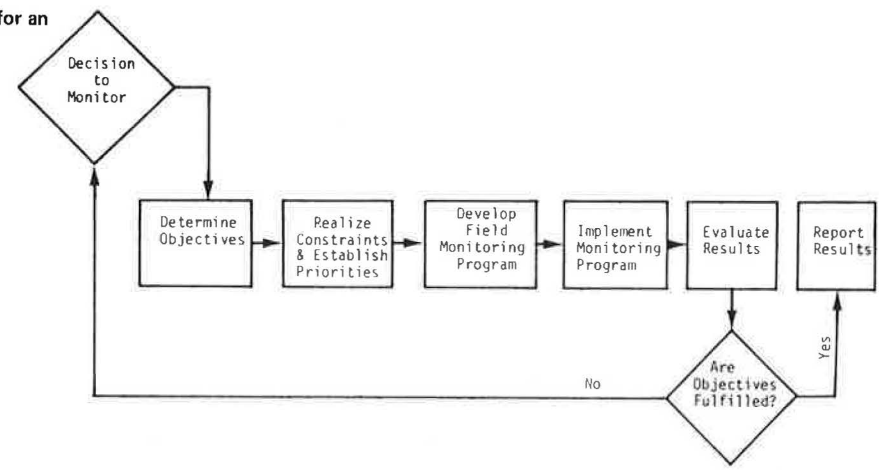


Figure 2. Planning and management process for an air quality survey.

termined, it is possible to develop and implement a field monitoring program that will fulfill the objectives. By working within the constraints of money, manpower, and time, priorities can be set in terms of the objectives that can be realistically met within the cost constraints imposed on the air quality study. If available resources are severely limited, a study for monitoring air quality may not be advisable at all because a poorly planned, poorly financed, and hurriedly conducted air-sampling survey will not provide data that can be easily evaluated in terms of the survey objectives and may thus detract from, rather than improve, the technical quality of the final report.

AIR MONITORING NETWORK SPECIFICATIONS

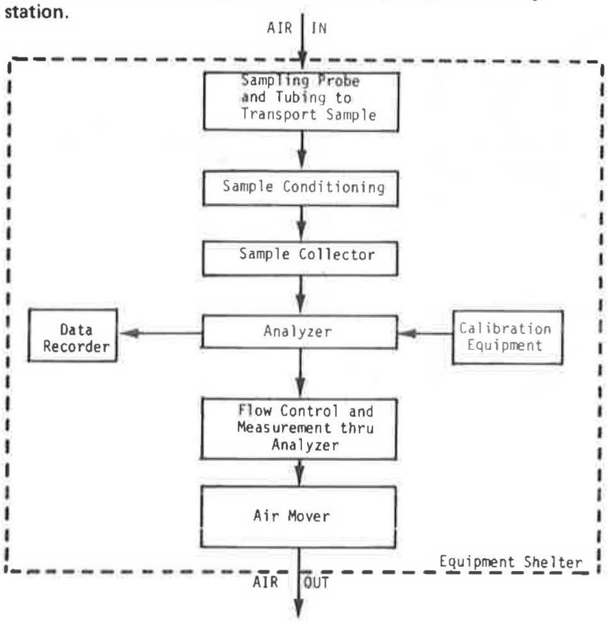
The sum total of all air monitoring stations, meteorological stations, calibration equipment, and data-acquisition equipment required to meet the total objective of an air quality survey represents the air monitoring network. To understand the interrelation between the component parts of the network and to allow decisions to be made about the number and type of each piece of equipment and their interdependence in meeting the survey objectives, a set of specifications for the network must be developed early in the planning process.

Network specifications for air monitoring should include (a) the number of sites to be monitored, (b) the air pollution and meteorological measurements required at each site, (c) the duration of the study, and (d) the manpower requirements. Network specifications should be determined in light of the known limitations of the physical, engineering, economic, and human factors, as well as the limitations due to completion deadlines or time delays in equipment procurement. Time, manpower, and budget constraints may significantly affect the feasibility of using specific techniques or equipment to conduct the study.

By considering all of the stated sampling requirements and available resources, the types of air quality and meteorological monitoring equipment to be employed, including the total number of samplers and analyzers for each type of measurement to be performed, can be identified. A consideration of possible equipment tradeoffs should also be evaluated, e.g., the number of continuous analyzers versus semiautomatic samplers versus manual sampling methods. Obviously, the mobility of air-sampling equipment should be considered and factors such as the high capital costs of continuous analyzers versus the high operating costs of semiautomatic or manual methods should be optimized for cost-effectiveness.

The calibration system to be used for ensuring that accurate data will be obtained from all the analyzers should be considered next. Each station can be equipped with calibration hardware, or calibration procedures can be employed either when portable instruments are calibrated at a central laboratory or when permanent instruments are calibrated with a portable calibration device. The choice of calibration equipment will depend on the number of stations, the distance separating the stations, and the frequency with which calibrations are performed. A typical integrated calibration system might consist of sophisticated apparatus used for dynamic calibration in a central laboratory and several portable calibrators or span gas bottles that are used for frequent on-site single-point calibration checks.

Figure 3. Hardware components of an air quality monitoring station.



Once the air monitoring and calibration specifications have been fixed, the requirements of the data-acquisition system can be evaluated. The data rates and data quantities generated by the monitoring network should be determined as a whole to calculate the data-handling capacity needed for the overall sampling network. Data-recording equipment that is ideal for a single monitoring station may be too expensive for use at numerous monitoring stations. Frequently, it is less expensive to replace a variety of special devices with a central general-purpose control element, even though the speed and flexibility requirements of the system do not require it. Data telemetry and central data-processing systems may represent the most efficient approach for monitoring networks that have many remote sampling stations that employ continuous air pollution analyzers. Conversely, for single-station applications, strip-chart recorders with manual data reduction may represent the most cost-effective method for data collection.

The network design will determine the number of air monitoring stations to be employed and set the functional sampling objectives for each station. At this point, station specifications for air monitoring can be developed for each monitoring site in the network. Station specifications include the setting of sampling objectives for each station and the selecting of compatible hardware components for the station, thus producing an integrated design for a station that monitors air quality samples.

Figure 3 shows the nine discrete component elements used in a station designed for air monitoring. The sequence of selection of station equipment should be arranged so that the primary components are chosen before the supportive or supplementary hardware. For example, equipment shelters should be designed after the total amount of equipment to be housed in the shelter has been determined; thus, the shelter will be of the proper size and will have adequate electrical wiring, plumbing, instrument mounting, and storage space for the hardware components.

SELECTING AIR MONITORING SITES

Choosing the right location for the right monitoring objective requires insight into the nature of air pollution emission, transport, and dispersion from different types of sources. Because many variables must be considered, site selection is one of the most complex and critical elements in the design of a survey for monitoring air quality. If the wrong sites are chosen or if a critical site is missed, no amount of accurate data will allow the objective of the study to be fully realized. Variables to be considered regarding the transport and dispersion of pollutants include the relative locations of pollution sources (inside and outside the study area), sensitive receptors, and the effects of topography, source size and configuration, and meteorology.

Before air monitoring sites can be effectively selected, an understanding of the spatial distribution of pollution concentration is necessary. It is useful to define three separate air pollution regimes, shown in Figure 4.

1. Microscale—The microscale air pollution regime represents a relatively small air mass that exhibits large variations in air pollution concentrations at ground level. This phenomenon usually occurs close to sources of air pollution when the rate of increasing atmospheric dispersion with downwind distance is great.
2. Mesoscale—The mesoscale regime represents a community-sized air mass that exhibits fairly homogeneous ground-level concentrations of air pollution, such as the ambient concentrations within urban areas that are caused by the emission of relatively small quantities of air pollutants from a large number of ground-level sources (i.e., automobiles, residential and commercial space-heating furnaces, and even numerous small industrial sources). These local background concentrations can vary considerably at different locations within an urban area.
3. Macroscale—A macroscale regime represents a regional background with air pollution concentrations that can be fairly homogeneous over linear distances of from tens to hundreds of kilometers. Large variations in pollutant concentration indicate the presence of mesoscale and microscale air pollution regimes that are superimposed on the regional regime.

Pollution levels near major sources consist of three component parts: the microscale concentration (the concentration directly due to nearby pollution sources), the mesoscale concentration (the local background concentration due to areawide sources of pollutant emissions), and the macroscale concentration (the regional background concentration due to distant pollution sources). The macroscale pollution concentration is frequently so low that this term can be ignored, leaving only two components to any ground-level observation—the local background air pollution concentration and the microscale concentration due to a nearby source. Air monitoring sites located outside the microscale regime measure only mesoscale and macroscale concentrations (background). Sampling sites located within the microscale regime measure the concentration due to the combined microscale, mesoscale, and macroscale regimes.

Monitoring Sites Near Highways

Whenever air quality near a specific highway is to be monitored, mathematical dispersion models should be employed to assist in the determination of sites for optimum air sampling. Line-source models are available (10, 11) that can be used to calculate where the maximum ground-level concentrations are expected to occur. Models can also be used to determine the profiles of ground-level concentration at various distances from the source so that the extent of the impact area can be determined.

According to the California line-source dispersion models (10), pollution emitted from automobiles is thoroughly mixed above the highway in the mechanical mixing cell, a region in which concentrations are uniform and relatively high. As the pollutants are transported and dispersed by the wind, the concentrations decrease as the distance away from the highway increases. When the concentration approaches the local background concentration, the boundary of the microscale regime has been reached (2).

This microscale regime can be illustrated graphically by plotting isopleth lines of concentration levels as a function of the concentration in the mixing cell. Figure 5 shows lines of similar concentration downwind from the edge of a highway source in both the horizontal direction, plotted on the abscissa, and the vertical direction, plotted on the ordinate. The family of curves illustrates the vertical and horizontal locations at which the pollution concentration is reduced to 80, 60, 40, 20, and 10 percent of the original levels. (All the curves plotted were determined for crosswind conditions and C stability.)

These curves may be used to locate optimum sampling sites for validating the microscale model. Sites should be spaced so that the pollution gradient can be sampled at equal intervals of decreasing concentration. This technique helps minimize the effects of errors in experimental measurements (ensuring measurable differences among sites) and avoids redundant sites. As described in Figure 4, the optimum locations for the five sites would be 0, 2, 7, 23, and 200 m (0, 6.5, 22.9, 75.4, and 656 ft) from the edge of the road.

Figure 5 provides optimum site spacing for C stability and crosswind conditions but less than optimum site spacing for other conditions. To design the best possible site-spacing pattern, an evaluation should be performed that uses historical meteorological data and diffusion models to determine the full spectrum of events likely to occur downwind of the source. Figure 6 shows the results of such an evaluation and the normalized concentration gradient versus normal distance from the road for all possible conditions described by the California line-source model (10).

Since the precise conditions of meteorology that will be encountered in the field cannot be predicted, the sampling sites should be selected according to optimum spacing for the most probable stability and wind direction condition determined from historical records. Choosing sites that use the most probable stability and wind direction should provide the greatest amount of data collected from sites that are optimally located. If optimum spacing of sites is desired for measurements under a different condition of meteorology, particularly the worst case condition for dispersion (e.g., F stability and parallel wind for at-grade highways) that occurs less often, additional sampling sites may be added to supplement the coverage provided by sites already chosen. In this way, the sampling array is designed to provide the best data under specific meteorological conditions (most probable and worst case), while providing less than optimum coverage for other conditions likely to be encountered during the study.

The site-selection procedure can be described by an example. The sites shown in Figure 6 that are located at distances 1, 2.3, 7.2, 33, and 260 m (3.3, 7.5, 23.6, 108.2, and 852.8 ft) from the road were chosen to allow sampling at regular intervals of 100, 80, 60, 40, and 20 percent of the roadside mixing cell concentration (C_{mc}) and within a measurement error of 10 percent of C_{mc} for crosswind conditions that, in this example, occur 85 percent of the time. The site locations shown in Figure 6 should meet their intended objectives 100, 85, 81.6, 78.2, and 51 percent of the time for sites 1 through 5 respectively. These sites, which measure roughly 100, 97, 89, 53, and <1 percent of the mixing-cell concentrations for sites 1 through 5 respectively, do not provide good coverage for parallel wind conditions. If model validation under parallel winds is desired, then additional sites may be needed to measure ~70 and ~25 percent of C_{mc}.

The optimum location of each site is the distance from the road at which the target concentration (e.g., 40 percent of the mixing cell) can be measured most frequently (i.e., where the tolerance limits cross the concentration profile lines that have the greatest frequency of occurrence). In the example given, the poorest coverage is at site 5, which meets its stated objective only 51 percent of the time. An alternative design, which should provide measurements of 20 percent C_{mc} ~78 percent of the time, would move site 5 to 150 m (492 ft) (site 5A) and add a sixth site at 600 m (1968 ft) (site 6). Site 5A could be used to measure 20 ± 10 percent C_{mc} during stabilities B, C, and D, and site 6 could measure 20 ± 10 percent C_{mc} during D, E, and F stabilities. This sixth site should allow an additional 27 percent of the data collected to be within the stated objectives at all sites. The use of a sixth monitoring site may be the most cost-effective design, especially if it allows the duration of the study to be shortened because of the improved coverage of the sites.

Mesoscale Sites

Data from downwind sampling sites describe the concentration of pollution from the project plus the background. For this reason, data from an additional station located outside the microscale regime but within the same mesoscale regime are required. These data separate the pollution concentrations into two components by subtracting the local background from the microscale concentrations measured. The contribution from the project can then be compared with model predictions. If the regional background levels are relatively low, a single properly chosen monitoring station can be used to establish background levels within the mesoscale regime. Usually, a microscale model-validation experiment will use an array of air samplers located on both sides of the highway. As long as winds do not blow parallel to the roadway, samplers on one side of the highway will provide background data, while samplers on the other side of the highway will measure the additional highway contribution.

Figures 5 and 6 can also be used to determine a good location for a mesoscale site. The isopleth lines can be used to estimate the theoretical boundary that separates the microscale regime of a nearby highway from the mesoscale regime. Once the boundary has been determined, stations designed to sample background concentrations can be located outside the boundary, while stations designed to sample pollutants from the highway plus background can be located within the boundary.

The isopleth that represents the boundary separating the microscale and mesoscale regimes depends on both the concentration within the mixing cell and the background concentration. Figure 7 provides a graphic method for determining which isopleth to use. The equations are

$$\% C_{mc} = (0.5 - BKG)/C_{mc} \quad (1)$$

where $C_{mc} \le 2.5$ or

$$\% C_{mc} = (0.2\, C_{mc} - BKG)/C_{mc} \quad (2)$$

where $C_{mc} \ge 2.5$. This calculation can be used to determine what percentage of the mixing-cell concentration

Figure 4. Definition of background air quality.

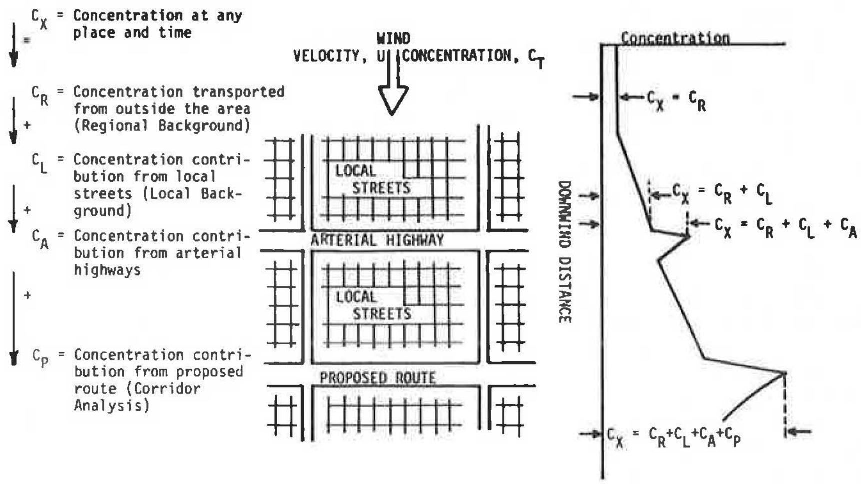


Figure 5. Isopleth concentration lines downwind of a highway line source.

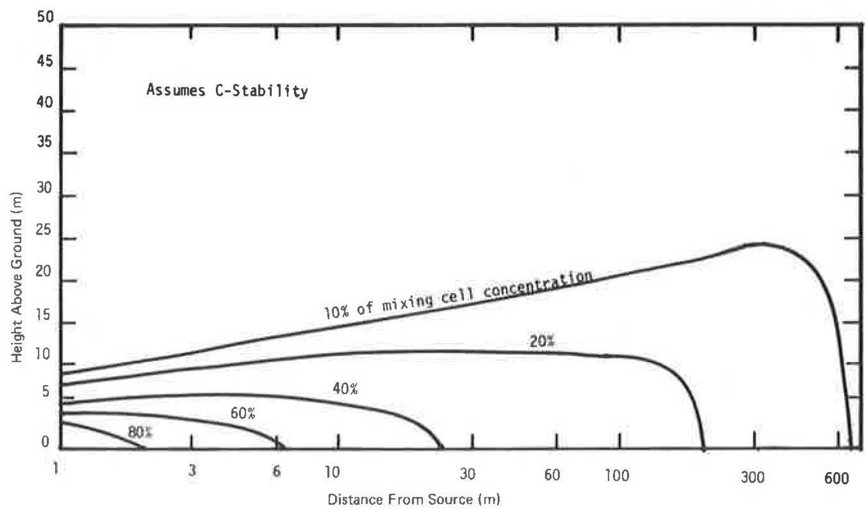
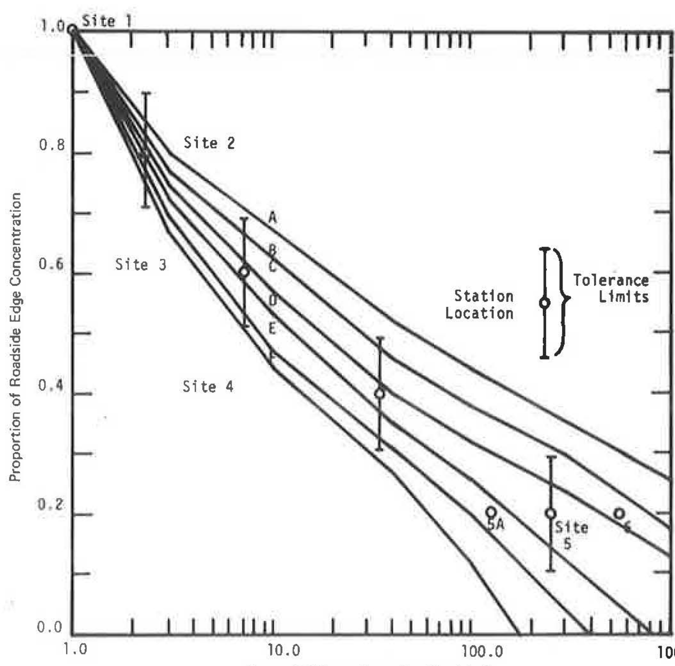


Figure 6. Optimum locations for air-sampling sites determined by crosswind conditions.



($\% C_{mc}$), when added to the background concentration (BKG), will increase the background concentration by 0.5 ppm of CO or 20 percent (whichever is larger). This means that an air monitoring site located outside the designated isopleth (measured as a percentage of the mixing-cell concentration) will measure the local background concentration with an error of up to 20 percent or 0.5 ppm CO (whichever is larger).

By using Figures 6 and 7, one can determine the minimum distance from a highway at which a monitoring site should be located for measuring background levels. There is also a maximum distance within which the sampling site should be located if there is another major source of pollutants nearby. Figure 8 shows the maximum distance that a background site should be located from the highway corridor. The limiting value is based on a maximum error of 20 percent between the value measured at the distant air-sampling site and the expected local background concentration at the highway corridor.

In addition to measuring the ground-level sampling-site locations discussed above, background air pollution concentrations can be measured at air-sampling stations located on the tops of buildings. However, the buildings must not be too tall or the measurements will not represent the background concentration at ground level. Pollution from nearby sources may contribute significantly to the ground-level concentrations of the background but not to those measured on top of a tall building. For this reason, the height of the sampling site must be limited.

As in the case of the maximum horizontal distance away from a project corridor, the maximum height for a mesoscale monitoring site depends on the distance from the closest major source. Figure 9 shows the relation between the distance from a contributing source and the percentage of ground-level air pollution concentrations that occur at different heights. This figure can be used to measure and to determine (within an accuracy of 20 percent of ground-level background) the maximum height at which the local background concentrations occur. This error estimate is conservative, since it assumes that the closest major source contributes 100 percent of the ground-level concentration and 80 percent at the indicated height. In practice, no single source would be responsible for all of the local background pollution concentration because sources located at greater distances from the sampling site contribute more uniformly to the concentration both at ground level and on top of a building.

Figures 5, 7, 8, and 9 were developed by using the California highway line-source model for at-grade highway sections, crosswind conditions, and atmospheric stability class C. Stability class C was selected as the stability class most frequently encountered under daytime conditions with urban terrain. Clarke and McElroy (12) have reported measurements of a relatively unstable boundary layer over urban terrain that tend to reinforce the use of C stability as the most probable condition encountered within urban environs.

Locations for Validating Microscale Models

The data obtained for validating microscale models may be taken at monitoring sites at which

1. The highest concentrations from the project occur (due to large traffic volumes or narrow right-of-way),
2. The highway configuration and upwind topography are most representative of the whole project, or
3. The basic assumptions of the model are violated (due to highway configuration or topography).

The model may be validated at a location at which the highest concentrations occur because this is where the greatest confidence in the model is needed. Model validation at the most representative location allows the model to be applied generally to the whole project. It may be required to validate the model for irregular terrain (hills, valleys, or nearby tall buildings) or complex highway configurations (intersections, elevated or depressed sections, and unusual land configurations), since models are the least reliable when basic assumptions are violated, i.e., smooth, level terrain, uniform wind flow field, and wind speeds greater than 1 m/s (3.3 ft/s).

Once the sites have been selected, the highway route can be monitored by a cross-section sampling at various horizontal or vertical distances from the highway. Special attention should be given to measuring the air pollution concentration at the right-of-way edge and other upwind and downwind sites near the highway. Background concentration measurements are subtracted from downwind measurements to determine the contribution due to the highway.

NUMBER OF SAMPLES NEEDED

Statistical methods for determining the number of samples needed to accurately define the mean and maximum pollution concentrations expected to occur during a year have been presented in the literature by several authors (13, 14, 15). In general, these methods assume that the samples collected are representative of the total population, which is either normally or lognormally distributed, and that each sample is chosen randomly. Under these conditions, the simplified methods presented by Hale (13) can be conveniently used to determine the total number of samples needed.

The methods presented by Hale assume that samples are collected from a finite population. For a lognormal distribution, the number of samples (n) required to determine the tolerance and confidence interval is given by

$$n = (NZ^2 \ln^2 S_g)/[N \ln^2(P + 1) + Z^2 \ln^2 S_g] \qquad (3)$$

where

- N = population size,
- Z = normal deviate corresponding to the upper percentage point for a specified level of confidence ($Z = 1.96$ for a 95 percent level of confidence),
- S_g = standard geometric deviation of samples, and
- P = fraction of the geometric mean by which it can differ from the true geometric mean with specified probability.

Figures 10 and 11 are based on equation 3 and can be used to determine the number of samples needed as a function of the standard geometric deviation and the size of the population being sampled. Figure 10 should be used to determine the geometric mean within ±10 percent (at 95 percent confidence), and Figure 11 should be used for a ±20 percent tolerance (at 95 percent confidence).

DURATION OF THE STUDY

The duration of study required can be estimated by using the historical meteorological data and the statistical methods given above. The average and range of concentrations expected to be measured at each station can be determined by using the model calculations. The range divided by about six can be used to estimate the standard deviation. The number of samples needed can be estimated from Figures 10 or 11. The duration required

Figure 7. Percentage of mixing-cell concentration for effective measurement of background concentrations.

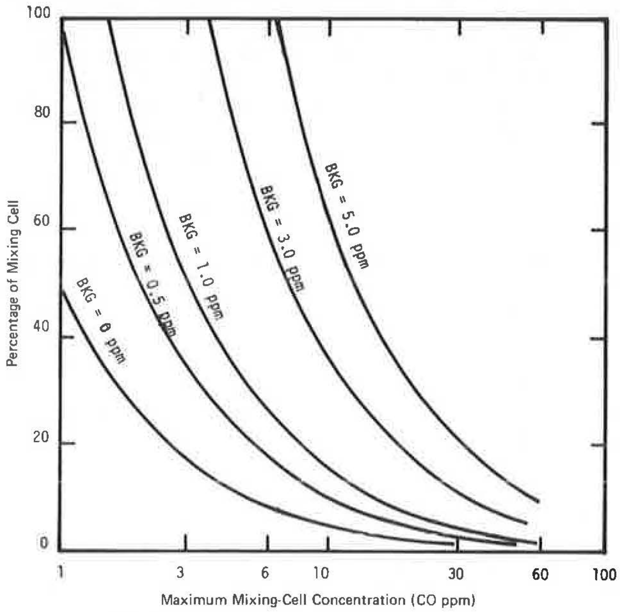


Figure 8. Maximum distance from project corridor for location of a mesoscale station.

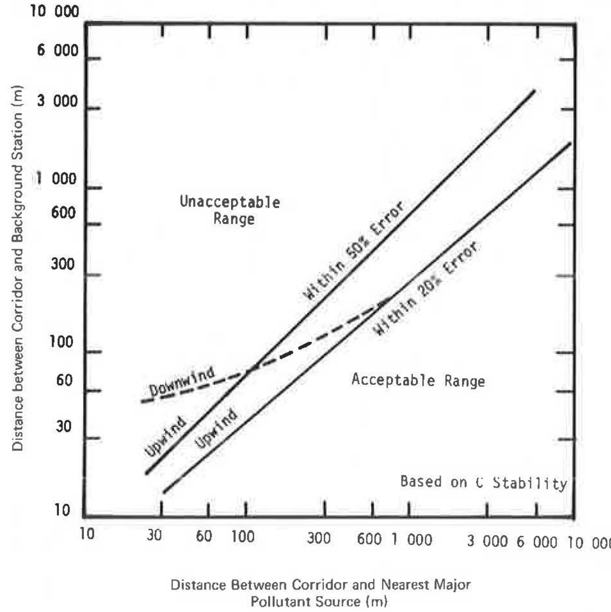


Figure 9. Height of a mesoscale monitoring station for measuring ground-level concentrations.

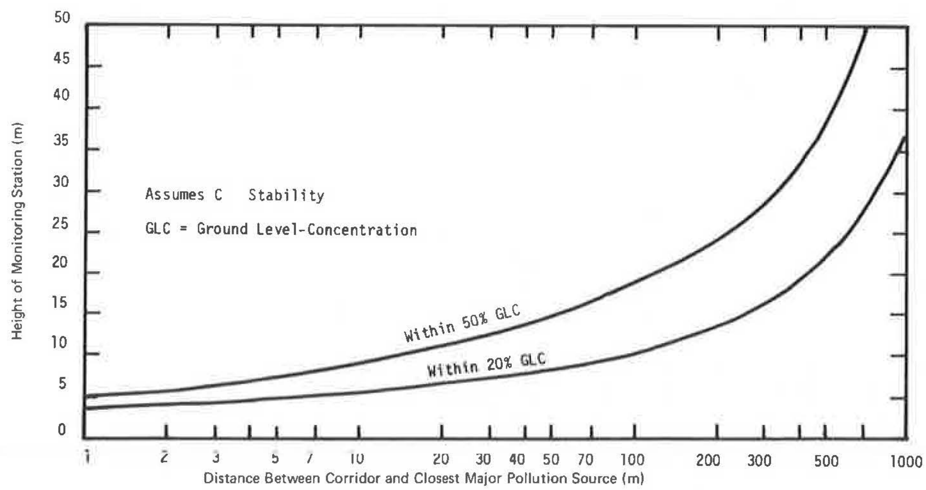
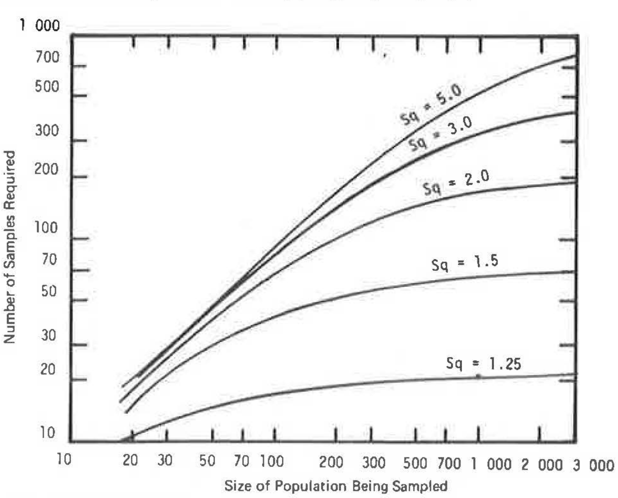
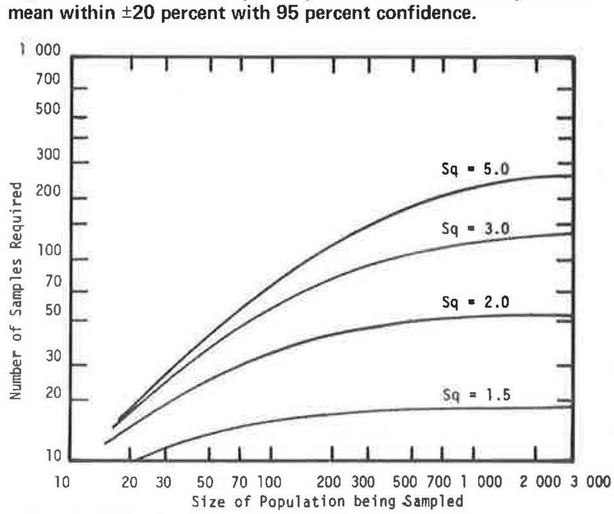


Figure 10. Number of samples required to determine the geometric mean within ±10 percent with 95 percent confidence.



Note: Z = 1.96 and P = 0.10.

Figure 11. Number of samples required to determine the geometric mean within ±20 percent with 95 percent confidence.



Note: Z = 1.96 and P = 0.20.

can then be determined by dividing the number of samples required (n) by the frequency of occurrence of a particular condition of meteorology (f) multiplied by the number of samples (S) collected at each site per day or hour (n/fS). Typically, values of n range from 30 to 100 samples and S might be from 1 to 4 per hour. For unusual events occurring less than 5 percent of the time and if 40 samples were needed with S = 4 per hour, then the duration of study required would be 200 h. At 12 hours a day of sampling, this would require a study lasting more than 2 weeks. For frequently occurring conditions, sufficient data might be obtained in a few days.

The sampling survey should be conducted for a sufficiently long duration that observations are made under a wide range of source-related and meteorology-related conditions. In most cases, air samples should be collected using 15 to 60-min averaging times, but sampling for shorter averaging times can be used if traffic and wind data are gathered for comparable intervals. Averaging samples for time periods longer than 1 h produces additional errors because changing wind direction has a nonlinear effect on the downwind concentration. Whenever the wind direction is not persistent, the short averaging time samples (i.e., 15-min averages) may be the best.

Air sampling may be conducted either during peak-hour traffic or 24 h/day. For model validation, it is desirable to sample during each hour of the day and night that there is enough traffic to produce measurable pollutant concentrations. In this way, a large amount of data can be collected over a short period of time to allow validation of the model under different meteorological conditions.

CONCLUSION

Air-monitoring survey design requires expertise in analytical instrumentation, diffusion meteorology, statistics, and systems engineering. The systems design overview and the definitive methods for site selection presented in this paper can be used to improve current survey design methodologies. If cost-effective surveys are to be designed, then available air sampling and analytical methods, atmospheric diffusion models, and historical meteorological data must be evaluated by using systems engineering principles. This procedure can result in objective decision making and design trade-offs that consider the magnitude of the survey required and the resources available to conduct the study. Without this approach, survey designs will be based on guesswork and subjective reasoning that may not be cost-effective or even adequate to meet the intended objective of the survey.

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Can Research in Volume Change Assist the Designer?

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Most theories on the time-dependent volume changes of concrete structures assume that when load is applied there is an initial elastic deformation followed by a long-term inelastic deformation. In reality, it is the instantaneous deformation that is seriously complicated by inelastic effects. Deformations at 1 year can be calculated with greater certainty than instantaneous deformations. The multiplicity of creep data in the literature, practically all of which are applicable only to a specific set of conditions and of no general use, is the result of the multiplicity of available aggregates. But the only properties of aggregates that influence creep of concrete are elastic properties and concentration, and these can be determined in 1 day. If there were a comprehensive data bank of creep behavior of cement pastes of various water-cement ratios and various degrees of hydration exposed to various ambient conditions, a designer would merely need to request a quick aggregate test to produce a good first approximation of volume change behavior of the structure.

Apparently, creep of concrete has not been a neglected area of research. The American Concrete Institute (ACI) published a bibliography (1) that listed 1279 entries on creep and shrinkage through 1966. The Cement and Concrete Association in Great Britain also published a bibliography (2) that contained 800 references through 1964. More recently, ACI published a supplement (3) on creep only that covers the years 1967 through 1970 and contains 256 entries. One might hope that with 1535 references available, the designer has all the information he or she might desire. But although the rate of publication is increasing, the need for new information is also increasing, and each year the designer appears to be less able to find the information he or she requires.

Most of the work that is reported is of a highly specific nature and is intended for a specific application with specific materials, or it is work reported only to satisfy an advanced degree requirement for a bright graduate student. Every few years, someone takes the available data and attempts to write a general expression for design purposes. These expressions usually suffer from algebraic complication and dependence on an imperfect data base.

An example of the needless complication of these expressions is found in the application of an expression to the shortening of high-rise buildings. Actual measurements have shown that 30-story buildings shorten 75 mm (3 in) in service, 50-story buildings shorten 125 mm (5 in), and the extensive instrumentation affixed to the 70-story Lake Point Towers (4) indicated a shortening of 175 mm (7 in). This uniformity of shortening of 2.5 mm (0.1 in) per story is not astounding. Column tests have shown that concrete in a reinforced-concrete column completely creeps out from under the load, leaving the load to be carried by the steel. The indicated strain of 0.0008 to 0.001 represents a steel stress of 172.4 to 206.8 MPa (25 000 to 30 000 lbf/in^2), a substantial but safe fraction of the available yield stress and one that might be repetitively used in design. Thus, if a complicated equation predicts an ultimate shortening different from 2.5 mm (0.1 in) per story, the higher order equation must be corrected to yield this value, regardless of the data base and the theory. Incidentally, steel buildings shorten the same amount, but the shortening occurs as soon as the construction loads are applied. In steel buildings, all the shortening has occurred by the time the first tenant moves in, whereas concrete buildings may require a year of occupancy before all the shortening has occurred.

In the literature, it is stated that when concrete is loaded there is an initial elastic deformation followed by a long-term inelastic deformation. Figure 1 shows the familiar rheological element known as the Kelvin unit, which consists of a spring and dashpot in parallel. Although the deformation of concrete cannot be adequately represented by such a unit, it is used here to illustrate a principle. If loads are applied to this unit, the resulting load-deformation curve can be used to calculate a modulus of elasticity. However, if these loads are applied at different rates, then different curves will occur (as shown in Figure 2), and the computation of the modulus of elasticity will be ambiguous. However, this is only useful if we are trying to determine the spring constant of the Kelvin unit, which it appears can only be done by applying a load and leaving it on the unit for an infinite period of time. Figure 3 shows that, although a given load may be applied at different rates, ultimately

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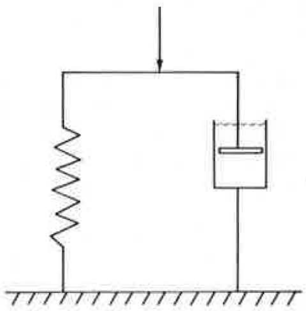


Figure 1. Kelvin rheological unit.

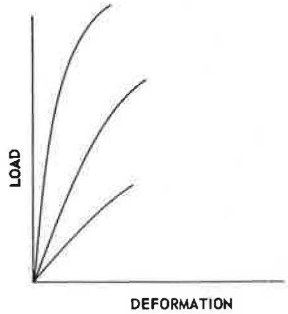


Figure 2. Load deformation curves for Kelvin unit.

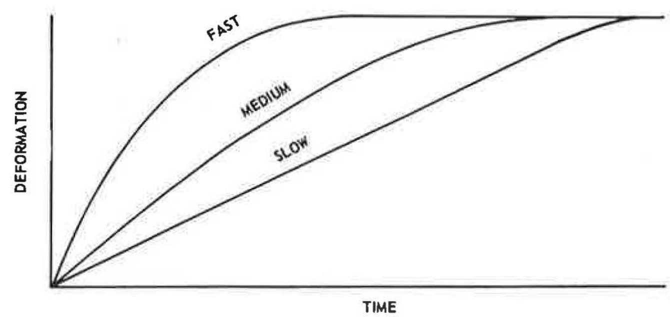


Figure 3. Ultimate deformation of a Kelvin unit.

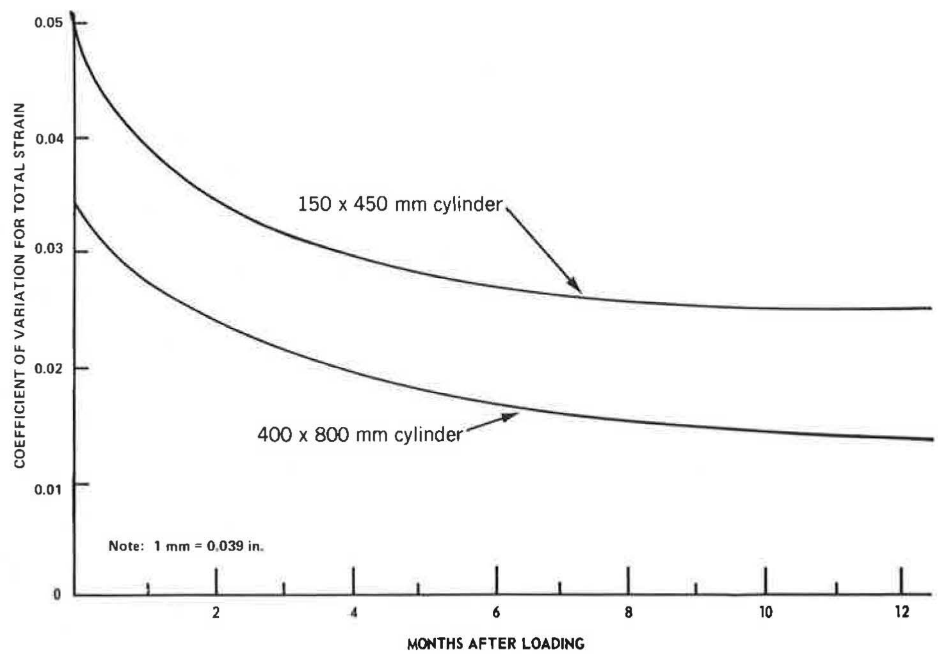
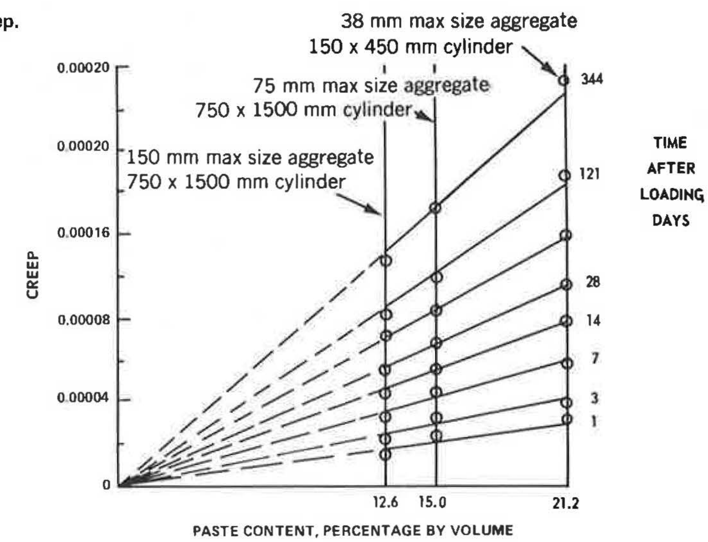


Figure 4. Effect of duration of load on variability of strain.



Note: 1 mm = 0.039 in.

Figure 5. Effect of paste content on creep.

Figure 6. Consequence of assuming a linear relationship between creep and paste content.

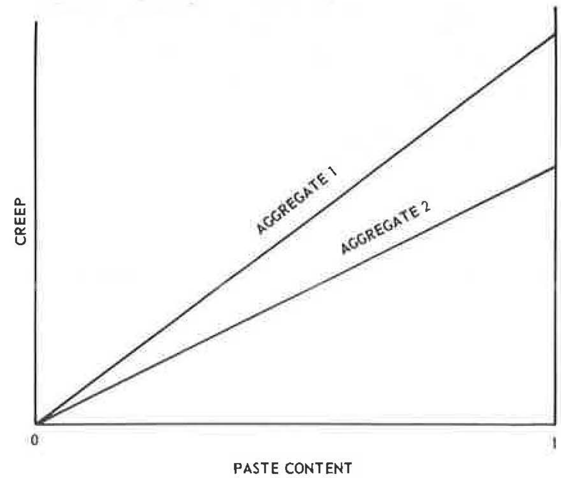
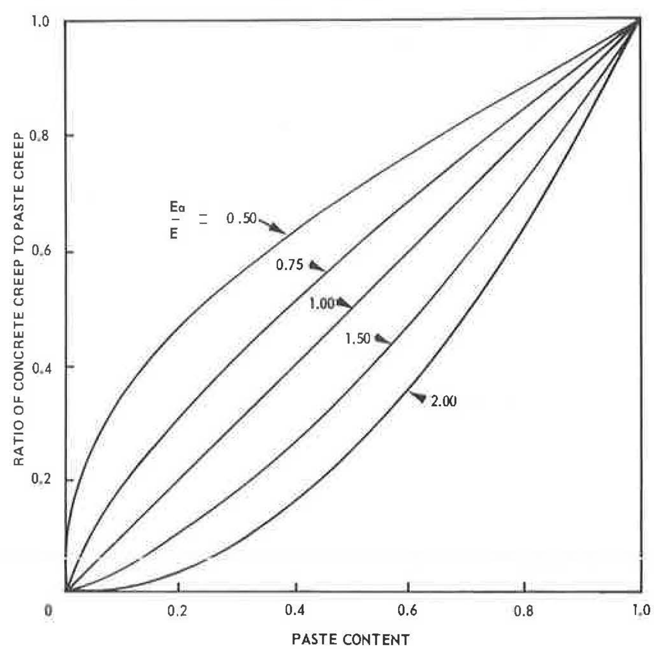


Figure 7. Dependence of creep on aggregate stiffness and paste content.



a fixed deformation will be attained so that the modulus of elasticity can be calculated. Thus, for a Kelvin unit, which like concrete has a time-dependent deformation, it is incorrect to state that there is an initial elastic deformation followed by a long-term inelastic deformation. There is, in fact, a long-term elastic deformation that is often confused in early ages by inelastic effects.

If concrete exhibits similar behavior, then it would be well to know about it, since design procedures are based on the assumption that there is an inherent modulus of elasticity that can be determined in a short time and that long-term effects can be estimated by multiplying the characteristically instantaneous deformation by appropriate factors. If, however, inelasticity manifests itself during the early part of the loading cycle, we are measuring a characteristic property at a very uncharacteristic time. To examine this possibility, the data published by Polivka, Pirtz, and Adams (5) regarding mass concrete are of interest, since the results of individual specimens as well as averages are given. Figure 4 shows the variability of total strain measurements as a function of time after loading. The results indicate that for two different specimen sizes the longer the time span is the more certain are the strain results. The use of coefficient of variation as the index of variability may be questioned, but it appears to be the proper parameter since, if late-age strains are estimated by multiplying the instantaneous strain by a factor, it is the percentage error in the original strain that is of concern rather than the absolute error.

The data compiled by the Subcommittee on Elastic and Inelastic Properties of the American Society for Testing and Materials (ASTM) for preparing precision statements for creep and for static modulus of elasticity demonstrate the same phenomenon. In Shideler's data on structural lightweight concrete (6), for 31-MPa (4500-lbf/in^2) concrete loaded 7 days, the variation among aggregates is such that the coefficient of variation for a 1-year strain is only about half that of the instantaneous strain. If all that is known about the concrete is that it is lightweight, predictions concerning the 1-year deflection can be made with twice the accuracy of those for instantaneous deflections. Calculating an instantaneous E-value makes the measurement at precisely the wrong time since it is vulnerable to small variations in the loading rate of the test specimen and small variations in the inelastic characteristics of the specimen, a possible result of variations in absorbed water somewhere in the system. A late-age deformation appears to be more nearly a characteristic of the given concrete.

There is another factor to be considered when reinforced concrete is used. In a paper on time-dependent deflections in a box-girder bridge (7) Adrian Pauw said: "... ultimate sustained load deflections may be calculated on the basis of some reasonable reduced modulus value.... Conversely, the initial dead load deflections are much more difficult to predict since the effective moment of inertia is quite dependent on the degree of cracking and hence the loading history of the structure." Thus, from both a materials and structural point of view, designing for a 1-year deflection appears to be a more certain procedure than designing for an instantaneous deflection. If the latter is needed, it can be calculated from the former. In most transportation applications, the dimensional tolerances are critical and the owner is less interested in the dimensions immediately after the forms are stripped than in those that will exist a year later.

AGGREGATE EFFECTS

Polivka and others (5) were concerned with the problem of determining creep in mass concrete and the expense involved in making and testing specimens with a 150-mm (6-in) aggregate. To determine the relevance of testing similar concrete with some of the larger particles removed, Polivka and others developed the data shown in Figure 5, reprinted by permission of the ACI. As shown in Figure 5, the further we move to the right, the more economical the specimens become. When creep strain is plotted against paste content, creep is directly proportional to paste content. Similar data have been developed by the U.S. Army Office of Engineers. From these data, it appears that it is legitimate to test the inexpensive specimens and to compute the results for mass concrete by linear interpolation. This approach has proven useful, but if pushed too far it leads to an absurdity. It is observed, for example, that different aggregates produce different amounts of creep in concrete. However, if the approach outlined here is extrapolated to 100 percent paste, the anomaly shown in Figure 6 results—two creep values for the same paste. The answer, of course, is that the relationship cannot be linear.

The effect on creep of aggregate properties and concentration has been comprehensively investigated by Neville (8). This work showed that, except for cases in which the creep of the aggregate itself is a factor, which is usually not the case, the influence of the aggregate on creep is confined to the two elastic constants and to its concentration. Size and grading have an effect, but only as they affect concentration. The family of curves applicable when both the aggregate and cement paste have the same Poisson's ratio is shown in Figure 7. It can be seen that, when there is no great difference between paste E and aggregate E, the relationship is reasonably linear in the range of practical paste contents. More importantly, however, this relationship lays the foundation for a technique that deals separately with the effects of paste and aggregate.

If every set of materials that might be used in every set of environmental conditions that might be encountered were tested, the 1535 references in the literature would continue to expand in number exponentially. However, the constituent that contributes primarily to the needed multiplicity of data is the aggregate. Fortunately, the aggregate properties are independent of environmental conditions, and the pertinent properties can be evaluated in 1 day. Therefore, what is needed is a catalog of cement paste behavior, and this is where the researcher can be of greatest help to the designer. Paste tests can be conducted quite inexpensively. With a comprehensive data bank of creep of pastes of various water-cement ratios and various degrees of hydration exposed to various ambient conditions, a designer would merely need to request a quick aggregate test to produce a good first approximation of the volume change of the structure.

SUMMARY STATEMENTS

1. Let us avoid a complicated approach when a simple approach is applicable.
2. It is simpler and more accurate to design directly for a long-term deformation by use of a sustained modulus of elasticity than to compute an initial deformation and a creep function.
3. The most productive research would produce a catalog of cement paste behavior so that aggregate effects could be added by calculation.

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Durability of Steel-Formed and Sealed Bridge Decks

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A 2-year research project was carried out to investigate the freeze-thaw durability of concrete bridge decks cast on steel forms, which remain in place after construction, and sealed on the top surface with a waterproof membrane. The long-term durability of the forms was also studied. Laboratory freeze-thaw tests equivalent to a winter season and one winter of outdoor exposure tests were carried out on simulated bridge-deck slabs. These slabs covered all combinations of the following variables: (a) form type (wood and steel) and (b) surface treatment (none, linseed oil, and waterproof membrane). In addition, 25 bridge decks with steel forms and waterproof membranes and 1 bridge deck without steel forms but with a membrane were inspected. The bridge decks ranged in age from 1 to 13 years, and all but six were 8 or more years old. By using a variety of inspection techniques that ranged from visual examination to pulse velocity measurement, it was determined that steel-formed bridge decks with surface sealing are no more prone to freeze-thaw deterioration than wood-formed decks. Generally, the forms themselves were found to be in good condition when designed for proper deck-surface drainage.

The primary goals of the research described in this paper were to evaluate the effect of steel bridge forms on the durability of concrete bridge decks and to evaluate the durability of the forms themselves. The extensive use of steel beams for bridge decks, especially along the eastern seaboard, coupled with current Federal Highway Administration guidelines that require the use of waterproof membranes for resurfacing bridge decks provided the impetus for the first goal. The question of form deterioration due to corrosion has often been raised; however, few definitive data can be found in the literature. In a study of 20 steel-formed bridge decks in New York State (1, 2), it was reported that less than 5 percent of the total area of sections of steel deck forms removed for inspection showed any signs of corrosion. A 1970 survey of all 4-year-old bridge decks in Pennsylvania (249 bridges, 146 of which were constructed with steel deck forms), did not reveal any instances of form corrosion problems (3). However, the question of form durability persists.

The goals of this research were pursued in a research plan that included two major phases. The first phase involved exposing sections of simulated bridge decks to laboratory and field freeze-thaw conditions. The second phase consisted of a program for inspection of bridge decks.

SIMULATED BRIDGE DECK SPECIMENS

Experimental Design

A complete factorial experimental design was employed in which the variables consisted of three levels of surface treatment and two levels of form type. Each condition was replicated twice to enhance the statistical significance of the experimental results. Therefore, the total number of specimens was 12.

Test Specimens

The test slabs were 76.2 cm (30 in) wide by 91.4 cm (36 in) long by 19.1 cm (7.5 in) deep. (This is the depth to the top of the ridges on the steel forms, i.e., minimum slab thickness.) The side forms consisted of 5.1 by 25.4-cm (2 by 10-in) timbers coated on the inside with 1.59-mm (16-gauge) galvanized sheet metal. The side forms remained in place throughout the testing to prevent moisture loss through the sides of the specimens. Calking compound was applied to the perimeter of the slabs to seal any separation that occurred between the side forms and the slabs. The bottoms of the wood-formed slabs were formed with plywood that was removed after curing. The steel-formed slabs were formed on the bottom sides with standard 0.80-mm (22-gauge) galvanized bridge forms with a 16.5-cm (6.5-in) pitch and a 5.1-cm (2-in) corrugation depth.

Relative humidity wells, to accept the Monfore gauge (4), were precast in each specimen by using brass rods and sleeves. Steel reinforcing bars were placed in two layers; 1.3-cm (0.5-in) number 4 bars were placed transversely, and 1.6-cm (0.6-in) number 5 bars were placed longitudinally. The lower mat was positioned 2.5 cm (1 in) above the steel- or plywood-formed bottom, while the upper mat was placed 5.1 cm (2 in) below the top of the form. Steel chair supports were used for position-

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ing and supporting the reinforcing steel.

Copper-constantan thermocouples were placed in seven specimens. One thermocouple was placed at middepth in each of the two replicate specimens to represent each combination of design parameters. One slab received three thermocouples: one at 2.5 cm (1 in) below the surface, one at middepth, and one 16.5 cm (6.5 in) below the surface or 2.5 cm (1 in) above the bottom surface.

The concrete for all 12 test specimens was mixed in one batch at a commercial ready-mix plant and delivered by a transit mix truck to eliminate batch-to-batch variation. Pennsylvania Department of Transportation class AA concrete (the type used in bridge decks in Pennsylvania) was specified. Slump and air content were measured before the slabs were placed and at the third points on the placement schedule (i.e., between placement of the fourth and fifth and the eighth and ninth slabs). A final determination for air content was made after all slabs had been placed. The air content determinations were slightly below specification. For the purpose of this experiment, this was not necessarily detrimental since it served to accentuate any differences in freeze-thaw behavior among the specimens.

All specimens were moist cured with wet burlap for a 28-day period, and polyethylene sheeting was used to retard evaporation. After completion of the curing period, eight slabs were given surface treatments in accordance with the research plan. The surfaces of these slabs were allowed to dry in the laboratory for 1 day to enhance the surface treatments.

A 50-50 mixture of boiled linseed oil and mineral spirits was applied to four slabs: two wood-formed and two steel-formed slabs. Application was accomplished by a hand sprayer. The first coat was applied at a rate of 1.0 L/m^2 (0.025 gal/ft^2) while the second coat was applied at the reduced rate of 0.6 L/m^2 (0.015 gal/ft^2). One-day drying time was allowed between applications.

A preformed moisture-proof membrane was applied to two wood-formed and two steel-formed slabs. The slabs first received a coating of liquid mastic supplied by the manufacturer. The membrane was then applied after the tack coat had dried. A special caulk was then applied to all outside edges to ensure a watertight seal. A hot mix of asphaltic concrete (PennDOT Specification ID-2) that was 3.8 cm (1.5 in) thick was applied and compacted over the membrane. The remaining four slabs, two steel-formed and two wood-formed, received no surface treatment.

Laboratory Freeze-Thaw Tests

To accomplish the objective of this research, certain criteria had to be established. First, it was necessary to ensure that the test slabs were saturated at all times, thus providing the worst possible condition one would expect in nature. This was accomplished by continuously ponding water on the slab surfaces and by monitoring internal hygrometric conditions throughout the test. A Monfore gauge for relative humidity was used for monitoring purposes. Second, the internal freezing of each slab had to be ensured, and this was accomplished by monitoring internal temperatures of the slab by using thermocouples and a temperature recorder. Third, if there was any freeze damage, it had to be detected and monitored, and this was accomplished by the use of a pulse-velocity meter for detecting cracking and delamination and by the use of linear variable differential transformers (LVDTs) for volume changes during freezing.

Before the freeze-thaw process was begun, initial readings were taken with the Monfore gauge and with the pulse-velocity meter to establish data values. The specimens were subjected to 75 freeze-thaw cycles under controlled conditions in the laboratory. The winter climate of Pennsylvania was used as the basis for setting the number of freeze-thaw cycles, since this climate is considered severe in the freeze respect. The decision to use 75 freeze-thaw cycles was based on data from previous research (5) that showed this number of freeze-thaw cycles to be the maximum found in Pennsylvania in an average winter.

Temperatures within the slabs and the air temperature of the cooling chamber were constantly monitored. The cooling rate within the slabs was approximately 1.4° C/h (2.5° F/h). Once the slabs had reached -3.9° C (25° F), the freeze cycle was considered complete and was reversed. The heating rate within the slabs was 2.8° C/h (5° F/h), and thawing was considered complete when slab internal temperatures reached 1.7° C (35° F).

To verify that the test slabs remained in a saturated condition throughout the test period, internal relative humidity measurements were taken in each slab 2.5 cm (1 in) from the top, the bottom, and at middepth before freeze-thaw testing began, after the fifteenth freeze-thaw cycle, and every 10 cycles thereafter. According to the literature (6), relative humidity values in excess of 80 percent are indicative of saturation of the capillary system but not of the entrained air voids, which, because of their large size relative to the capillary pores, saturated with great difficulty. In other words, the gel and capillary pores hold water tenaciously and completely outstrip the larger air voids in competition for available water. When the gel and capillary pores are saturated (a feat they are able to accomplish in the presence of liquid water or by capillary condensation at relative humidities above 80 percent), the air voids are prevented from filling except under hydrostatic pressure (e.g., during freezing). Throughout the laboratory test phase, the relative humidities in the test wells were never below 95 percent, and most of the time they showed 100 percent relative humidity (RH) or the presence of free water.

Pulse-velocity measurements were obtained before the freeze-thaw cycles commenced, after the fifteenth freeze-thaw cycle, and every 10 cycles thereafter. These readings were taken to evaluate the extent of progressive freeze-thaw damage in the slabs. Pulse-velocity decreases with increasing cracking and deterioration in concrete because of the longer flight paths of the sonic waves passing around discontinuities when the distance between the transmitting and receiving transducers remains constant. Pulse-velocity readings were taken at five locations on each slab—near the corners and at the center. The results that include the subsequent field exposure tests are shown in Figure 1. It is quite evident that, with the possible exception of slab 9, there are no consistent trends in a degradation of pulse velocity. In slab 9, the steel bridge form had become debonded from the concrete, and this created an air space, which readily explains the drop in pulse velocity.

Length changes of the test slabs during freezing that were measured parallel to the thickness of the slabs were obtained by using the highly sensitive LVDTs. Equipment limitations permitted monitoring at only one point on each of six test slabs that represented each of the six combinations of test variables.

At the instant of freezing, all concrete containing moisture expands because of the hydraulic pressure generated by the freezing water. This expansion, termed "dilation," is transitory and completely recoverable in concrete that is immune to frost damage. In frost-susceptible concrete, however, dilations are large and mostly nonrecoverable; i.e., permanent deformation and cracking results. The critical value of dilation is a function of the elastic properties of the concrete (modulus of elasticity, tensile strength, and Poisson's ratio), but for normal concretes it is approximated by the relation

$$D_c = 70L \tag{1}$$

where

D_c = critical dilation (micrometers) and
L = dimension of concrete in the direction that length changes are measured (meters).

Therefore, for the case at hand, $D_c = (70)(0.19) = 13.3$ µm (525 µin). Thus, dilations in excess of 13.3 µm (525 µin) must be consistently encountered if the concrete has failed because of freeze-thaw action. Larson and Cady (7) give a more detailed discussion of dilation as a measure of frost damage. Length change measurements were made on some or all of the instrumented slabs during the cycles given in Table 1. Considerable difficulty was encountered because of condensation and subsequent freezing of moisture in the cores of the LVDTs. Despite the efforts to circumvent this problem, the data obtained are somewhat meager. As given in Table 1, the dilation for any case did not exceed or even closely approach the critical value of 13.3 µm (525 µin). These data support the findings of the pulse-velocity determinations and confirm the absence of frost damage to the test slabs.

Field Tests for Exposure

After the 12 slabs were tested by simulating a winter season of freeze-thaw cycles in the laboratory, they were moved out-of-doors for field exposure tests. The major objective of this phase of the project was to monitor the performance of the slabs during one winter of actual field exposure. The information gathered during the 1974 to 1975 winter included weather data (daily high-low temperature, precipitation, and depth of snowfall), moisture content, pulse velocity, and visual inspection. Deicer salt (calcium chloride) was applied to the slabs at a rate of 141 kg/lane-km (500 lb/lane-mile) or about 0.039 kg/m^2 (0.008 lb/ft^2) as dictated by weather conditions. In all, 14 salt applications were made.

The site chosen for outdoor exposure of the test slabs is the oval-shaped, 1524-m (5000-ft) long, paved test-track facility that is situated approximately 8 km (5 miles) northeast of the main campus of the Pennsylvania State University in open, rolling country. A reinforced concrete slab was constructed at the site, and the test slabs were supported about 0.4 m (16 in) above the slab on concrete blocks.

From daily temperature data, it was determined that 39 freeze-thaw cycles occurred during the period from August 9, 1974, to March 14, 1975. This number of cycles appeared to be somewhat lower than the 57 freeze-thaw cycles that were determined to occur in an average winter in this part of Pennsylvania. However, it is reasonable to assume that 5 to 10 additional freeze-thaw cycles occurred during the remainder of the 1974 to 1975 winter after March 14. A freeze-thaw cycle is defined by the dropping of air temperature below -3.3° C (26° F) followed by a rise in temperature above 1.1° C (34° F). Total precipitation was about 15 percent above normal, and snowfall amounts were about 33 percent above normal during the test period.

It was originally intended that the degree of saturation within the test slabs would be monitored weekly during the winter season by using the Monfore relative humidity probe. However, some difficulties were encountered with the probe; e.g., probe test wells were plugged with insects, and a probe was disabled. Therefore, an effective acquisition of internal RH data was prevented until January. Nonetheless, all data obtained, as in the laboratory test phase, revealed internal relative humidities in excess of the critical 80 percent level necessary for the maintenance of capillary saturation.

Periodic pulse-velocity determinations were made on the test slabs to determine presence and extent of any freeze-thaw damage to the slabs during the exposure period. As shown in Figure 1 and mentioned earlier, the pulse-velocity determinations indicated no evidence of freeze-thaw damage.

BRIDGE INSPECTIONS

The laboratory and field exposure tests previously described were carried out to evaluate potential freeze-thaw damage on simulated bridge-deck slabs under controlled conditions. To provide the added dimensions of traffic loading and longer periods of exposure, a program of field inspection of bridges having steel forms and membrane-sealed deck surfaces was undertaken.

It was decided that the oldest bridges having the same (within the limits of practicality) superstructure type and traffic loadings would be selected from diverse geographic locations. Also, the ages of the bridges should ideally be about the same. Unfortunately, for reasons that will be discussed, it was necessary to compromise some of these initial selection criteria.

Selection of Inspection Sites

Although no problem existed in finding sufficient numbers of bridges having steel bridge forms, the extent of the use of membrane waterproofing systems in conjunction with steel bridge forms was not known. Also, although membranes applied as liquids (polymeric resins and epoxy modified bitumens) have been in use for some time, the preformed materials for sheet membranes are fairly recent newcomers to the bridge-deck construction scene. The latter class of membrane materials is currently recommended by the Federal Highway Administration for waterproofing bridge decks. Therefore, as a starting point, the three leading manufacturers of preformed membrane materials were contacted to ascertain which highway agencies were using the membranes. The contractor on NCHRP project 12-11, Waterproof Membranes for Protection of Concrete Bridge Decks, was also contacted to obtain information on potential bridge inspection sites. A consulting engineering firm that pioneered in work in this field was also contacted. As a result of these initial inquiries, five state highway agencies (Massachusetts, New York, Pennsylvania, Vermont, and Virginia) and the New Jersey Turnpike Authority were contacted, and two facts became immediately evident. First, bridges that were sufficiently advanced in age (at least 5 to 10 years) to provide meaningful results had membranes that were not of the preformed type but that had been applied as liquids. Second, most highway agencies that have extensively used steel bridge forms for long periods of time have not used membrane sealing systems (e.g., Pennsylvania), and most highway agencies that have used the membranes have not extensively used steel bridge forms (e.g., Vermont). In the final analysis, sufficient numbers of bridge decks having both the steel forms and waterproof membranes to warrant further consideration were found to exist only in New York State and on the New Jersey Turnpike.

New York State Bridges

The New York State Department of Transportation was

Table 1. Dilations during freezing.

Slab			Dilation (μm) by Cycle												
No.	Form	Treatment	17	21	22	23	24	25	26	29	38	45	51	53	68
2	Wood	None	—[a]	2.5	1.5	—[a]	3.6	3.8	5.8	—[a]	5.1	—[b]	—[b]	—[b]	2.8
3	Wood	Membrane and asphaltic concrete	—[a]	7.6	4.1	5.6	4.1	9.1	5.6	9.1	7.4	—[b]	—[b]	—[b]	5.1
5	Steel	None	8.1	—[a]	—[a]	6.1	1.5	3.3	4.1	—[a]	—[a]	—[b]	—[b]	—[b]	3.0
6	Wood	Linseed oil	—[a]	2.0	1.5	1.5	1.5		—[a]	—[a]	4.3	4.1	9.1	8.6	9.4
7	Steel	Linseed oil	11.2	2.0	4.1	—[a]	—[a]	—[a]	4.6	6.1	—[a]	—[b]	—[b]	—[b]	—[a]
9	Steel	Membrane and asphaltic concrete	12.2	4.1	—[a]	—[a]	1.8	—[a]	—[a]	5.3	—[a]	—[b]	—[b]	—[b]	—[a]

Note: 1 μm = 0.039 μin.

[a]Transducer frozen. [b]Not tested.

Figure 1. Effect of freeze-thaw cycles on pulse velocity for test slabs.

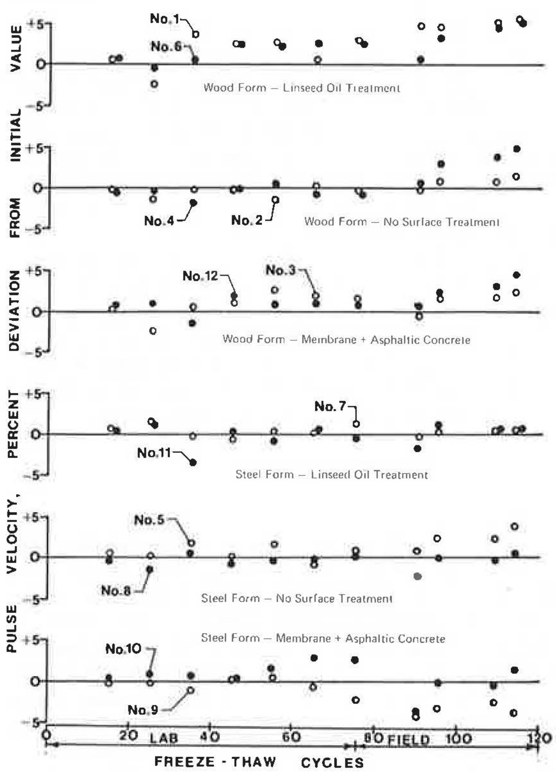
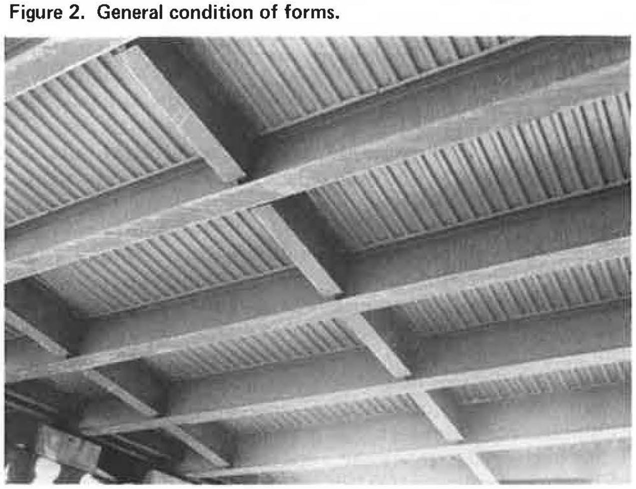


Figure 2. General condition of forms.



able to provide a listing of 34 bridges that had been constructed with the use of steel deck forms and waterproof membranes. The bridges were built during the 1961 to 1963 period and had either polyester resin or epoxy modified coal tar for membranes and were topped with an asphaltic concrete wearing course. Twelve of these bridges, widely distributed across the state, were selected for inspection. Also, one bridge built during the same period and having the membrane system, but constructed with wood forms, was chosen for purposes of comparison.

New Jersey Turnpike

The selection of bridges on the New Jersey Turnpike was more complicated than the selection of bridges in New York State. Three distinct categories of structures existed on the New Jersey Turnpike:

1. Type A—replacements of deteriorated sections of existing decks using steel forms, liquid membranes, and asphaltic concrete wearing courses (this replacement started in 1963);
2. Type B—new construction of decks by using steel forms, liquid membranes, and asphaltic concrete wearing courses (this construction was from 1963 to 1972); and
3. Type C—new construction of decks by using steel forms, preformed membranes, and asphaltic concrete wearing courses (this construction started in 1972).

The type A and type B systems are basically the same as those used in New York State. The difference between A and B is simply that A involved replacement sections of existing bridges and B was new construction. To account for the age factor, the two oldest bridges of type B (built in 1963 and 1966) and five oldest replacement sections of type A (done in 1965) were selected.

Since all of the bridges selected to this point had liquid membrane systems (polyester resin or epoxy modified coal tar), it was felt that a representative sample of steel-formed decks having preformed membrane systems should be selected for inspection in spite of the fact that they would have to be fairly new bridges. Accordingly, six type C bridges were selected. All but two of the bridges selected are simply supported, steel I-beam structures. The other two, both in New Jersey, are continuous plate girders. The bridges ranged in age from 1 to 13 years, and all but six were 8 or more years old.

Inspection Procedures

The rationale behind the inspection procedures adopted for this study was based on the premise that general deck conditions could be reliably assessed by means of a few simple observations. These inspections were not intended to be detailed condition surveys like those that might be conducted by a highway agency before making repairs. Rather, the purposes were

1. To determine if the presence of steel bridge forms in combination with waterproof membranes has any effect on deck durability, especially freeze-thaw susceptibility; and
2. To assess the long-term durability of steel bridge forms.

Except for a few locations on the New Jersey Turnpike, all close inspections of the undersides of the bridges were limited to those areas that could be conveniently reached, i.e., near the abutments. The areas that could not be reached for close inspection were carefully scanned with the use of 7 by 50 binoculars. To the extent that it was practical to do so, the following inspection procedures were carried out on each bridge:

1. Visual examination of the deck riding surface by noting general conditions and presence of cracking, potholing (including incipient failures), and recent patching or repairs;
2. Visual examination of the underside of the deck by noting the general condition of the steel forms and the location and severity of corrosion and its relationship to details of the deck design;
3. Hammer sounding of accessible regions on the underside of the deck to determine the percentage of area that indicated separation (debonding) of the forms from the concrete and insufficient filling or consolidation of the concrete in the valley (flute) portions of the forms; and
4. Pulse-velocity determinations by using the James V-scope to assess the soundness of the decks.

It was not possible to perform procedures 1 and 4 on many of the New Jersey Turnpike bridges because of the unavailability of traffic control that was a prerequisite for topside deck inspection. In addition to the four procedures described above, form panels were removed from five of the New York State bridges and seven of the New Jersey Turnpike bridges. The insides of the forms were examined for evidence of corrosion, and the concrete areas exposed by removal of the forms were closely studied for evidence of freeze-thaw deterioration, insufficient consolidation or segregation, and the presence of foreign materials.

Inspection Results

Deck Surface

In general, the decks were in excellent condition, especially in view of the ages involved. However, because of the inability to obtain maintenance histories of the deck surfaces, observation of the current condition of individual decks cannot, by itself, be considered a valid assessment of the condition of the concrete structural deck. It is known that some of the decks were overlaid with asphaltic concrete subsequent to initial construction. Nonetheless, the general overall lack of potholes, recently repaired areas, or other signs of distress tends to indicate satisfactory performance.

Form Condition

Overall, the forms were in excellent condition, even on the oldest (13-year-old) bridges. Figure 2 illustrates this fact and shows a general view of one of the bridges that exhibited the most severe and extensive corrosion problems of the bridges inspected. Where corrosion was found to exist, it could almost invariably be traced to the drainage of salt-laden waters from the deck surface. Consequently, most of the corrosion observed was found to exist along the facia girders and at the span ends, as shown in Figure 3. A particularly striking example of the effect of deck drainage on form corrosion was found on the four bridges inspected in the Binghamton area in which grating type of drainage features permitted ready access to the forms by water running off the decks, as shown in Figure 4.

Corrosion on the inside of removed sound-form panels was virtually nonexistent. Two of the form panels that were removed displayed a few spots of light rusting, and a third showed evidence of slight corrosion on an area in which sawdust and wood chips had been inadvertently left when the concrete was placed.

Hammer Soundings

A positive correlation, significant at the 95 percent level, was found to exist between percentage of hollow soundings and deck age. This would tend to indicate that the development of hollow areas is progressive with time and, therefore, results from separation of the form from the concrete rather than from the incomplete consolidation of the concrete.

Observation of the concrete that was exposed by the removal of selected form panels revealed 1 area in 12 (about 8 percent) that showed the existence of incompletely consolidated concrete. Since this value exceeds the 6 percent overall average of hollow-sounding areas, it fails to substantiate the debonding theory. However, because the sample size is so small it also does not refute it.

Pulse Velocity

In general, the pulse-velocity values for the outdoor test slabs are lower than those observed for the laboratory test slabs. The four test slabs that had membranes and asphaltic concrete wearing courses had initial pulse-velocity values that averaged 4026 m/s (13 209 ft/s) in comparison with the pulse-velocity values with an overall average of 3529 m/s (11 578 ft/s) for the bridge decks. One reason for this disparity is that the thickness values assumed for the bridge decks and used for the pulse-velocity calculations were quite likely on the low side. The thicknesses used were design values. Also, it is quite likely that overlays were applied to the decks subsequent to construction, and this would add to the deck thickness in some cases. However, this factor was taken into account when known. Another factor considered was that most of the overlay thicknesses used on the bridge decks were much greater than those used on the test slabs, and the overlay layer does modify the pulse velocity. For example, the average initial pulse velocity for the test slabs that had the overlay was over 305 m/s (1000 ft/s) lower than the average for the test slabs that had no overlay. Finally, lower pulse-velocity readings may not necessarily indicate concrete damage—they could also result from form or membrane debonding.

In view of the factors cited above and the fact that the pulse velocity will vary because of differences in the water to cement ratio, the aggregate type, entrained air content, and degree of water saturation of the concrete, it is not feasible to establish a threshold value below which concrete deterioration may be positively assumed to have occurred. However, concrete that is of sound quality should, theoretically, exhibit a pulse-velocity value of 3292 m/s (10 800 ft/s) or greater. Furthermore,

Figure 3. Corrosion of forms near span end and along facia girder.

Figure 4. Corrosion of forms in vicinity of grating type of drainage feature.

Figure 5. Condition of concrete in form removal area.

Figure 6. Sand embedded in concrete surface in form removal area.

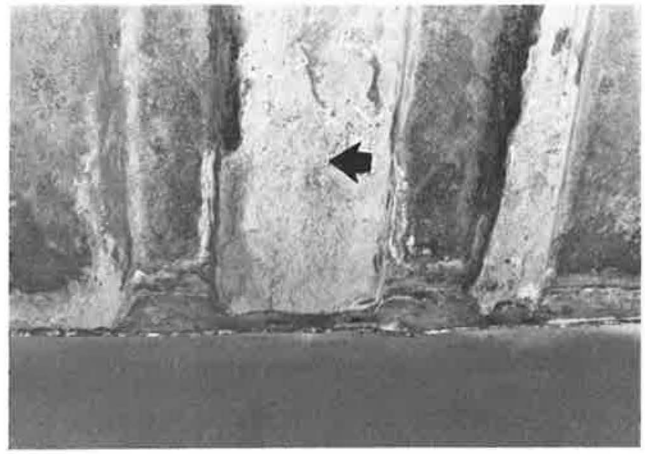


Figure 7. Corroded form and deteriorated concrete at location of foreign material in deck.

on a given bridge deck, large differences in pulse velocities taken at different points should be viewed as an indication that some areas of concrete deterioration may exist. From the use of these guidelines, it appears that the two bridge decks in New York State may contain deteriorated concrete. One of the decks does not have steel bridge forms. It appears that the south end of this deck may be experiencing deterioration of the concrete. However, the north end of this deck gives highly satisfactory readings; therefore, one would judge that the cause of the problem might be corrosion of the reinforcing bar and subsequent delamination rather than freeze-thaw damage, which would tend to be more general. The other suspect deck displays consistently low readings that might indicate freeze-thaw damage. The only way to be certain in either of these cases would be to extract cores for examination. Although it cannot be stated positively that the remaining bridge decks did not suffer freeze-thaw damage, the pulse-velocity values for these bridge decks did not indicate freeze-thaw damage.

Condition of Concrete Exposed by Form Removal

The concrete exposed by form removal was found to be in sound and excellent shape, as shown in Figure 5. Only one instance of incomplete consolidation of the concrete in the valley portion of the form was observed. In two cases, foreign materials left on the forms at the time of concrete placement were found embedded in the concrete, as shown in Figure 6. An unusual instance of form corrosion and concrete deterioration due to a block of wood left on the form is shown in Figure 7.

CONCLUSIONS

Based on the research described in this paper, the following conclusions appear to be warranted:

1. Steel-formed bridge decks, with or without surface sealing, up to 13 years of age are no more prone to freeze-thaw deterioration than wood-formed decks or steel-formed decks without membranes;
2. In carefully controlled laboratory freeze-thaw tests, steel-formed decks and wood-formed decks behaved no differently;
3. Although separation of the steel forms from the concrete occurs in some cases, it apparently has no effect on the durability of either the deck or the forms; and
4. Form corrosion is generally not a problem if the deck design provides proper drainage from the deck surface to prevent contact with the forms.

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Corrosion-Inhibiting Properties of Portland and Portland-Pozzolan Cement Concretes

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Within the framework of studying properties that inhibit corrosion of concrete that contains admixtures and additives, concretes with various amounts of fly ash were tested and evaluated. Cylindrical specimens with a centrally located reinforcing bar were partially immersed in a 5 percent sodium chloride solution after various curing periods and curing procedures. An impressed-current test was used to accelerate the corrosion process. Time to cracking and current flow were measured, and total energy requirements to induce failure calculated. Other variables studied included sorption, resistivity, strength, consistency, and time to cracking failure of the specimens. Test results indicated that the use of pozzolanic material (fly ash) as an additive or an admixture improves the property of the concrete to prevent corrosion of the embedded reinforcing steel.

Large sums of money are expended anually to repair and mitigate the corrosion damage to concrete bridges and other transportation structures. In many cases, the benefits derived from such repairs may be questionable since some repairs are only for maintaining the appearance of a structure and actually intensify the corrosion process.

To serve the growing population centers along the seashores, many structures have been placed in corrosive environments. Use of concrete in such structures has increased over the last 20 years because of the large-scale use of prestressed concrete. Over this same period, changes in fabrication techniques and the quality of concrete may have influenced the corrosion problem. Coincidental with these factors has been the increased use of deicing salt on concrete bridge decks and pavements, which results in corrosion.

The research needs associated with corrosion in reinforced concrete structures may conveniently be divided into four major areas:

1. Determination of intensity, rate, and extent of corrosion in an existing structure;
2. Establishment of limiting specification values based on field corrosion tests that will assist in deciding the extent of maintenance action, i.e., whether routine repair, major replacement, or any other action is necessary;
3. Criteria for suitable repair materials and procedures; and
4. Designs for preventing corrosion of new structures.

This presentation is mainly concerned with the fourth area, but may also have application to the other areas.

Basically, there are three protective measures to mitigate the corrosion of reinforcing steel:

1. An appropriate insulator around the reinforcing steel;
2. A protective barrier applied to the concrete surface; and
3. Modification of the concrete.

This presentation discusses the last solution: the modification of concrete to achieve better protection of reinforcing steel from corrosion. A variety of admixtures and additives has been used or suggested for use in modifying concrete to inhibit corrosion of the reinforcing steel (1, 2, 3). Some measures have been used to maintain the passivity of the steel by retaining a high pH-value in the concretes.

Corrosion of major concern in reinforced concrete is of two types: galvanic, which is related to variability in characteristics of the reinforcing steel, and concentration, which is related to environmental changes in the concrete. For the activation of either type, anodes and cathodes must be present, and direct current (dc) must flow. Conducive to the electrochemical process of corrosion in reinforced concrete is the presence of water, carbon dioxide, oxygen, and chloride salt. The water acts as a carrier for salt, oxygen, and carbon dioxide. Water containing dissolved salts is an electrolyte with low electrical resistivity that favors the passage of the corrosion current.

The more permeable the concrete is, the more readily the water can move; thus the probability of corrosion increases. Therefore, an important requirement

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for protecting concrete from corrosion is a low permeability. It has been suggested (4) that the permeability of concrete is a characteristic property that assists in preventing the corrosion of embedded steel. Therefore, the use of concrete with low permeability may be a partial solution to the problem of hindering the development of corrosion in reinforced concrete.

To take remedial action after corrosion has occurred in a structure is a complicated process. Indeed, in some cases, the remedial action may intensify and promote further corrosion. Therefore, it is essential that the protective properties be built into the structure at the time of fabrication and construction through the selection of appropriate materials and design.

The primary objective of this preliminary investigation was to compare the properties that inhibit corrosion in concrete made of portland cement and with or without pozzolanic admixtures. A further objective was to determine the relative impact that curing procedures and cement content would have on inhibiting the corrosion of steel in the concrete.

A review of the literature has revealed that researchers have previously defined those variables used in mixtures of concrete that influence the protection of embedded steel against corrosion. Based on this past research, the following variables were chosen for this project:

1. Curing method and period,
2. Cement content,
3. Water to cement ratio, and
4. Cement type and pozzolanic admixtures.

In this investigation, a Florida limestone that is porous and perhaps more permeable than rock types found elsewhere in the country was used exclusively. The corrosion test procedure made use of an impressed electrical current to accelerate the corrosion process.

EXPERIMENTAL VARIABLES AND DESIGN

The independent variables are those selected in the experimental design, and the dependent variables are those measured or observed during the course of the investigation. In the analysis of data, the two sets of variables are examined to detect a possible cause-effect relationship. The independent variables selected in this investigation are shown in Figure 1. Some of the variables were combined into a factorial experiment, and other variables were studied and related to the main factorial experiment.

Cement

Studies by others (2, 3) have indicated that portland-pozzolan cement and portland cement with fly ash as a partial cement replacement may afford better protection against corrosion of reinforcing steel than a corresponding mix of only portland cement. In this investigation, commerical type I and type IP cements were used, and the corrosion protective properties of cement-pozzolan combinations were evaluated by using a single mix design.

The amount of cement has been shown to have an appreciable influence on inhibiting corrosion in concrete. In this experiment, cement contents of 334, 418, and 501 kg/m^3 of concrete (6, 7½, and 9 bags/yd^3) were used in the design of the respective mixes. In other studies (3, 5), the water to cement (w/c) ratio has been established as having a significant effect on the quality of concrete. A wide range of w/c ratios was impractical for the cement content and aggregate proportions used in this experiment. It was decided to compare the various mixes on a consistency basis that was determined by AASHTO T-119—slump tests for plastic concrete. Instead of comparison by w/c ratio, three slump values, 5.08, 10.16, and 15.24 cm (2, 4, and 6 in), were designated as the test parameters in the factorial design.

Fly Ash Admixture

Fly ash that replaced an equal amount of cement in percentages of 20 (type IP), 35, and 50 was used as an admixture.

Curing

The effect of curing procedure on permeability of concrete and its corrosion-inhibiting properties is significant and widely documented (3). One objective of this research, mentioned previously, was to determine the relative effectiveness of a variety of curing procedures by considering both curing method and curing time; therefore, six different procedures were used. These procedures along with their respective designations are as follows:

Designation	Curing Procedure
A	Stripped from mold in 1 d and moist-cured at 25°C (77°F) and 97 percent relative humidity (RH) for 6 d
B	Stripped from mold in 1 d and moist-cured at 25°C (77°F) and 97 percent RH for 27 d
C	Stripped from mold in 1 d and water-cured by submergence for 6 d at 25°C (77°F)
D	Stripped from mold in 1 d and water-cured by submergence for 27 d at 25°C (77°F)
E	Stripped from mold in 1 d and cured no further
ES	Steam-cured in mold at approximately 65.6°C (150°F) for 16 h and cured no further

Aggregates

Florida crushed limestone with a fineness modulus of 5.97, absorption capacity of 31 percent, specific gravity of 2.51, and Los Angeles abrasion coefficient of 35 was used as the coarse aggregate. The silica sand used as the fine aggregate had a fineness modulus of 2.19, specific gravity of 2.63, color number of 20, and loss on decantation of 1 percent. All aggregate properties were determined in accordance with the appropriate AASHTO procedures.

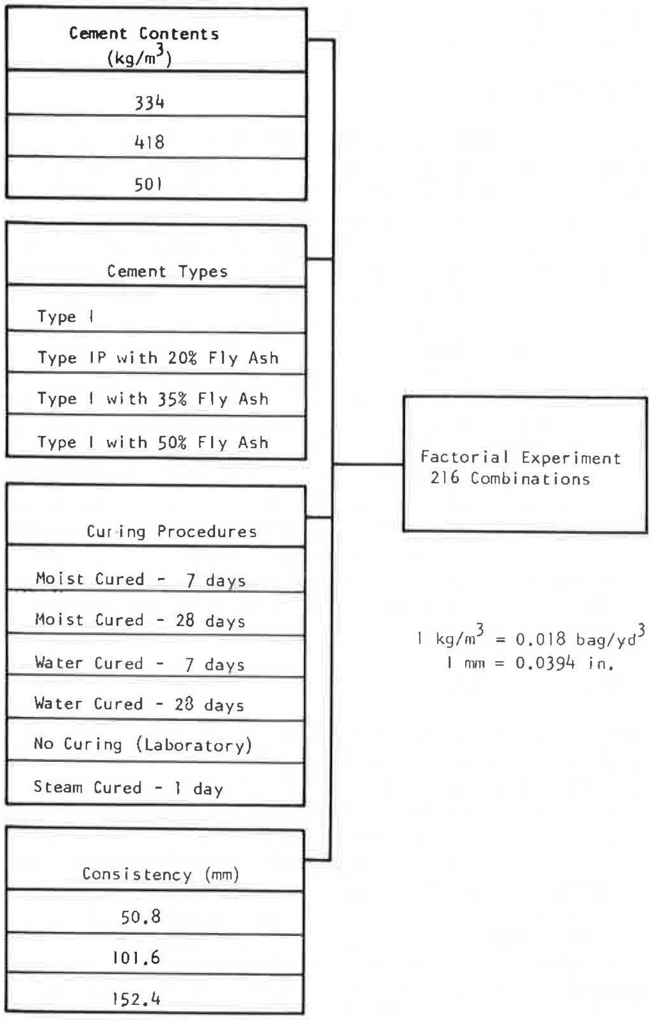
Batching of Concrete

The factorial design for the experiment was developed for cement content, cement type, curing procedure, and consistency. A single batch of concrete was made for each design mix given in Table 1. The concrete was mixed in a 0.071-m^3 (2.5-ft^3) laboratory mixer. The mixer was buttered before onset of batching operations. The batches were charged and then mixed for a minimum of 2 min.

Corrosion Specimens

Eighteen cylindrical corrosion test specimens, all 10.16 cm (4 in) in diameter by 12.70 cm (5 in) in height, were molded from each batch. The procedure used was to place and rod each specimen in a manner similar to that used for making concrete specimens of standard strength. After the cylinder was rodded, a 1.27-cm (0.5-in) diameter reinforcing bar was placed vertically in the cylinder along the centerline. The lower end of the bar was approximately 5.08 cm (2 in) from the bottom surfaces.

Figure 1. Experimental design.



In some cases, it was determined that the bar had settled to give less than the required cover at the bottom surface. In such instances, an epoxy was applied to the cylinder bottom.

Measured Variables

The measured variables for this research included slump, entrained air, strength, sorption, relative resistivity, and time to corrosion failure.

1. The slump test (AASHTO T-119) may not directly relate to consistency; however, it does give an indication of workability and may be easily related to current specification requirements. Slump tests were made on each batch of concrete.
2. Tests for entrained air were made on each batch of concrete in accordance with AASHTO T-196 by using a roll-a-meter. This method has been found to be preferable to the pressure method when aggregates have high porosity.
3. Since compressive strength and strength gain characteristics must be considered when pozzolanic material is used as a replacement for cement, strength specimens were made and tested in accordance with appropriate AASHTO specifications for the design mixes used. These values are shown in Figure 2.
4. Based on tests conducted in California and Texas (3, 4), it has been suggested that the basic mode of water movement through concrete is by capillary action. As such, sorption measurements were used to determine the relative permeabilities or water penetration of the design mixes used. After curing, the specimens were dried for 100 h at 110°C (230°F). The cylindrical wall was coated twice, once upon cooling and once 24 h later, and permitted to dry for 24 h. The specimens were then soaked vertically, and a 15.24-cm (6-in) hydrostatic head acted on the bottom face for 24 h. The difference between the wet and dry mass was determined, and the amount of moisture penetration was measured after the cylinders had been split open. Observed values and the split tensile strengths are given in Table 2.

Table 1. Experimental design for mixture of concrete.

	Mix Proportions							
Batch No.	Fly Ash[a] (%)	Cement (kg)	Coarse Aggregate (kg)	Fine Aggregate (kg)	Water (kg)	Slump (mm)	Air (%)	Cement Content (kg/m³)
1	0	42.68	138.5	87.6	20.1	50.8	6	334.2
2	0	42.68	137.1	86.3	21.3	101.6	5	334.2
3	0	42.68	135.3	84.0	22.6	152.4	6	334.2
4	0	42.68	101.7	64.9	18.8	50.8	4	417.8
5	0	42.68	100.3	63.6	20.1	152.4	4	417.8
6	0	42.68	104.4	61.3	19.2	76.2	4	417.8
7	0	42.68	75.8	48.6	18.8	152.4	2	501.3
8	0	42.68	77.6	49.0	18.3	101.6	4	501.3
9	0	42.68	77.6	49.5	17.9	50.8	2	501.3
10	20	42.68	148.9	79.9	18.3	50.8	4	334.2
11	20	42.68	146.6	77.2	20.5	101.6	3	334.2
12	20	42.68	136.2	82.2	22.6	152.4	4	334.2
13	20	42.68	102.6	62.7	18.8	50.8	3	417.8
14	20	42.68	104.4	59.5	19.2	101.6	4	417.8
15	20	42.68	79.5	46.3	17.9	50.8	4	501.3
16	20	42.68	79.0	46.3	18.3	101.6	4	501.3
17	20	42.68	77.6	45.6	18.8	152.4	2	501.3
18	20	42.68	102.6	59.5	20.1	152.4	4	417.8
19	33	27.69	137.1	84.0	21.2	127.0	6	334.2
20	33	27.69	104.4	59.0	19.3	101.6	4	417.8
21	33	27.69	75.8	46.3	18.9	76.2	3	501.3
22	47	21.34	137.1	81.3	21.2	101.6	5	334.2
23	47	21.34	104.4	56.8	19.3	101.6	4	417.8
24	47	21.34	75.8	44.1	18.9	127.0	5	501.3

Note: 1 kg = 2.2 lb; 1 mm = 0.0394 in; 1 kg/m³ = 0.018 bags/yd³.

[a] Batches 10 through 18 used type 1P cement containing approximately 20 percent fly ash as an additive. Batches 19 through 24 contained fly ash as an admixture.

5. The approach in the corrosion tests was to expose all specimens to the same uniform environment and to impress current in the reinforcing rod to simulate an accelerated corrosion process. During the accelerated testing, specimens were placed in a tank and immersed to three-quarters of their length in a 5 percent by mass aqueous solution of sodium chloride (NaCl), as shown in Figure 3. The tanks were lined with fiberglass and accommodated 84 specimens each. The specimens were equidistantly spaced in the tanks.

The setup for the accelerated test is shown in Figure 4. The impressed-current apparatus consisted of an air-cooled selenium rectifier, insulated wiring that connected each test specimen, and a cathodic steel bar for current return. In addition to this apparatus, shunts were used for monitoring the current impressed on each specimen. Test equipment and instrumentation used for monitoring of the current included a sensitive multimeter, shunts, and an ac electrical test meter.

TEST PROCEDURES

After fabrication and curing, the specimens were conditioned in the sodium chloride solution for a minimum of 28 d. This conditioning reduced the effect of the experimental variables for curing age and procedure. After the conditioning process, the specimens were subjected to current, as described in the following section.

Test Principles

It is recognized that reinforcing steel encased in concrete will corrode if an electrical current is present (6). Such a current may be initiated by a potential difference between two different areas of the metal. Potential difference (voltage) may be due to the nonhomogeneity of the steel, in which case the resultant corrosion is termed galvanic, or due to a difference in the environment of the metal, in which case corrosion is termed concentration. Introduction of chlorides into the concrete lowers the effective resistivity at the steel-concrete interface. Because high pH is the primary concrete property that inhibits corrosion and because the protection of steel is directly proportional to the corrosive resistance of concrete, the problem with chloride presence cannot be overstated.

The electrical current was impressed to the reinforcing bar (anode) in each specimen by using a rectifier. The current passed from the rod, through the concrete, into the water, and to a steel return rod (cathode). The use of the electrical current simulated and accelerated a natural corrosion process. As the steel corrodes, the product of the corrosion has a volume that is approximately twice the original volume of the material. Consequently, these corrosion products induce tensile stresses in the surrounding concrete and eventually result in a cracking and spalling of the concrete.

Procedures for testing remained uniform throughout the experiment. The saltwater bath (electrolyte) surrounding the test specimens was kept at a 5 percent salt concentration by mass. The water temperature varied several degrees, 21.1 to 22.8°C (37.8 to 41.0°F), but remained uniform throughout all test tanks.

Test Instrumentation

During the tests, electrical test meters monitored the alternating and direct current voltages. The latter voltages ranged from 0.1 mV to 100 V. Voltage measurements were taken across shunts to determine rectifier output and current flow to each cylinder.

Cathodic-protection rectifiers were bridged, selenium, air-cooled units with an adjustable output up to 20 V and 5 A. Connections to test cylinder reinforcing rods were made with high-pressure battery clips to ensure good conductivity.

Test Procedure

Saltwater, 5 percent sodium chloride by mass, was added to the fiberglass test tanks and was brought to a level approximately 1.91 cm (0.75 in) below the top of the concrete surface of the specimens. Electrical connections between the rectifier and specimens were then made, as shown in Figure 5. The system was energized, and the current was adjusted so that ac current of approximately 25 mA flowed to each specimen. Currents to individual specimens were monitored twice each day by measuring the voltage drop across the shunts that were installed in the respective connecting wires. Visual observations were also made twice daily to detect specimen failure. The system remained energized and adjusted without interruption throughout the testing period.

Other parameters monitored included water temperature, water salinity, and overall voltage readings at the rectifier terminals and on the specimen steel. Voltage potential readings were initially taken by using a copper sulfate half-cell but were found to add an insignificant amount of information.

Criteria for Failure

Before the initiation of the tests reported here, preliminary tests were made to determine the range in test parameters and the influence of various test methods. It was concluded that the optimum amount of dc current required to accelerate the corrosion process was approximately 25 mA. This level of current could be measured by the equipment used for monitoring and accelerated the corrosion to a desired period of time.

One criterion for specimen failure was established during the preliminary testing: Failure occurs when cracks become visible on the surface of the concrete. A before-and-after view of a failed specimen is shown in Figure 6. In such preliminary tests, the cylinders were not disconnected from the system when the initial cracks appeared, and, consequently, the cracks continued to widen until the cylinder eventually fell apart. Since the widening allowed the corrosion product to pollute the saltwater and provided no additional useful data, subsequent tests were terminated at the sight of the first crack.

A second criterion for failure was defined during the data reduction process. This criterion was based on the observance that all specimens markedly decreased in resistance, i.e., increased in current consumption, before a cracking failure. It was found that the energy applied to this point was related to the time required for visual cracking to occur.

DATA ANALYSIS AND PRESENTATION

In the first phase of data analysis, plots of current and voltage versus the time function were made. The trends established by analyzing these graphs led to the following findings.

In the initial analysis, the time of failure was when the visual crack was seen. However, after reviewing the data, such as those shown in Figure 7, it was obvious that failure was progressive and that the time to first visual crack did not correspond to the time of initial decrease in the electrical resistance of a specimen. Furthermore, it was reasonable to assume that the strength of the concrete had some effect on the time

Table 2. Permeability values for concrete.

Cement Type	Age (d)	Dry Mass (kg)	Wet Mass (kg)	Water Mass (kg)	Load[a] (kg)	Penetration (mm)
1	14	11.59	11.75	0.16	7 710	63.5 ± 3.2
1	14	11.58	11.70	0.12	9 760	63.5 ± 3.2
1P	14	11.63	11.74	0.11	11 580	57.2 ± 3.2
1P	14	11.65	11.74	0.09	12 030	50.8 ± 6.4
1	28	11.85	11.94	0.09	12 140	54.0 + 3.2
1	28	11.87	11.97	0.10	13 620	57.2 + 3.2
1P	28	11.90	12.00	0.10	15 660	50.8 + 6.4
1P	28	11.98	12.06	0.08	15 430	50.8 ± 3.2

Note: 1 kg = 2.2 lb; 1 mm = 0.0394 in.
[a]Loads shown are for testing in split tension.

required for the appearance of visual cracking.

Shown in Figure 7 is the idealized plot of current flow versus time. The current flow remained approximately constant over the initial period of testing. Then a marked increase occurred in demand of current flow to the specimen. This sharp increase was designated as the time of failure. After the failure criterion was exceeded, a significant increase in current flow took place and was followed by a period of alternate decreases and increases in current flow and then by visual cracking of the specimen.

The sharp increase in current flow immediately after the failure criterion is exceeded is the result of a marked decrease in the electrical resistance of the specimen due

Figure 2. Compressive strength.

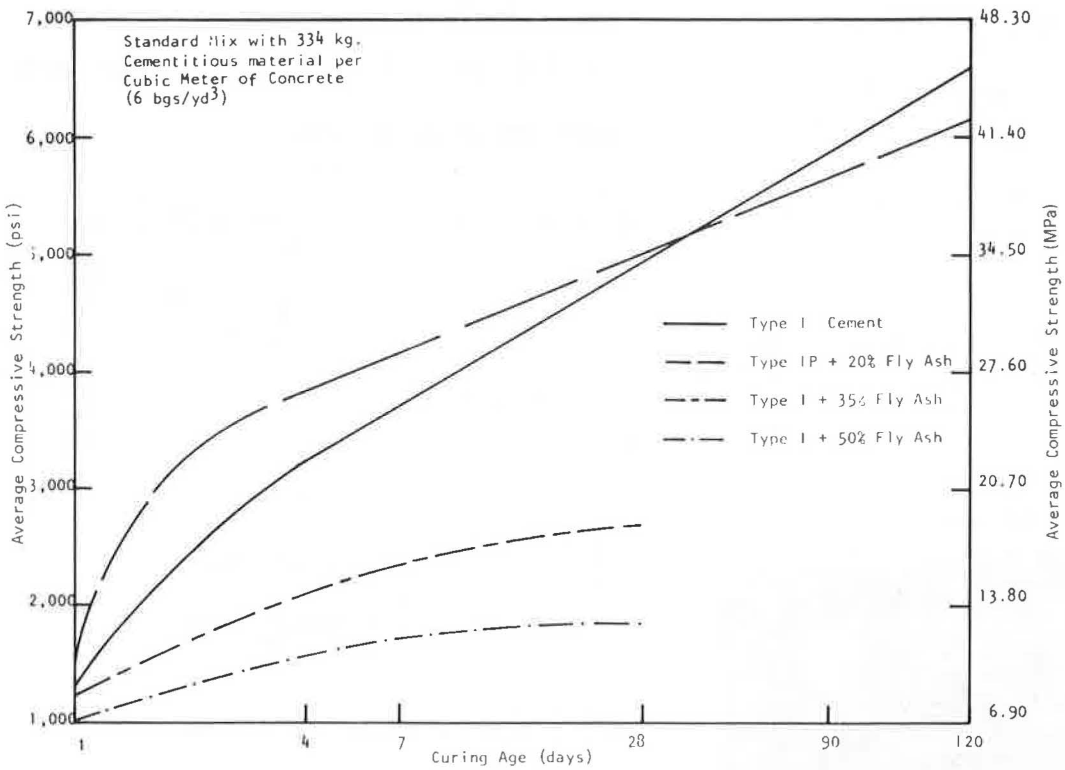
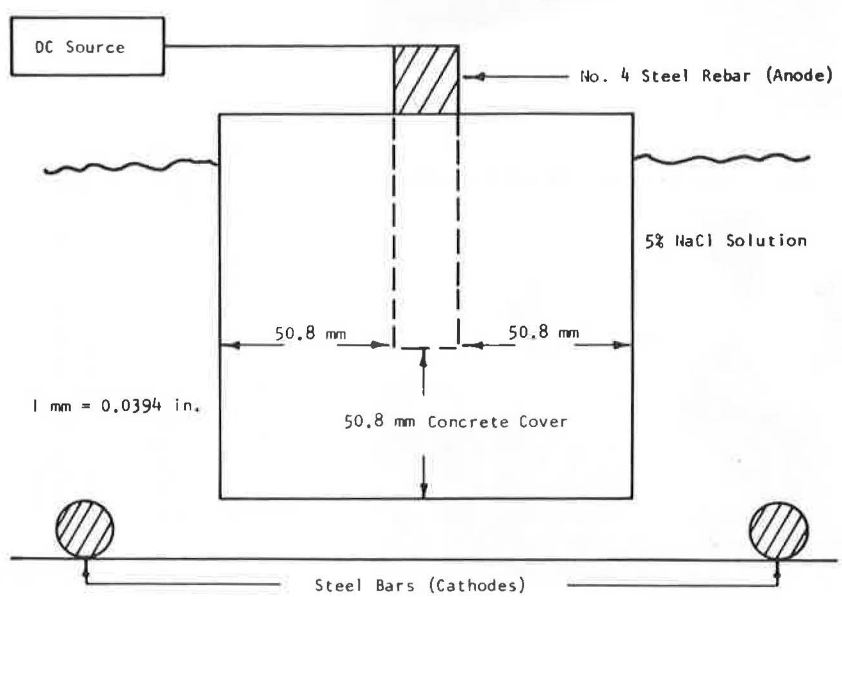


Figure 3. Components of test system.



to internal cracking. When the corrosion product begins to fill the crack, the resistance leading to decreasing the current flow is increased because of the constant voltage applied. This buildup of corrosion products results in stress in the concrete and causes further cracking and corresponding increases in current flow. A specimen may undergo several cycles of progressive cracking before visual cracks develop.

The reduced test data indicated that the best correlation between the independent and dependent variables was obtained when the dependent variables were combined in energy units required to produce failure. Such correlations are demonstrated by the test results shown in Figures 8 through 11. The electrical energy units, expressed as the product of current, of electropotential, and of time to failure, were calculated for each specimen. Although considerable variation was observed, the trends established and shown in Figure 11 lend considerable support to using the concept of energy in the data analysis. The data analysis consisted of comparing the energy consumption versus the dependent test variables. It was theorized that the energy requirement reflected the ability of the concrete to prevent corrosion.

Influence of Cement Type

The influence of cement type is shown in Figure 8 in which the energy requirements for failure are compared for specimens with type IP and type I cements. The concrete with type IP cement was superior to a corresponding concrete with type I cement.

Influence of Curing Method

Figure 8 also shows the influence of the various curing

Figure 4. Test schematic and formation of chemical compounds.

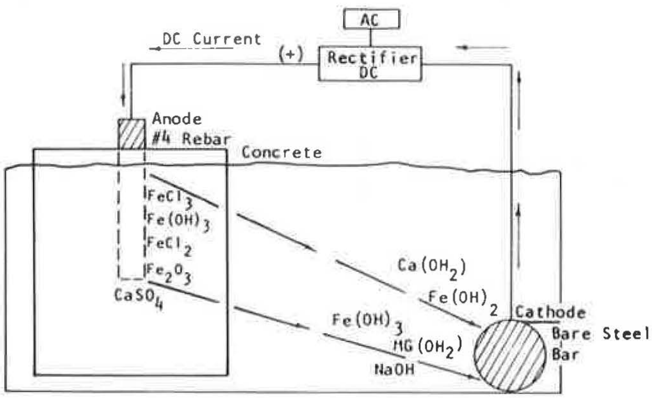


Figure 5. Overall view of typical test system.

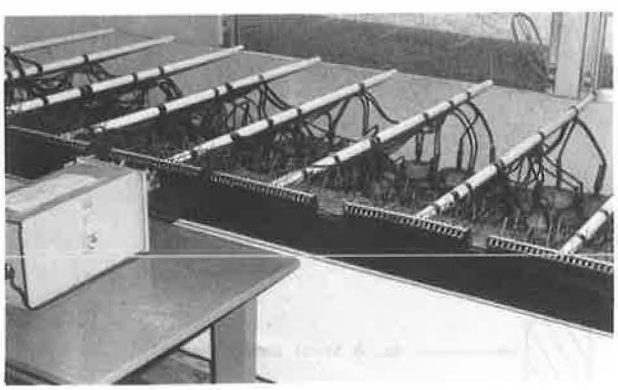


Figure 6. Before-and-after failure photographs of typical specimen.

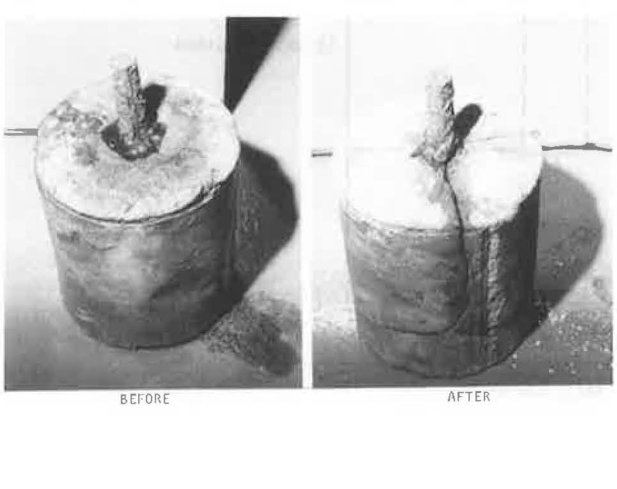


Figure 7. Idealized trends for test data.

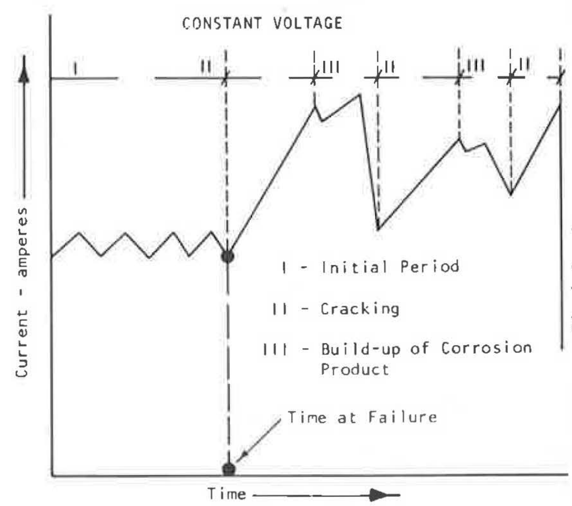


Figure 8. Influence of curing on corrosion resistance.

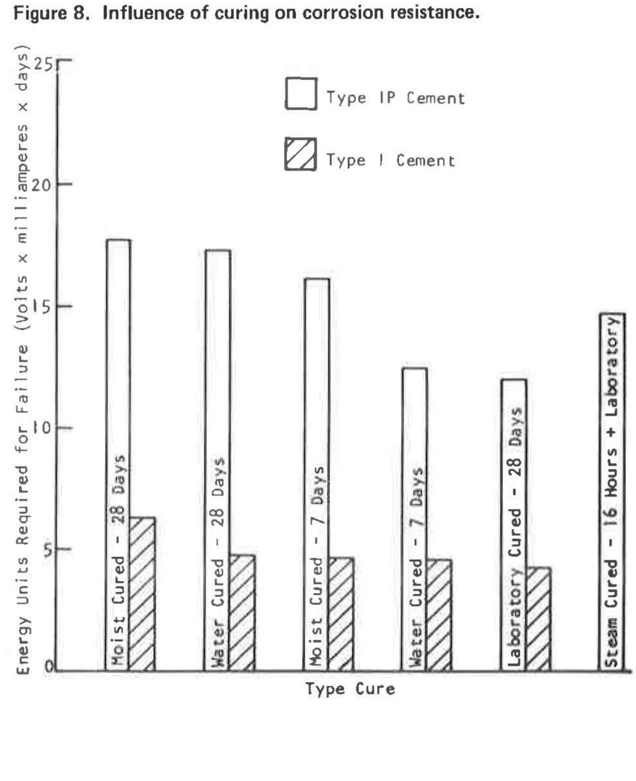
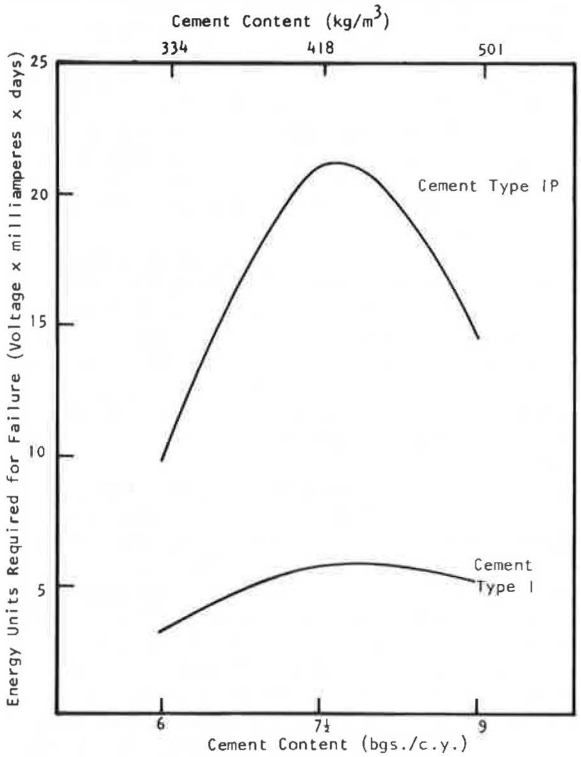


Figure 9. Influence of cement content on corrosion failure.

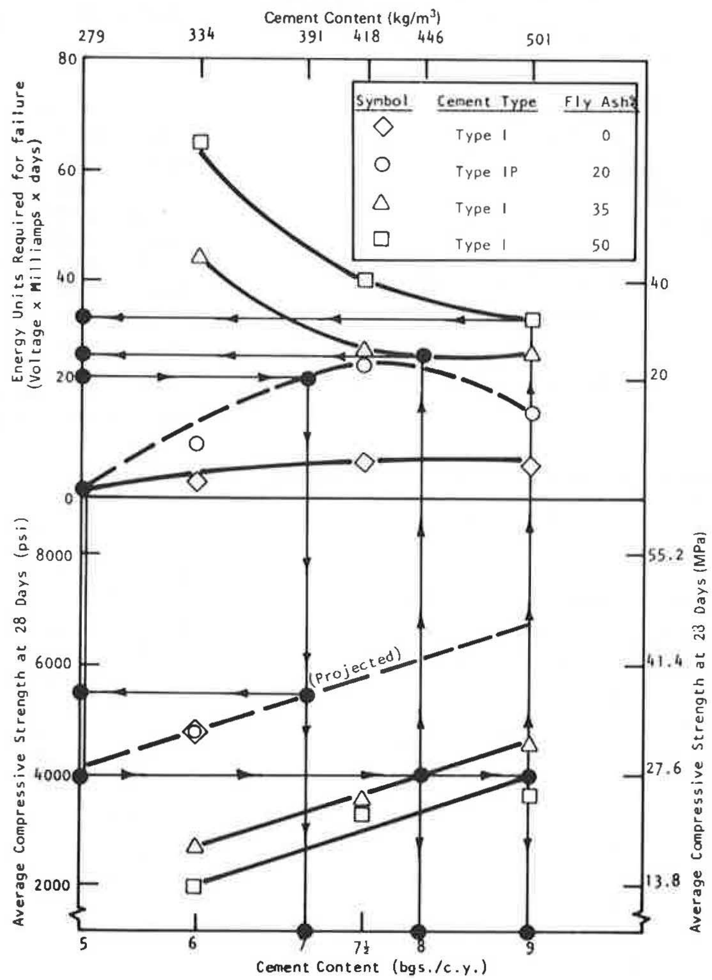


methods. The moist- and water-cured specimens with type IP cement showed similar performance for the 28-d curing period. For the 7-d curing period, the moist-cured specimens performed better than the water-cured specimens. Specimens cured under the environmental condition of the laboratory (air-cured) showed the lowest performance for both cement types. Specimens made with type IP cement, steam-cured for 16 h and then left in the laboratory environment, performed well, but exhibited a lower performance than those moist-cured or water-cured. For similar curing conditions, improvement was noted between the 7- and 28-d curing periods. Of the specimens made with type I cement, those cured for 28 d showed the best performance. For all other type I cement specimens, regardless of curing method, the energy requirements to induce failure were approximately the same.

Influence of Cement Content

The influence of cement content on the energy required to induce failure is shown in Figure 9 in which energy is plotted versus cement content. For both cement types, the performance improved as the cement content was increased from 335 to 418 kg/m^3 (6 to 7.5 bags/yd^3). Cement type IP showed a considerable decrease in performance when the cement content was increased from 418 to 502 kg/m^3 (7.5 to 9 bags/yd^3). This decrease was less marked for specimens with type I cement, and the lack of improvement was thought to be related to the mix design.

Figure 10. Influence of cement on corrosion failure and average compressive strength.



Water to Cement Ratio and Consistency

For the selected mix design, the desired range in the w/c ratio was impractical. The ratio used generally varied between 0.40 and 0.50 and showed no correlation with the energy requirement to induce failure. Because the desired range in the w/c ratio could not be attained, a variation in consistency was included for 5.08, 10.16, and 15.24 cm (2, 4, and 6 in) of slump. In the case of specimens with type I cement, the performance decreased slightly with increased slump. The reverse was true for specimens using type IP cement.

Fly Ash Content

In addition to the specimens containing type IP cement and 20 percent fly ash, a partial factorial experiment was made for specimens containing 35 and 50 percent fly ash. The data for the energy required to induce corrosion failure are shown in Figure 10. Also shown in Figure 10 are the compressive strength values and the comparative data for type I and type IP concretes.

Generally, the data demonstrate that the energy required to produce corrosion failure increased as the fly ash content increased. The energy value decreased as the cement content increased in the specimens with fly ash. This is contrary to the observations made for concretes with type I and type IP cements. Figure 10 also shows the 28-d compressive strength versus the cement content for concretes with various amounts of fly ash from 0 to 50 percent.

From the data shown in Figure 10, the deductions given in Table 3 may be made. For 27.58 MPa (4000 lbf/in^2) and 28-d strength, the cement requirement is 279 kg/m^3 (5 $bags/yd^3$) for both the type I and type IP cement. Energy required to produce failure is one unit. Using 35 percent fly ash would require 341 kg/m^3 (8 $bags/yd^3$) of cement and 24 energy units, and using 50 percent fly ash would require 384 kg/m^3 (9 $bags/yd^3$) of cement and 32 energy units.

A specification may contain requirements for both the strengths and the corrosion protective properties. If the latter is specified at 20 units and the strength is better than 27.58 MPa (4000 lbf/in^2), the type I cement would not meet the energy requirement. The type IP cement would require a cement content of 390 kg/m^3 (7 $bags/yd^3$) to give 37.9-MPa (5500-lbf/in^2) strength and a minimum energy requirement of 20 units. For 35 percent fly ash content, 446 kg/m^3 (8 $bags/yd^3$) of cement is required to meet the minimum strength requirement. In this case, the energy required to produce corrosion failure is 24 units. When the fly ash content is increased to 50 percent, the cement content is 502 kg/m^3 (9 $bags/yd^3$) of cement to give the minimum strength requirement. The corresponding energy resistance is 32 units.

Figure 11. General trend of test data.

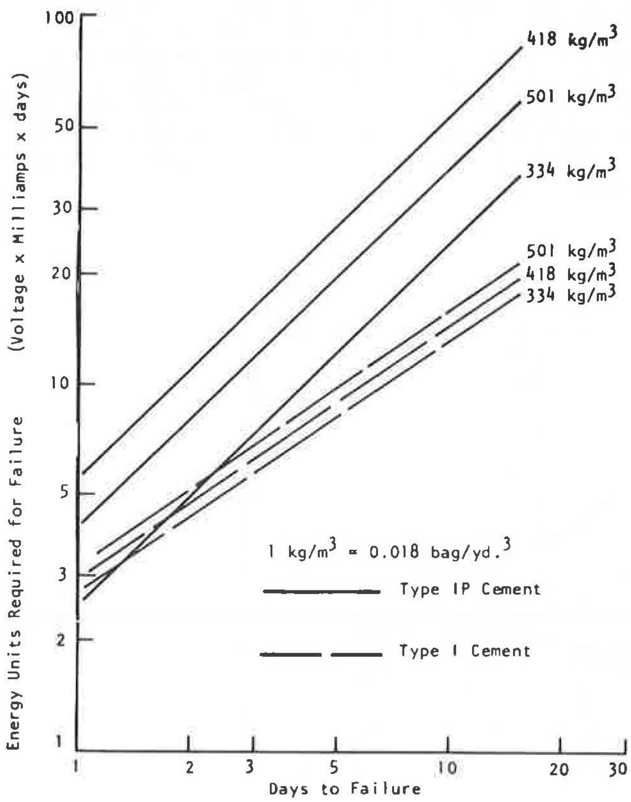


Table 3. Fly ash content and physical properties of concrete.

		Requirement		From Figure 10		
Cement Type	Fly Ash (%)	Compressive Strength (MPa)	Energy Units (V·mA·d)	Compressive Strength (MPa)	Cement Content (kg/m^3)	Energy Units (V·mA·d)
I	0	27.6	0	27.5	279	1
IP	20	27.6	0	27.6	279	1
I	35	27.6	0	27.6	446	24
I	50	27.6	0	27.6	502	32
I	0	27.6	20	—[a]	—[a]	—[a]
IP	20	27.6	20	38.0	391	20
I	35	27.6	20	27.6	446	24
I	50	27.6	20	27.6	502	32

Note: 1 Pa = 0.021 lbf/in^2; 1 kg/m^3 = 0.018 $bags/yd^3$.

[a]Not feasible.

CONCLUSIONS

1. The compressive strengths of concrete made with type I and type IP cements do not differ significantly at an age of 28 d.
2. The consistency of concrete measured by the slump test does not appreciably influence the time to induced corrosion failure.
3. The cement content of concrete generally correlates with the corrosion protection afforded to encased steel. However, this experiment showed that no protection advantage is obtained by further additions of cement beyond 418 kg/m^3 (7.5 $bags/yd^3$).
4. The method of curing has a definite effect on corrosion-inhibiting properties of concrete. Regardless of cement type, the most beneficial methods are moist-curing and water-curing followed in order by steam-curing and air-curing.
5. The electrical energy required to induce corrosion failure is a feasible means for relating concrete parameters to the effectiveness of concrete to inhibit corrosion.
6. The permeability of concrete made with type IP cement is somewhat lower than the permeability of concrete made with type I cement.
7. The addition of fly ash increased the energy required to produce corrosion failure. However, fly ash content beyond that for type IP cement (20 percent) lowered the 28-d compressive strength.

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Strength Improvements in Mortar and Concrete by Addition of Epoxies

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To raise the tensile strength of cement mortar and concrete, we conducted a series of experiments in which suitably formulated epoxy resins were added to standard cement mortar and to selected concrete mixes. Tensile and compressive strength measurements were made on various formulations, and strengths were determined as a function of epoxy-added content. Both tensile and compressive strengths are appreciably increased by additions of epoxy to cement mortar and concrete. These additions are 5 and 10 percent by weight of mortar for the cement mortar and concrete respectively. The effects of polymer additions on aggregate-paste bond have been studied by use of a scanning electron microscope. The influence of the epoxy on workability, stiffness, and water absorption is also discussed.

Various methods have been proposed in recent years for improving the strength characteristics of cement-mortar and cement-concrete systems. One technique involves removing the water in precast concrete slabs by oven drying, evacuating the dried concrete, and then impregnating a monomer. This monomer is subsequently polymerized by radiation, heat, or use of chemical catalysts (1, 2, 3). The polymer-impregnated concrete (PIC) system develops greater strength and exhibits greater freeze-thaw resistance than conventional systems. However, these products are expensive to produce, and the methods are not directly applicable to field use.

Another technique that is more economical and more applicable for field use is to directly add a polymer component to cement or concrete during the mixing operation. This final product is a polymer-portland cement-concrete (PPCC). It has been stated that PPCC systems have shown limited promise of improvement unless high concentrations of polymer are added (1). Most of the PPCC systems reported in the literature have involved the addition of a polymer in the form of emulsions or natural or synthetic latexes; polyvinyl acetate has been one of the preferred materials for this application (2).

In this investigation, epoxy systems that are capable of being converted into a three-dimensional network structure are used as the polymer component to produce higher strength polymer-cement mortars and higher strength polymer-cement concretes. These particular PPCC systems have several advantages. First, both the resin and the hardener can be obtained in the form of relatively low-viscosity liquids that can be easily incorporated with the other ingredients—cement, water, sand, and gravel—in the mixing operation. Second, epoxy systems can be selected that, at ambient temperature, will cure in time to a three-dimensional network without appreciable shrinkage. Third, by the addition of a liquid epoxy component, the amount of mixing water can be reduced without serious detriment to workability. Furthermore, the epoxy resin and the hardening agent can readily be premixed before incorporation into the cement-water-aggregate system.

This paper emphasizes the amount of strength improvement that can be expected from epoxy-cement mortars and epoxy-cement concretes. In addition, the use of the scanning electron microscope (SEM) to provide information about the nature of the microstructure in these samples is discussed. SEM techniques have been used with some success in nonpolymer cements (4, 5, 6), and these techniques have been used with electron microscope techniques (7, 8, 9) to provide useful information about the microstructure. Observations of fracture-surface morphology are used to interpret the tensile and compressive strength data and to clarify the role played by addition of the epoxy component. Some aspects of this study relative to epoxy-cement mortar systems have been previously described (10, 11), and other aspects pertaining to epoxy-cement concrete systems have been presented elsewhere (11, 12, 13).

COMPOSITION OF TEST SPECIMENS

In this study, the cement was type 1 portland cement, the sand was a graded natural sand, and the coarse aggregate was a 19.05-mm (¾-in) graded crushed stone. The liquid epoxy resin was a diglycidal ether of bisphenol A. Several hardeners were tried, and the one found to give the best results was a liquid-curing agent, di-β-hydroxyalkylamine.

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*Messrs. Sun and Cook were with the College of Engineering, Rutgers—the State University of New Jersey, when this research was performed.

The liquid hardener was blended with the liquid epoxy resin of low molecular weight in a stoichiometric ratio of 100:75 shortly before addition to the cement or concrete mix.

Two series of epoxy-cement mortar samples were prepared. The composition of the samples in each of these series is given in Table 1. In series B specimens, the cement:sand:water ratio was maintained constant at 1:2.75:0.3, and percentages of epoxy by weight of mortar from 0 to 15 percent were added. In the series E specimens, the amount of sand was maintained constant, and a higher water to cement ratio was adopted. However, as the epoxy was added, the amounts of water and cement present were reduced as indicated in Table 1. Thus, series E1 specimens are epoxy-free cement-mortars, and series E6 specimens are cement-free polymer mortars.

The composition of the epoxy-cement concretes is given in Table 2. Again, samples were prepared from two different test series, designated B* and C*. In series B*, the cement:sand:aggregate ratio was maintained constant at 1:2.68:4.26, and ratios of epoxy varying from 0 to 15 percent by weight of the mortar and 0 to 64.1 percent by weight of cement were added. The water to cement ratio was not maintained constant in this series, but was varied, as given in Table 2, to maintain an approximately constant slump of 76.2 to 88.9 mm (3 to 3.5 in). In test series C*, the cement: sand:aggregate ratio was held at 1:1.62:2.44, and the water to cement ratio was gradually reduced from a high initial value of 0.67 to a final value of 0.15 as the polymer to cement ratio was increased as given in Table 2. Thus the percentage of water replaced by epoxy varied from 0 percent for test series C*1 to 77.7 percent for test series C*6.

EXPERIMENTAL PROCEDURE

For the polymer-cement mortar specimens, the dry sand and fresh cement were mixed by hand, and then the desired amount of water was added until a uniform mixture was obtained. The blended epoxy resin and hardener were added in the desired proportions, and mixing continued until a uniform dispersion was attained. The freshly mixed epoxy-cement mortar was placed in cylindrical molds 50.8 mm (2 in) in diameter and 101.6 mm (4 in) in length. The material was compacted by tamping repeatedly with a circular rod. The molded specimens were maintained in a moist atmosphere (100 percent RH) for 24 h and then removed from the molds and stored under water until ready for testing.

For the epoxy-cement concrete specimens, the mixing procedure was essentially as indicated above; the coarse aggregate was added after the polymer-cement mortar had been formed. Mixing was continued until a uniform dispersion was achieved. Test specimens were prepared from the mix by putting the mix into steel molds 152.4 mm (6 in) wide and 304.8 mm (12 in) high and storing the molds under moist conditions (100 percent RH). After 24 h, the specimens were removed from the molds and kept in the moist room until ready for testing.

A hydraulic testing machine was used to carry out both compressive tests and tensile-splitting tests for specimens in all series listed in Tables 1 and 2 and for specimens aged 7 and 28 d. In most cases, three to four specimens were tested for each specific composition and for each age level. For the compression tests, the ends were capped with bond about 2 h before the test, and an aligning device was used to ensure perpendicularity of the caps to the axis of the specimen in accordance with ASTM requirements.

The fracture surfaces of a number of specimens from the tensile-splitting tests were examined by using the SEM with a secondary electron or emissive mode of operation. Before examination under the microscope, the fracture surfaces were coated with carbon and platinum-gold in a vacuum evaporator. SEM micrographs were taken at a variety of magnifications to resolve as completely as possible the nature of the microstructure and the efficiency of the bond between matrix and aggregate.

Test Results for Epoxy-Cement Mortars

The average tensile and compressive strengths for the series B epoxy-cement mortar specimens are shown as a function of resin-added content in Figure 1. The relatively low value for the 28-d compressive strength of the cement-only mortar (0 percent added resin) may reflect the difficulties encountered when trying to obtain a uniform mix by using low water to cement ratios. From these test results, it is evident that appreciable increases in strength were realized for an epoxy-added content (based on weight of mortar) of only 5 percent. These increases in strength are approximately proportional to epoxy content up to 5 percent, but higher percentages of epoxy content gave no additional improvement.

SEM micrographs of the fracture surfaces for three tensile-splitting test samples are shown in Figure 2. The fracture surface of the cement-mortar specimens (BO series) is quite granular in nature, and the fracture appears to have bypassed most of the sand particles. The fractured sand particles in the B1 series indicate that the 2 percent additions of epoxy resulted in some improvement of the bond between the cement matrix and the sand. The fracture surface of the B2 series containing a 5 percent addition of epoxy shows many such fractured sand grains and less evidence of fracture occurring around the grains.

It can be concluded that addition of the epoxy to the mortar has at least two beneficial effects. First, by increasing the liquid component available (water plus liquid epoxy), workability has been improved and a more uniform dispersion of the various components throughout the matrix has been achieved. Workability alone could have been increased by simply adding water, but excess water tends to decrease strength and to increase porosity. Second, because of the presence of many fractured sand particles on the fracture surfaces of the epoxy-modified specimens, the bond of the matrix to the fine aggregate appears considerably improved. For the 28-d cured specimens, tensile strengths increased about 90 percent, and compressive strengths increased over 125 percent by the addition of only 5 percent epoxy (by weight of the mortar) to the mix. This is a considerable improvement and may justify the added costs of the epoxy-modified mortar in critical applications in which high tensile strengths and compressive strengths are desired.

The average tensile and compressive strengths of test samples from series E epoxy-cement mortars are shown in Figure 3 as a function of resin replacement of the cement-water phase. For the cement-only (0 percent resin) mortar specimens, the 28-d tensile and compressive strength values are higher than those for the control specimens (0 percent resin) of test series B. Thus, further evidence is provided by the low strength values of the BO series that too low a liquid content does not give adequate workability and uniform dispersion. For the E1 series with a water to cement ratio of 0.4, workability is greatly improved. As shown in Figure 3, the data indicate a minor strength improvement with 20 percent epoxy replacement, but with replacements in the range of 40 to 60 percent both the 7 and 28-d strength values are diminished. One possible reason for this behavior is that the higher resin content interferes with the hydration of the cement, and hence a continuous cement gel is not maintained. At still a higher percentage of epoxy replacements, the average 28-d strength values rise quite sharply; however, this is not the case for the 7-d strength values. The large differences between the 7 and 28-d strength values indicate that the epoxy cures only with time and under ambient conditions. If it is desired to accelerate the curing time, this could be done by post-curing at elevated temperatures or possibly by adding a known accelerator to the epoxy formulation.

Although the epoxy-mortar specimens of the E6 series exhibit greater tensile and compressive strengths than the strengths of the cement-mortar specimens in test series E, it is somewhat surprising that the differences are not greater. For example, the average compressive strengths of specimens in the E6 series are about 59.3 MPa (8600 lbf/in^2), while the attained compressive strengths of specimens in the B2 series with only a 5 percent addition of epoxy are about 55.2 MPa (8000 lbf/in^2). The SEM micrographs resolved the question as to why higher values were not obtained for specimens of the E6 series with an epoxy matrix of 100 percent.

Figure 4 shows low magnification SEM micrographs of specimens from test series E1 (no epoxy), E3 (40 percent epoxy), and E6 (100 percent epoxy). For the E1 specimen, a large void is present on the fracture surface, and some voids contain hydrated particles as well as some fractured and unfractured sand grains. However, the bond of the cement paste to the aggregate in the E series is superior to the bond in the B series, and this accounts for the higher strength values of the E series specimens. For sample E3, with the binder consisting of 60 percent of a cement-water phase and 40 percent epoxy, the fracture surface is more powdery and has several unfractured sand grains. Thus, the bond of the matrix to the fine aggregate appears poor, and the strength values are minimal. For the E6 sample, with only epoxy as the matrix, the matrix is more compact and uniform, but there are many small voids on the fracture surface.

A higher magnification, shown in Figure 5, reveals

Table 1. Composition of epoxy-cement mortar.

Specimen Series	Cement:Sand:Water Ratio	Epoxy Content (%) Cement Mortar	Cement and Water Replacement
B0	1:2.75:0.3	0	—
B1	1:2.75:0.3	2	—
B2	1:2.75:0.3	5	—
B3	1:2.75:0.3	10	—
B4	1:2.75:0.3	15	—
E1	1:2.75:0.4	—	0
E2	0.8:2.75:0.32	—	20
E3	0.6:2.75:0.24	—	40
E4	0.4:2.75:0.16	—	60
E5	0.2:2.75:0.08	—	80
E6	0:2.75:0	—	100

Table 2. Composition of epoxy-cement concrete.

Specimen Series	Ratio Cement:Sand:Aggregate	Water:Cement	Resin:Cement	Water Replaced by Resin (%)
B*1	1:2.68:4.26	0.60	0.00	—
B*2	1:2.68:4.26	0.545	0.214	—
B*3	1:2.68:4.26	0.557	0.428	—
B*4	1:2.68:4.26	0.340	0.641	—
C*1	1:1.62:2.44	0.67	0.00	0
C*2	1:1.62:2.44	0.55	0.106	17.9
C*3	1:1.62:2.44	0.45	0.221	32.9
C*4	1:1.62:2.44	0.35	0.321	47.7
C*5	1:1.62:2.44	0.25	0.422	62.8
C*6	1:1.62:2.44	0.15	0.522	77.7

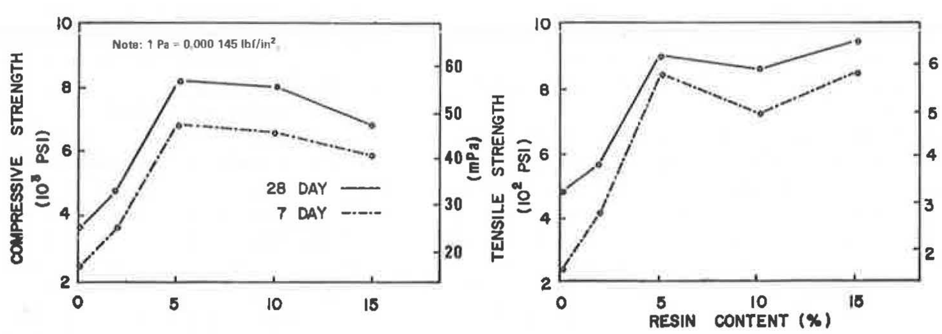


Figure 1. Tensile and compressive strengths versus resin content for series B epoxy-cement mortar specimens.

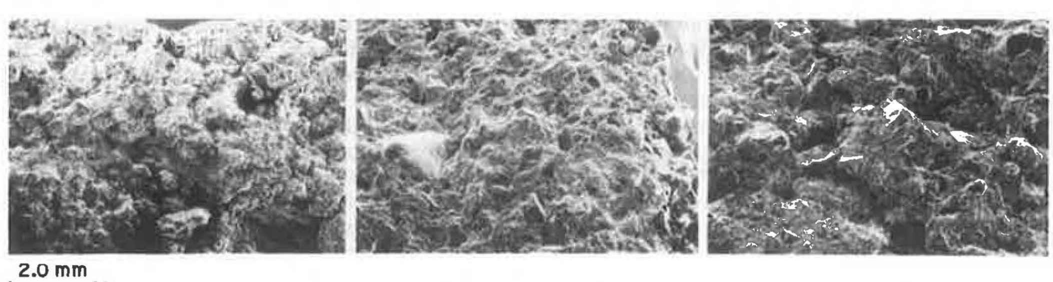


Figure 2. Scanning electron micrographs of fracture surfaces of tensile splitting specimens of series B epoxy-cement mortars (left, B0 series with no epoxy; middle, B1 series with 2 percent epoxy; right, B2 series with 5 percent epoxy).

additional features of the microstructure present in the cement-mortar specimens and in the epoxy-mortar specimens. For the E1 specimens, large voids are seen containing many dentritic hydrate particles as well as some fractured sand particles. For the E6 specimen, the resin matrix is less powdery and more homogeneous than the cement matrix, and the bond to the fine aggregate particles is excellent. This factor leads to the higher strength values shown in Figure 3. However, many small isolated voids are present on the fracture surface, and these presumably exist throughout the bulk material. Evidently, the voids are due to the presence of air bubbles in the initial mix. If these bubbles could be eliminated or reduced by more efficient mixing or by evacuating the mix, then higher strength values than those shown in Figure 3 for the E6 specimens would be expected. Nevertheless, despite the many air pockets, the more homogeneous matrix phase in which there is good adhesion of epoxy to the fine aggregate is considered responsible for the observed strength increases.

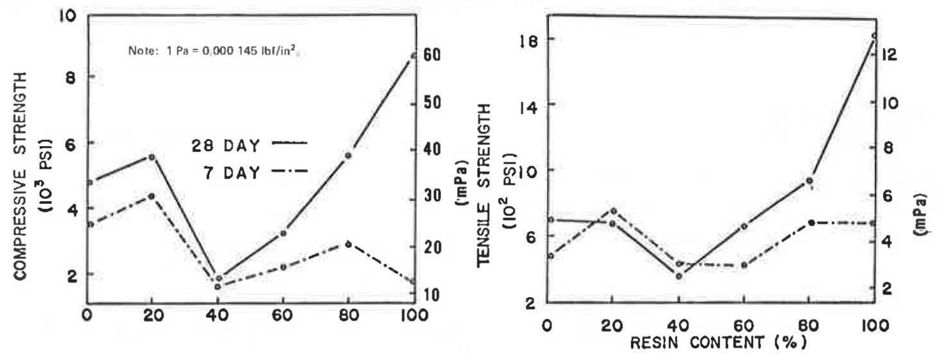


Figure 3. Tensile and compressive strengths versus resin content replacing cement-mortar for series E epoxy-cement mortar specimens.

Figure 4. Scanning electron micrographs of fracture surfaces of tensile-splitting specimens of series E epoxy-cement mortars (left, E1 series with no epoxy; middle, E3 series with 40 percent epoxy replacement; right, E6 series with 100 percent epoxy replacement).

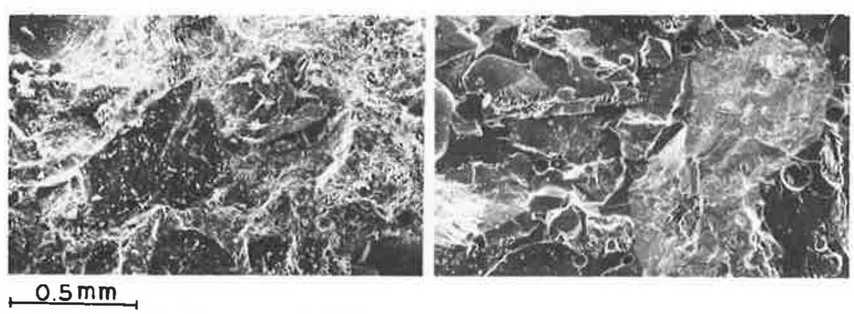


Figure 5. High magnification SEM micrographs (28X) of fracture surface of tentile-splitting specimens for series E epoxy-cement mortars (left, E1 series with no epoxy; right, E6 series with only epoxy).

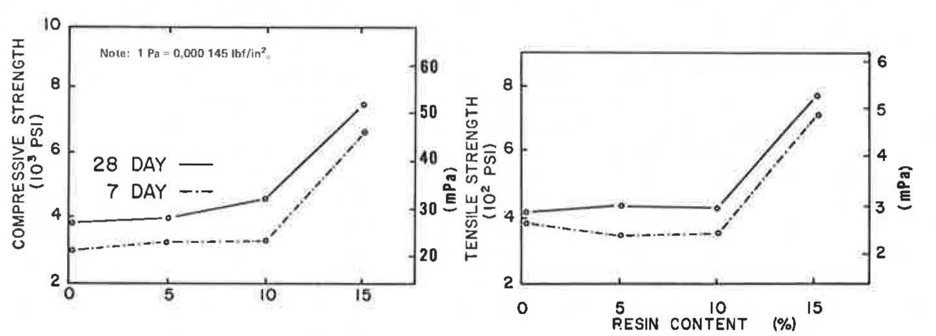


Figure 6. Tensile and compressive strengths versus resin content for series B* epoxy-cement concrete specimens.

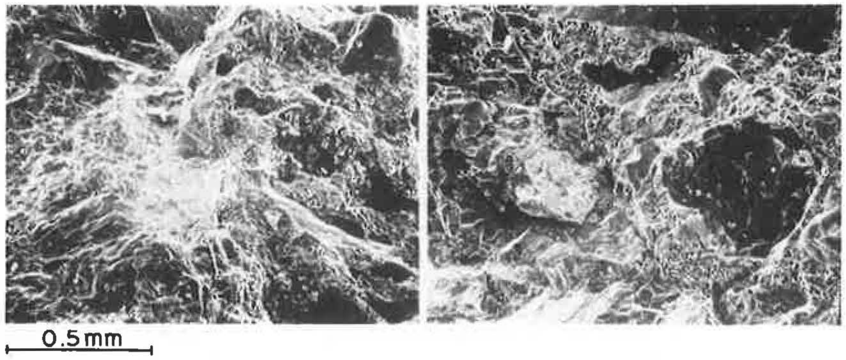


Figure 7. High magnification SEM micrographs of the fracture surfaces of tensile-splitting specimens of series B* epoxy-cement concretes (left, B*1 series with no epoxy; right, B*4 series with 15 percent epoxy).

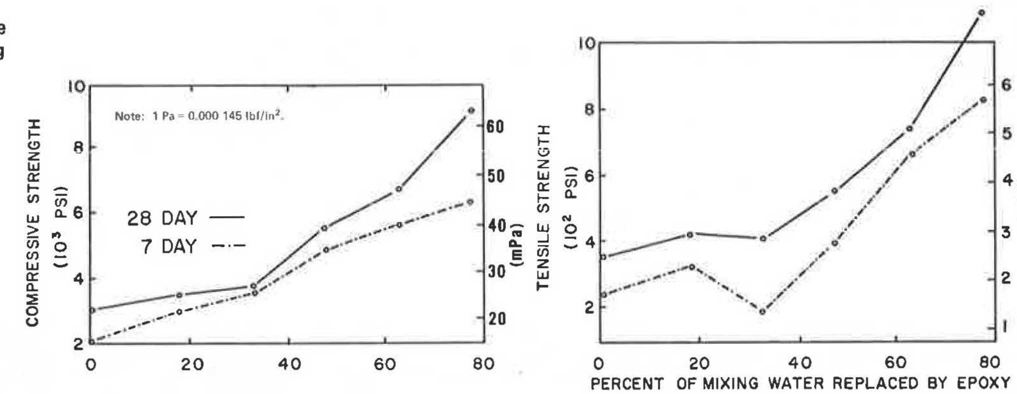
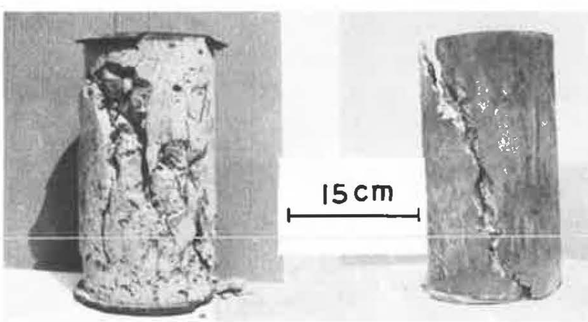


Figure 8. Tensile and compressive strengths versus percent of mixing water replaced by epoxy for specimens of series C* epoxy-cement concretes.

Figure 9. Photographs showing fracture modes in compression of series C* epoxy-cement concrete specimens (left, C*2 series with resin cement ratio of 0.106 and water cement ratio of 0.55; right, C*6 series with resin cement ratio of 0.522 and water cement ratio of 0.15).



A B

The increase of epoxy content in the resin-cement mortars has advantages other than strength alone. Measurements of the elastic modulus of specimens in series E show that stiffness decreases essentially linearly with increasing epoxy content (10). Since the strengths of the less stiff specimens with high epoxy content are higher, the toughness and energy-absorbing ability of these specimens will also increase considerably in comparison with the regular cement mortars. Similarly, freeze-thaw resistance should improve as resin content increases, since measurements of water absorption in E series specimens show that water absorption decreases in an essentially linear manner with resin content (10). In fact, for the addition of 100 percent epoxy in specimens, water absorption is essentially nil.

Test Results in Epoxy-Cement Concretes

The tensile and compressive strengths for B* series epoxy-cement concrete specimens are shown as a function of epoxy content in Figure 6. It is evident from these data that there is little strength benefit derived from a 5 or 10 percent addition of epoxy; however, for a 15 percent addition of epoxy (by weight of the mortar), there is an appreciable increase in strength. For example, the average tensile strength for specimens cured 28 d increased 90 percent, and the average compression strength increased by 98 percent. From these data it is clear that appreciable improvements in the strength of the concrete can be realized by appropriate additions of epoxy while still maintaining sufficient water content to hydrate the cement and give good workability.

SEM micrographs of the fracture surfaces of tensile splitting samples from series B*1 (no epoxy) and from series B*4 (15 percent epoxy) are shown in Figure 7. For the B1 sample, a mortar region is visible between the fractured aggregate particle (lower left) and the unfractured aggregate particle (upper right). The mortar region shown as mostly grey cement grains together with some hydrated cement particles indicates that the fracture has bypassed many of the fine aggregate particles. There are some visible cracks at the matrix-coarse aggregate interface, but these probably occurred after fracture since the large aggregate particles appear to have cleaved.

For B*4 specimens, with 15 percent epoxy, there is more evidence of fractured sand particles in the mortar phase and less evidence of unfractured fine aggregate. Also, a crack is visible at the region of the interface between the epoxy-cement matrix and the coarse aggregate particle. Nevertheless, it can be clearly seen from the sharp cleavage features of the stone particles that the fracture surface has clearly passed through rather than around the coarse aggregate.

It appears from both the strength data and the SEM pictures that the addition of some 15 percent epoxy to the mix (by weight of the mortar) results in a more uniform mortar phase in which the adherence between the matrix and the aggregate particles is improved. As a result of this addition, the strength of the concrete is considerably improved. One would also expect some improvement in freeze-thaw resistance because, as noted earlier, water absorption is reduced with the addition of epoxy (10).

For the C* series specimens in which the added epoxy component simply replaced part of the water, the average 7 and 28-d strength values that are obtained as a function of percentage of water replaced are shown in Figure 8. For the samples cured 28 d the compressive strength rises, from an initial value of about 20.7 MPa (3000 lbf/in^2), rather gradually with epoxy replacement, and, after 30 percent replacement the strength rises at a much faster rate. Similar behavior is also noted for the tensile strength. Up to about 30 to 40 percent replacement of water by epoxy, it appears that sufficient water is still present to form a cement hydrate phase, and the strength values obtained are representative of that phase. However, for epoxy replacements of 50 percent and above, the epoxy component appears to provide the primary matrix material, and both unhydrated and hydrated cement particles are incorporated into the epoxy network and into the aggregate as essentially filler particles.

Series C*5 specimens showed a considerable strength improvement, even though they have an epoxy-cement ratio of 0.422 that is comparable to that of series B*3 specimens, which had little strength improvement. This improvement is probably a direct result of the lower water content for the C*5 specimens. Also, for series C*6 specimens with a resin-cement ratio of 0.522 (less than that of test series B*4), the average tensile strength has increased 160 percent to 7.7 MPa (1110 lbf/in^2) and the average compressive strength has increased more than 200 percent to 63.0 MPa (9140 lbf/in^2). Thus, by reducing the water-cement ratio and by adding epoxy to maintain adequate workability, it appears that this is a promising method for obtaining strength improvements in concrete. Because of the reduced water content and the already noted imperviousness of cured epoxy to moisture, the freeze-thaw resistance of these modified cement concretes should improve.

An examination of the fracture surfaces of tensile-splitting specimens helps to delineate the role played by the addition of epoxy. As the epoxy component increased, there was an increase in the amount of aggregate failure. This failure indicated an improvement in the bond of the cement-epoxy matrix to the aggregate particles. This improved bond is also evident from the observed fracture patterns shown in Figure 9 of the compression specimens tested at low- and high-resin content. For a low-resin content specimen, the fracture pattern is similar to that of an ordinary cement-concrete specimen with considerable disintegration and many secondary cracks. However, for the high-resin content samples (series C*6), the fracture mode is predominantly caused by shear and there is less evidence of secondary cracks and interface bond failure between aggregate and mortar even though much higher loads were needed to produce failure. Measurements of the elastic modulus were also made for the series C* specimens. These test results show that the stiffness varied comparatively little with the resin content in these concrete specimens, but for high-resin content in the cement-concrete specimens the stiffness is somewhat lower (12). Thus, since tensile and compressive strengths are higher and the elastic moduli about the same or somewhat lower, the energy-absorbing capacity of the polymer-added concrete specimens should be greater than that of the cement-concrete specimens.

CONCLUSIONS

1. Strength improvements of the order of 100 percent are possible in both mortar and concrete specimens by addition of appropriate amounts of a liquid epoxy system to the mix.
2. In the case of mortar, these strength improvements can be realized by addition of only 5 percent epoxy by weight of the mortar. Greater strength improvements, but at a higher cost, are possible by having the epoxy largely replace the cement phase.
3. In the case of concrete, these strength improvements can be realized by adding 15 percent epoxy by weight of the mortar while, at the same time, proportionately reducing the water content to maintain comparable slump. Still greater strength improvements are possible by reducing the water to cement ratio further and replacing the water by epoxy.
4. SEM micrographs indicate that the addition of epoxy results in a more uniform dispersion, especially in the mortar phase, and leads to an increased bond between the matrix and the aggregate.
5. With increased resin content, there is little change (polymer-cement concretes) or reduction (polymer-cement mortars) in stiffness, and since strengths are higher both toughness and impact resistance should improve.

ACKNOWLEDGMENTS

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Influence of Sands Used in Switzerland on the Durability of Air-Entrained Concrete

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The strict Swiss specifications (SNV 640464 and 670745) concerning the content of fines in air-entrained concrete used for road paving often make the use of local sands impossible. Hence, sands from distant localities, sometimes from abroad, must often be shipped at higher costs. Results of tests on air-entrained concrete made with different sands commonly used in Switzerland are summarized in this paper. The durability of concrete was tested by means of the conventional frost method and also by applying the following new methods: (a) the quantitative microscopic determination of frost-salt resistance by pore analysis; (b) the qualitative control of concrete by microscopic morphological analysis; and (c) the frost-salt test with rapid cycles of frost and thaw according to the Dobrolubov-Romer method. The results indicate that (a) among the test methods used, a correlation was obtained; (b) the non-air-entrained concrete that consisted of Gebenstorf sand and a high content of fines failed after 20 cycles; (c) the same concrete but air-entrained resisted 300 cycles without decrease in the modulus of elasticity; (d) the air-entrained concretes that consisted of other sands and high contents of fines also reached 300 cycles without decrease in the modulus of elasticity; and (e) the air-entrained concretes that meet Swiss specifications showed the same durability after 300 cycles as the concretes that do not meet the specifications.

This study attempts to clarify the influence of different sands commonly used in Switzerland on the durability of air-entrained concrete. The test sands (between 0 and 4 mm) and the localities from which they were taken are listed below.

Locality	Designation	Locality	Designation
Gebenstorf (non-air-entrained)	GO	Weiach (round)	WR
Gebenstorf (air-entrained)	G	Flüelen	F
Weiach (crushed)	WG	Bedretto	B
		Pontresina	P

Aggregates, with sizes ranging from 2 to 32 mm, were taken from the Köppel quarry in Weesen. Figure 1 shows a schematic of the research program and the testing methods used. In addition to the conventional tests by the Swiss Federal Materials Testing Institute (EMPA) (1), the following new test methods were applied at the Laboratory of Preparation and Methods (LPM) (2):

1. The microscopic, quantitative, pore-analytical determination of frost-salt resistance;
2. The microscopic, morphological, qualitative control of concrete; and
3. The frost-salt test that used rapid cycles of frost and thaw at -20 and +20° C in accordance with the Dobrolubov-Romer (D-R) method.

Distinct from the conventional frost-test method (SIA-162), a frost test that uses temperatures of frost and thaw at +14 and -25° C respectively was carried out without making use of a parallel specimen stored in water. Instead of storing a specimen in water, three specimens were measured for linear expansion. This technique allowed for a comparison between the SIA frost test and the D-R frost-salt test.

PETROGRAPHIC ANALYSIS

The main test results are given in Table 1. With the exception of the B-sand, the other five sands were rated as petrographically acceptable for use in high-quality concrete. The B-sand contains an excessive amount of soft components that are not frost resistant; therefore, in accordance with the EMPA finding, this sand is not acceptable for use in concrete. The B-sand was intentionally included in the test program to analyze its effect on the frost and frost-salt resistance of the air-entrained concrete.

AGGREGATES AND CONTENTS OF FINES

The following Swiss specifications should be complied with to ensure the durability of concrete:

The sum of cement and fines between 0 and 0.2 mm must not exceed 400 to 420 kg/m³ of compacted concrete.

The maximum content of fines smaller than 0.02 mm must not exceed 1 percent of the total aggregate weight.

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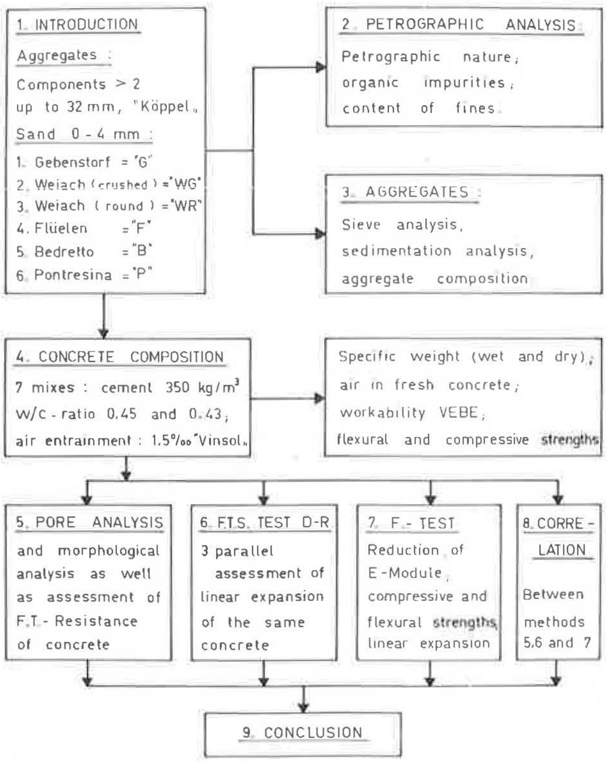
According to the data given in Tables 1, 2, 3, and 4, four of the six tested concretes do not comply with the above specifications.

CONCRETE COMPOSITION AND MIXTURE

Figure 2 shows the grading curves and the curve for concrete made with G-sand. In Table 3 the compositions and the mixtures of all tested concretes are summarized. A non-air-entrained concrete made of GO-sand with a w/c ratio of 0.45 was used for comparison purposes. All other concretes were air-entrained and had a w/c ratio of 0.43. The variation of the air content in air-entrained fresh concrete ranged between 5.1 and 6.9 percent, the volume weight between 2310 and 2375 kg/m^3, and the workability VEBE between 8 and 11 s. The values are all within narrow limits.

As given in Tables 5 and 6, the strength values assessed after 28 d also remained within relatively narrow limits: the flexural strength ranged between 62 and 82 kg/cm^2, and the compressive strength ranged between 363 and 413 kg/cm^2.

Figure 1. Design of research program.



MICROSCOPIC AND MORPHOLOGICAL CONTROL OF CONCRETE

Methods for quantitative microscopic determination of the frost-salt resistance by pore analysis and morphological qualitative control of concrete have been developed and successfully applied on all main road projects in Switzerland since 1969. A detailed description of these methods and of the frost-salt test has been given in a previous study (3); therefore, it will not be repeated here. Figure 3 shows the summation curves of pores of the tested concretes. Table 7 gives the characteristic values resulting from the analysis. In addition to the spacing factor (AF), nine other characteristic values are considered. An evaluation of the data in Figure 3 and Table 7 is shown in Figure 4. If the resulting characteristic values given in Table 7 are plotted, a curve is obtained that will lie within the limits of one of the five frost-salt resistance groups. In accordance with Figure 4, concretes G, WR, F, B, and P have a very high frost-salt resistance and concrete GO has a very low frost-salt resistance.

D-R FROST-SALT TEST

The D-R method makes the rapid frost-salt testing of concrete (frost at -20° C and thaw at +20° C) possible by using six test prisms, a specimen size of 3 by 3 by 6 cm per sample and a daily average of up to 50 frost-thaw

Table 1. Petrographic analysis of aggregates and sands.

Designation	Petrographic Nature	Sedimentation Analysis Weight (¢) by Grain Size (μm): 50 to 20	20 to 10	10 to 5	<5	Total >20	Applicable to High-Quality Concrete
Köppel	Siliceous and sand lime (f, k, h) hornstone; alpine limestone (f, k, mh)	0.76	0.46	0.32	0.67	1.45	Yes
G	Quartz-sandstone, silicic stone and sandstone, hornstone, and quartz; 3 to 4¢ molasse	0.90	0.39	0.31	0.56	1.26	Yes
WG	50 to 60¢ alpine sandstone, silicic stone and sandstone hornstone (f, k, h);	1.35	0.42	0.28	0.55	1.25	Yes
WR	20 to 25¢ crystalline sandstone (k, sp); 20 to 25¢ limestone and dolomite (f, k, mh); 1 to 2¢ weathered stone	0.42	0.23	0.15	0.57	0.95	Yes
F	25 to 30¢ alpine sandstone and silicic limes (t, k); 20¢ limestone (f, k); 35 to 40¢ crystalline stone (f, k, sp); 10 to 15¢ quartz-hornstone (k, sp); ~2¢ soft stones	0.31	0.16	0.11	0.29	0.56	Yes
B	Granite and mica-gneiss, phyllite, and carbonic stones; 15 to 20¢ non-frost-resistant stones	1.76	0.77	0.35	0.27	1.39	No
P	Only crystalline stones (f, k, some sp)	0.23	0.10	0.07	0.26	0.43	Yes

Note: Firm = f; compact = k; hard = h; medium hard = mh; and brittle = sp.

Table 2. Granulation analysis of sand.

Designation	Residue in Weight (¢) by Mesh Width (mm)									Residue in Weight (¢) by Grain Size (mm)					
	8.0	4.0	2.0	1.0	0.4	0.2	0.12	0.09	0.06	>0.05	0.05 to 0.02	0.02 to 0.01	0.01 to 0.005	<0.005	Total <0.02
G	0	0.4	22.8	24.8	22.4	15.1	7.2	2.8	2.4	97.1	1.2	0.5	0.3	0.9	1.7
WG	0.1	1.1	25.0	21.8	16.2	12.2	9.8	5.4	4.9	97.4	1.4	0.5	0.2	0.0	0.7
WR	0	0.8	8.7	8.4	17.6	38.2	18.3	4.4	1.9	98.6	0.4	0.3	0.1	0.6	1.0
F	0	1.7	16.8	26.8	33.5	15.2	4.3	0.9	0.4	99.1	0.3	0.2	0.1	0.3	0.6
B	0	0.8	14.2	21.2	22.0	18.6	12.3	4.5	3.0	96.8	1.8	0.8	0.3	0.3	1.4
P	0	0.8	17.1	25.0	27.3	19.7	7.3	1.6	0.7	99.3	0.3	0.1	0.0	0.3	0.4

Figure 2. Composition of concrete prepared with G-sand.

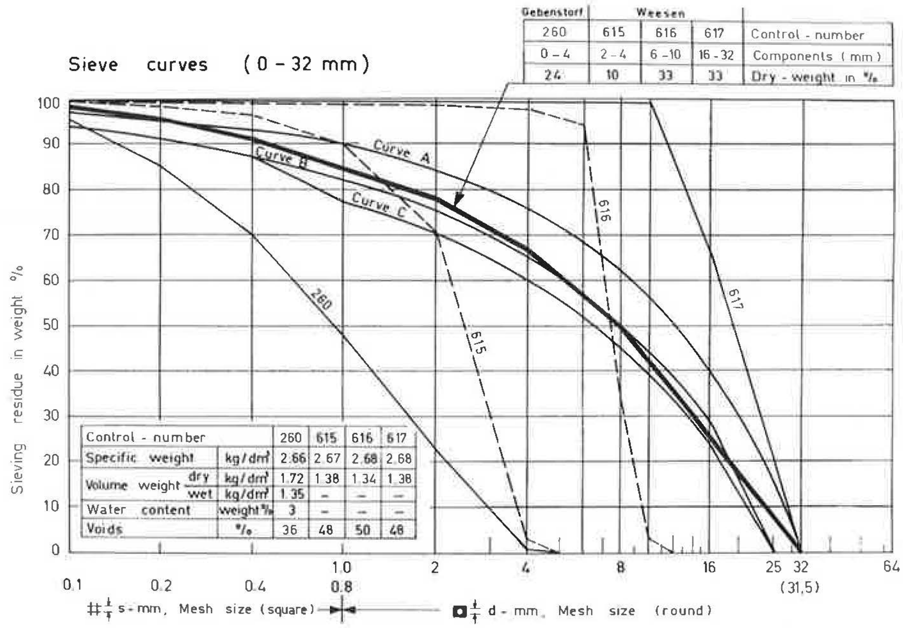


Table 3. Composition and preparation of concrete at EMPA.

Designation	Particle Size (mm)	Weesen Aggregates (dry wt)	Concrete Mix Estimated (kg/m^3)	Concrete Mix Determined (kg/m^3)	Concrete Mix 48 L Mixed (kg)	Holderbank Cement Estimated (kg/m^3)	Holderbank Cement Determined (kg/m^3)	Holderbank Cement 48 L Mixed (kg)	W/C Ratio	Water Estimated (kg/m^3)	Water Determined (kg/m^3)	Water 49 L Mixed (kg)	Volume Weight Estimated (kg/m^3)	Volume Weight Determined (kg/m^3)	Air Content in Fresh Concrete 10 Min	Air Content in Fresh Concrete 20 Min	VEBE (s)	Mixing Time (min)
GO	4	24	469	461	23.45	350	344	17.50	0.45	158	155	8.90	2462	2420	1.4	1.35	4.5	2
	4	10	195	192	9.75													
	10	33	645	634	32.25													
	32	33	645	634	32.25													
G	4	24	469	446	23.45	350	333	17.50	0.43	150	143	7.50	2454	2340	5.0	5.8	11	2
	4	10	195	186	9.75													
	10	33	645	616	32.25													
	32	33	645	616	32.25													
WG	4	24	469	451	23.45	350	336	17.50	0.43	150	144	7.50	2454	2360	5.6	5.6	9	2
	4	10	195	187	9.75													
	10	33	645	621	32.25													
	32	33	645	621	32.25													
WR	4	24	469	444	23.45	350	330	17.50	0.43	150	142	7.50	2454	2320	5.8	6.2	9	2
	4	10	195	184	9.75													
	10	33	645	625	32.25													
	32	33	645	610	32.25													
F	4	24	469	444	23.45	350	330	17.50	0.43	150	142	7.50	2454	2320	5.8	6.2	9	2
	4	10	195	184	9.75													
	10	33	645	610	32.25													
	32	33	645	610	32.25													
B	4	24	469	453	23.45	350	339	17.50	0.43	150	145	7.50	2454	2375	4.8	5.1	9	2
	4	10	195	188	9.75													
	10	33	645	625	32.25													
	32	33	645	625	32.25													
P	4	24	469	440	23.45	350	330	17.50	0.43	150	141	7.50	2454	2310	6.3	6.9	8	2
	4	10	195	183	9.75													
	10	33	645	608	32.25													
	32	33	645	608	32.25													

Notes: Test cores: 6 prisms, 12 x 12 x 36 cm.
Tests: 2 flexural and 2 compressive strengths tests; 3 frost and 3 frost-salt.
Air entrainer: 26.2 g/48 L mixed.
Storage: 18°C, 90% humidity.

Table 4. Amount of fines <0.02 mm in sand and total mix.

Designation	Amount in Sand (%)	Amount in Total Mix (kg) Fines	Cement	Total
G	1.7*	69	350	419
WG	1.3*	115	350	465*
WR	1.0	121	350	471*
F	0.6	7	350	357
B	1.4*	105	350	455*
P	0.4	49	350	399

*Values are above the allowable limits and do not comply with specifications.

cycles. The assessment of the frost-salt resistance is based on the linear expansion (L) and the corresponding number of cycles (Z). The chart for the determination of frost-salt resistance is shown in Figure 5: The expansion (L) and the loss in modulus of elasticity ($\Delta E/E$), are represented on the abscissa, and the number of cycles (Z) is represented on the ordinate. Figure 5 shows that the concrete GO has a very poor durability (triangle C0D, durability factor ≤50 percent, and the concretes G, WR, F, B, and P have a very high durability (triangle A0B, durability factor >80 percent).

Table 5. Comparison of flexural and compressive strength (EMPA tests).

	Flexural Strength							Compressive Strength				
	After 28 d			After 300 Cycles				After 28 d		After 300 Frost Cycles		
Designation	Volume Weight (kg/dm³)	Strength (kg/cm²)	Mean Value (kg/cm²)	Volume Weight (kg/dm³)	Strength (kg/cm²)	Mean Value (kg/cm²)	Loss (%)	Strength (kg/cm²)	Mean Value (kg/cm²)	Strength (kg/cm²)	Mean Value (kg/cm²)	Loss (%)
GO	2.40	70						441				
	2.38	71	70.5	—	—	—	—	448	444.5	—	—	—
G	2.34	71		2.36	71			410		530		
	2.34	69	70	2.35	58	63	-10	413	411.5	548	535.7	+30
				2.34	62					529		
WG	2.37	68		2.35	69			417		544		
	2.36	79	73	2.36	63	66	-10	410	413	548	544	+31
				2.35	68					540		
WR	2.35	74		2.36	63			410		537		
	2.34	75	74.5	2.36	67	64.7	-14	406	408	537	532.3	+30.5
				2.36	64					523		
F	2.33	73		2.35	61			379		534		
	2.33	73	73	2.36	64	63.6	-13	386	382.5	506	527	+37
				2.36	66					541		
B	2.38	80		2.39	73			410		550		
	2.38	76	78	2.40	72	73.7	-5	410	410	530	540	+32
				2.38	76					541		
P	2.31	74		2.33	63			558		496		
	2.32	76	75	2.33	58	61.7	-18	368	363	482	489	+35
				2.33	64					490		

Table 6. Comparison of results from EMPA frost test and LPM frost-salt test after 300 cycles.

				EMPA Tests				Linear Expansion of Water-Stored Specimen (%)		LPM Tests		
	E-Modulus (t/cm²)			Linear Expansion (%)		Frost Durability (%)				Linear Expansion (%)		
Designation	0 Cycles	300 Cycles	Change (%)	Range	Avg	In Relation to Modulus	In Relation to Expansion	Range	Avg	Range	Avg	Frost-Salt Durability (%)
GO	410	171	-58	+0.65[a]	+0.83[a]	3	1	+0.02[b]	+0.03[b]	+0.95[c]	+0.69	2
	406	184	-55	+0.92[a]				+0.04[b]		+1.05[c]		
				+0.90[a]						+0.26[c]		
										+0.53[c]		
G	363	355	-2	+0.41	+0.48	101	84	+0.02	+0.03	+0.71	+0.40	88
	376	395	+5	+0.55				+0.03		+0.27		
				+0.46						+0.21		
										+0.40		
WG	369	388	+5	+0.45	+0.42	104	87	+0.04	+0.05	+0.18	+0.22	98
	378	388	+3	+0.40				+0.06		+0.18		
				+0.41						+0.31		
										+0.20		
WR	378	406	+7	+0.41	+0.38	105	90	+0.03	+0.04	+0.23	+0.26	96
	393	403	+3	+0.38				+0.05		+0.43		
				+0.36						+0.21		
										+0.18		
F	366	373	+2	+0.35	+0.36	103	91	+0.03	+0.04	+0.08	+0.17	101
	375	393	+5	+0.38				+0.04		+0.36		
				+0.36						+0.13		
										+0.11		
B	382	382	±0	+0.32	+0.37	102	90	+0.02	+0.04	+0.22	+0.31	93
	396	413	+4	+0.41				+0.06		+0.19		
				+0.36						+0.27		
										+0.56		
P	359	361	±0	+0.34	+0.38	101	90	+0.05	+0.05	+0.20	+0.26	96
	351	362	+3	+0.48				+0.05		+0.16		
				+0.32						+0.25		
										+0.41		

[a]After 20 cycles. [b]After storage in water (corresponding to 10 cycles). [c]After 10 cycles.

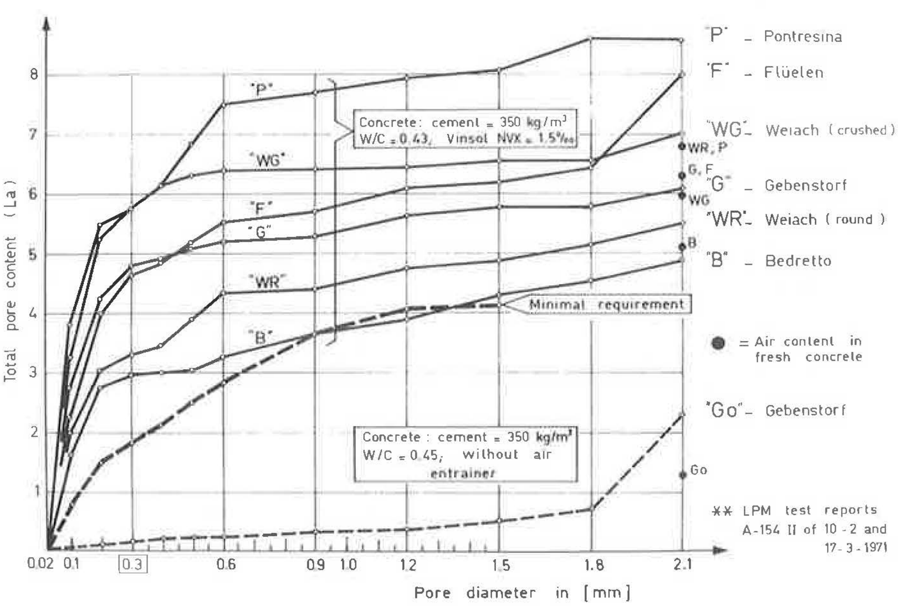


Figure 3. Summation curves of pore contents for tested concretes.

Figure 4. Pore analysis and characteristic values for frost-salt resistance.

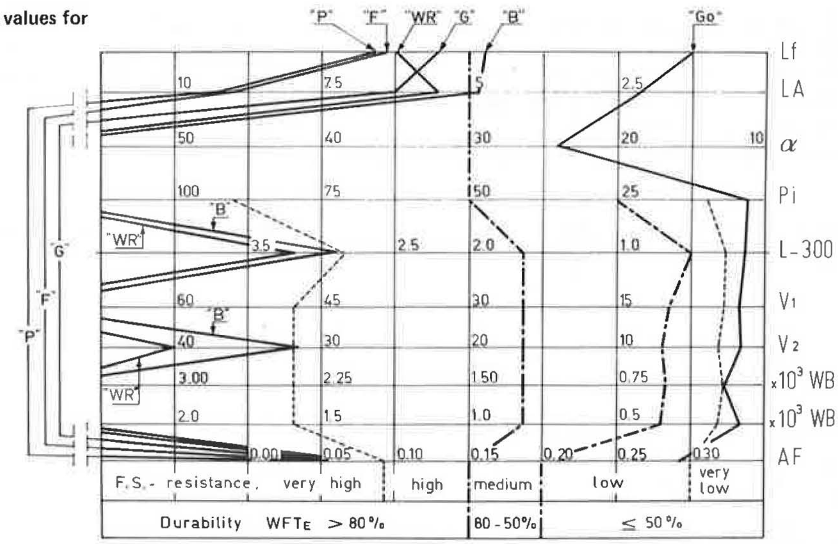


Table 7. Characteristic values of concretes tested.

Designa-tion	Lf	La	mm^{-1}	Pi	L_{300} (%)	V_1	V_2	WB_1 (10^3)	WB_2 (10^3)	AF
GO	1.4	2.3	24	3	0.1	3	1	0.2	0.1	0.28
G	5.3	6.1	91	402	4.8	208	64	14.56	4.48	0.047
WR	5.9	5.5	83	292	3.3	103	39	7.62	2.89	0.054
F	6.1	8.0	79	332	4.7	179	60	13.07	4.38	0.042
B	4.7	4.9	81	239	2.9	78	32	6.08	2.50	0.052
P	6.3	8.6	86	476	5.8	157	65	11.78	4.88	0.032

Note: Lf = fresh concrete/air volume, percent; La = hardened concrete/air volume, percent; α = mean specific pore surface, mm^{-1}; Pi = number of pores per mm^3 of sample; L_{300} = volume of pores up to 300 micron diameter, percent; $V_1 = (L_{300}/K_5) \cdot 100$, where $K_5 = A_5$ - gel water; $V_2 = (L_{300}/A_5) \cdot 100$; WB_1 = supplementary value = $V_1\,\sigma$ kg/cm², where σ = flexural strength at 28 days; WB_2 = supplementary value = $V_2\,\sigma$ kg/cm²; and AF = spacing factor in mm (after Powers).

Figure 5. Results of frost-salt test.

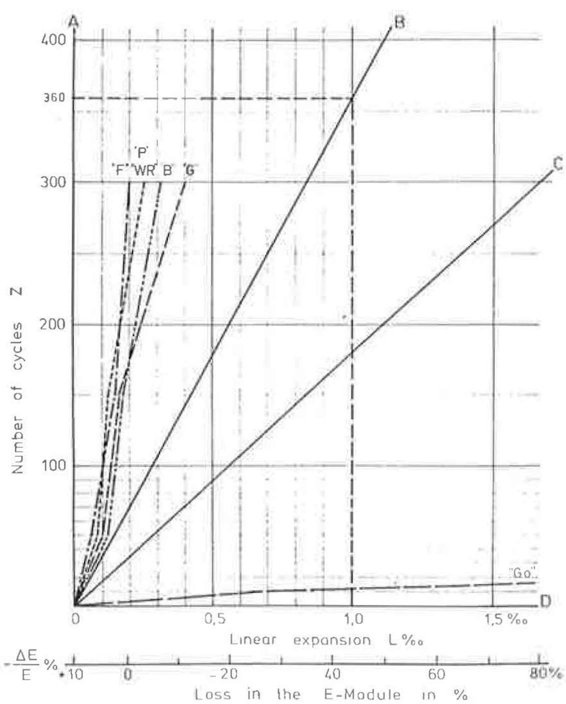
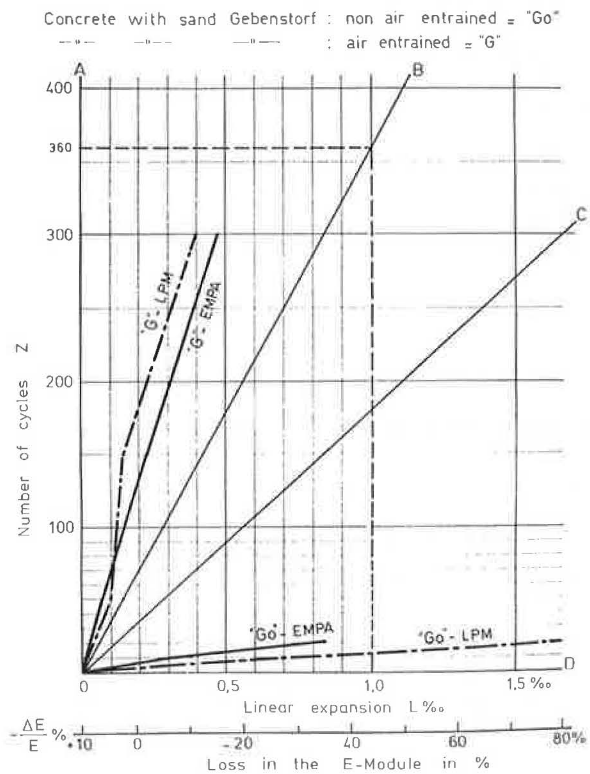


Figure 6. Results of tests for G- and GO-concrete.



CONVENTIONAL FROST TEST

Conventional tests were done in accordance with SIA-162 specifications (frost at -25° C and thaw at +14° C). Specimens of 12 by 12 by 36 cm in size were used. In addition to the modulus of elasticity, measurements were taken of the loss of flexural and compressive strengths and the linear expansion in relation to the number of cycles. The relation between flexural and compressive strengths, measured after 300 frost-thaw cycles and after 28 and 197 d respectively, is shown in Table 5. The air-entrained concretes show an average reduction of -10 percent in flexural strength and an average increase of approximately +32 percent in compressive strength. These results are not fully accurate since water-stored specimens were not used. The reduction in the modulus of elasticity during the EMPA frost test and the corresponding linear expansion during the EMPA and LPM tests are given in Table 6.

In the conventional EMPA frost test, the durability calculated from the linear expansion (4) and the durability actually obtained are in close agreement. A comparison of the data given in Table 6 shows a good correlation between EMPA test results (after 300 cycles) and those of the LPM frost-salt tests (Table 6).

Figure 7. Results of frost tests for WG-concrete.

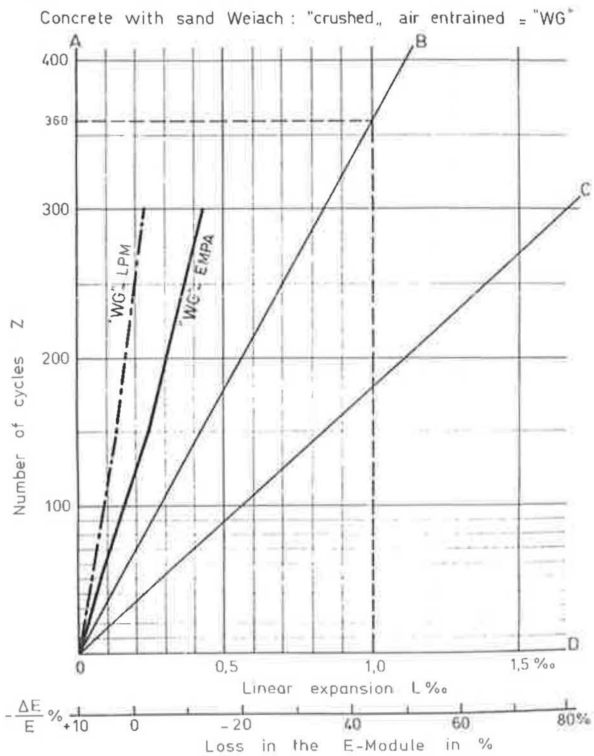


Figure 9. Results of frost tests for F-concrete.

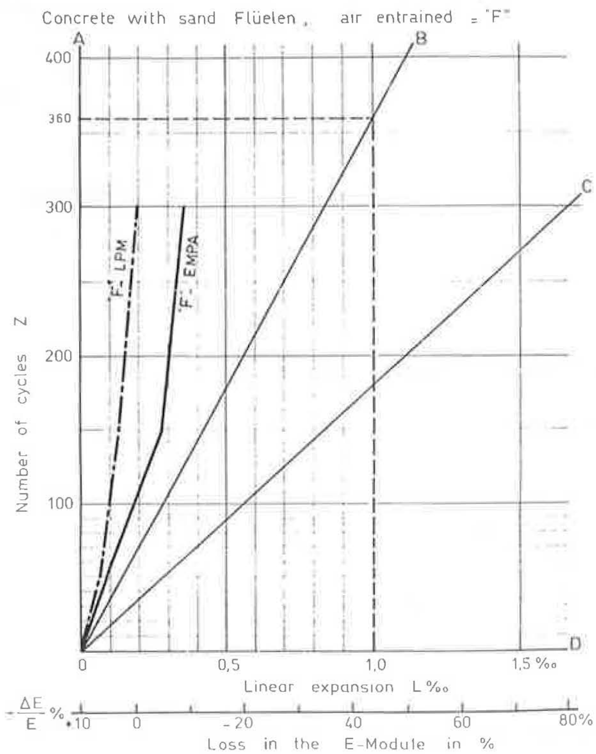


Figure 8. Results of frost tests for WR-concrete.

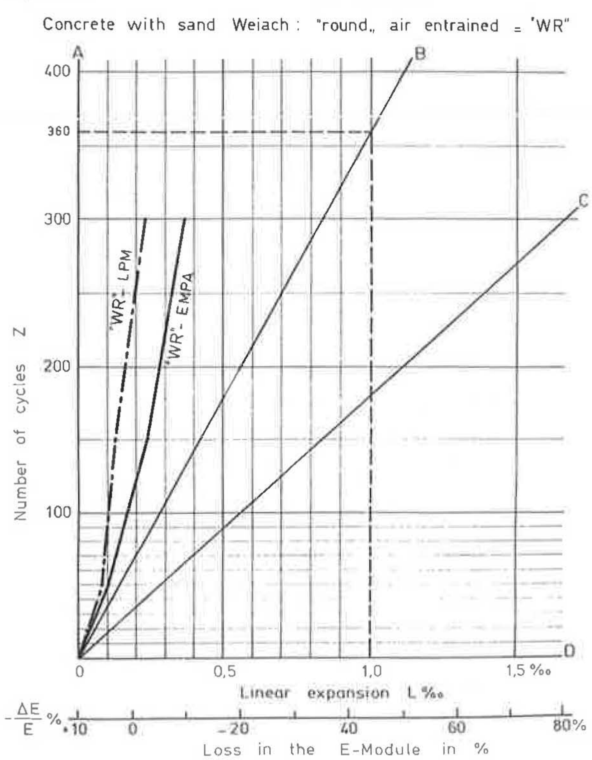


Figure 10. Results of frost tests for B-concrete.

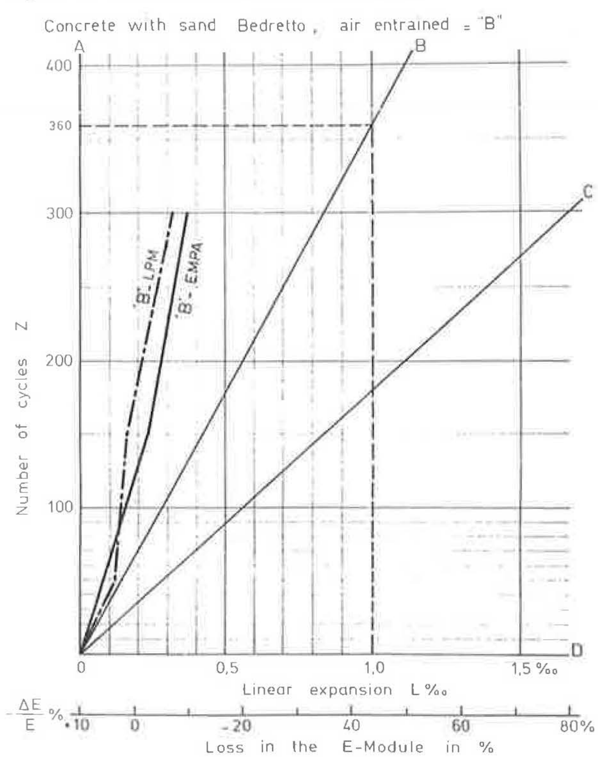
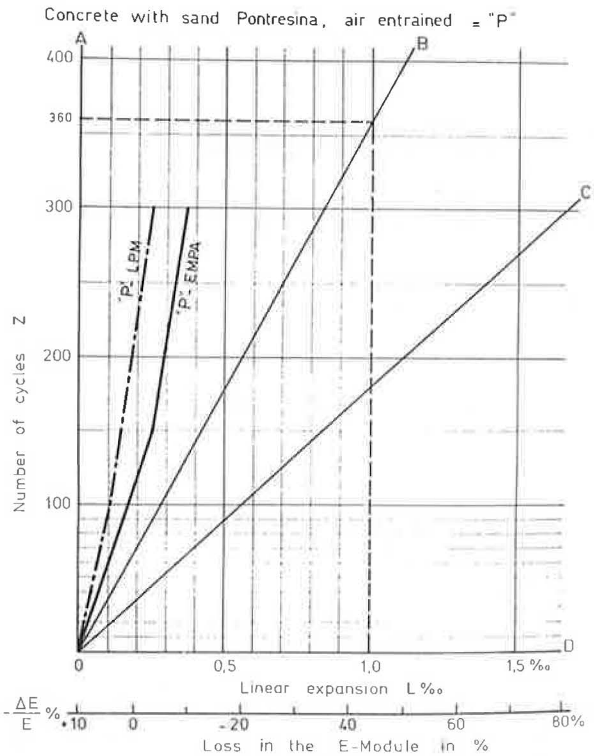


Figure 11. Results of frost tests for P-concrete.



CORRELATION BETWEEN THE METHODS

The correlation between the conventional frost test and the D-R (LPM) frost test is given by the linear expansion in relation to the number of cycles for all tested concretes and is shown in Figures 6 through 11. This correlation applies to concretes with very poor durability (GO) or to very good, air-entrained concretes (the six tested) without morphological disturbances and of a very high durability. For concretes with morphological disturbances and of medium durability another relation should exist.

CONCLUSIONS

1. The amount of fine aggregates in the concretes G, WG, and B, which exceeded the admissible limits of the Swiss specifications, did not affect the durability of the air-entrained concretes. These concretes resisted 300 cycles and had an average linear expansion of +0.41 percent. They also had an increase in the modulus of elasticity of +3 percent when tested by the conventional frost method and an expansion of +0.3 percent when tested by the frost-salt test (D-R method). Moreover the concrete made of B-sand was rejected since it was not frost resistant and therefore not suitable for high-quality concrete.

2. Non-air-entrained concrete that was made with GO-sand and a high content of fines failed after 20 cycles. This concrete showed a loss of elasticity modulus of -56 percent and an expansion of +0.83 percent and +2.25 percent when tested by the conventional frost method and the D-R frost-salt method respectively.

3. The air-entrained concretes of F and P were the only admissible concretes according to the Swiss specifications. These showed the same durability as the above-mentioned air-entrained concretes. After 300 cycles, these concretes had an increase of modulus of elasticity of +2 percent and an average expansion of +0.37 and +0.22 percent when submitted to the conventional frost test and D-R test respectively.

4. This investigation shows no decrease of durability in air-entrained concrete if the amount of fine aggregate exceeded the permissible limits of the Swiss specifications.

5. A very good correlation between the conventional frost test and the frost-salt test was obtained.

ACKNOWLEDGMENTS

The study was authorized by the Betonstrassen AG and the Association of Swiss Cement, Lime, and Plaster Industries (VSZKGF) and was carried out by the Swiss Federal Materials Testing Institute (EMPA) and the Laboratory of Preparation and Methods (LPM Laboratory).

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Basic Techniques and Examples of Computer Simulation

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Computer simulation is one of the most useful techniques available to the statistician. However, current literature indicates that this approach has not been widely used by engineers. This paper provides the basic tools for anyone with an elementary knowledge of statistical theory and computer programming to simulate a random normal process on a digital computer. The usefulness of this technique is illustrated by two examples that involve concrete strength specifications. The first deals with the determination of the best sampling plan for a quality-assurance testing program, and the second illustrates the simulation of a statistical acceptance procedure with a reduced pay schedule. Several FORTRAN subroutines are described for generating both uniform and normal random numbers, sorting a data array, selecting a random sample, calculating statistical parameters, and printing a histogram.

In recent years, there has been an increasing tendency to use statistical analysis to develop end-result specifications. Based on information obtained from a random sample, the product in question may be accepted, rejected, or accepted with a reduced payment. Since specifications control the ultimate quality of the product and determine the reimbursement to the contractor, it is important that they are well designed.

A useful tool for developing specifications of this type is computer simulation. Because it is basically a try-and-see approach, computer simulation enables the specification writer to see how a new specification can be expected to perform in the field. It is then possible to spot potential problems and correct them before they lead to more serious difficulties after the specification is adopted. In this manner, specifications can be written that are not only more effective but also equitable to all parties concerned.

REQUIREMENTS FOR COMPUTER SIMULATION

For the effective use of computer simulation, certain requirements must be met. An obviously basic requirement is a high-speed digital computer. And, if at all possible, the programming should be done from a communications terminal on a time-sharing basis that allows the user to interact directly with the computer.

Although work of this type could be done by an engineer and a programmer working together, it is more efficient if done by one individual who has an understanding of the process to be simulated and the ability to write the simulation program. A college level course in both statistics and programming should be sufficient to understand the material described in this paper.

Once the equipment and manpower requirements are met for the simulation of a real process, it is necessary to obtain some basic information that describes the variability of the real process. This information may be obtained from a variety of sources such as historical data, literature searches, preliminary field studies, or other simulations. The reliability of any simulation depends on the accuracy of the input parameters, so care must be taken to ensure that they are well defined.

The random generators used in this paper will generate either uniform or normal random numbers and, consequently, may be used only if these distributions are appropriate. Fortunately, these two distributions are suitable for most situations. The uniform distribution is used for random sampling, and the normal distribution is used for approximating the variability of most construction parameters. For those instances in which it is known that the actual distribution is not normal, the central limit theorem may have to be relied on (sample means from any population tend to be normally distributed). If necessary, another alternative is to use a transformation (such as logarithm, power, or root) to convert a skewed distribution to one that is approximately normal.

ADVANTAGES OF COMPUTER SIMULATION

Many advantages are derived from computer simulation. Of prime importance are the considerable savings in both time and expense. Programs that simulate a typical construction specification require only a day or two to write and a matter of seconds to run on the computer. In

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this manner, information based on the equivalent of several months or years of field data can be obtained in less than a week.

Of possibly greater importance is the use of computer simulation to solve problems for which direct analytical solutions are not known. For example, consider the expression Q = (XY)/Z in which X, Y, and Z are independent random variables, and the means and standard deviations of the variables are known. To determine the mean and standard deviation of the variable Q, some type of approximation must be used. To solve this problem by computer simulation, one would simply generate random values for X, Y, and Z and calculate Q; repeat this process many times (perhaps 1000); and store the Q values in the memory of the computer. Then, it is possible to compute the mean and standard deviation of the simulated Q values and, if desired, to print a histogram for checking the shape of the Q distribution.

Another example that is difficult to analyze by conventional means is the performance of a statistical acceptance procedure with a reduced pay schedule. In this case, computer simulation can be used to determine the distribution of pay factors resulting from any selected quality level of production. By averaging these simulated pay factors, an overall expected pay factor can be determined. The specification writer can then develop a realistic pay schedule that will provide payment commensurate with the quality received.

To assist in the study of various sampling plans, further advantages can be gained by using computer simulation. Since the simulated data are stored in the memory of the computer, it is possible to test different sampling plans on the same set of data. This technique is usually impossible with field tests because the sampling and testing process disturbs or destroys the material being tested. Also, the true mean value is never known in actual field tests and must be estimated. Because of the nature of the simulation process, the true mean is always known. Thus, it is possible to check the difference between the true mean and the sample mean in the simulation and to decide which of the several sampling plans is most accurate. Finally, if different acceptance procedures are being tested, the number of correct and incorrect decisions can be counted to determine which procedure is most often correct.

USE OF RANDOM GENERATORS

Random numbers that are generated by digital computers are not truly random because many numbers generated in this manner will eventually repeat in a sequence. However, random generators with long cycles have been developed and are satisfactory for most purposes (1).

Two subroutines, RAND and NORM, are used for simulation purposes (2). RAND generates uniform random numbers between the values of 0 and +1, and NORM generates normal random numbers with a mean of 0 and a standard deviation of 1. For both of these subroutines, a seed number must be transmitted from the main program each time the subroutine is called. In the examples presented here, the seed number is referred to as RANDSTART outside the computer program and is assigned the FORTRAN variable name NSTART within the program. Initially, NSTART should be an odd integer that does not exceed nine digits. NSTART is read into the main program once and it is automatically changed each time one of the subroutines is called. Typical coding for reading NSTART from a time-sharing computer terminal might be as follows:

```
      WRITE (6,100)
100   FORMAT ('0', 'ENTER RANDSTART')
      READ(5,110) NSTART
110   FORMAT(I9)
```

A free read feature is more convenient than the formatted read statement shown above.

Occasionally, the simulation will be such that it is desired to use both RAND and NORM in the main program. In this situation, a single NSTART term is sufficient for the independent operation of both random generators.

The random generators can be used in several ways. For example, it may be desired to have a simulated event occur randomly a certain percentage of the time, say, 15 percent. Since the numbers generated by RAND are uniformly distributed between 0 and +1, approximately 15 percent of them will have a value of 0.15 or less. If RUX is defined as the uniform random number and GO TO 1 is the event we want to occur 15 percent of the time, the coding for this operation might be as follows:

```
      CALL RAND(NSTART, RUX)
      IF(RUX.LE.0.15) GO TO 1
```

Then, at label 1 in the program, we would have additional FORTRAN statements that simulate whatever random event is desired.

The primary use of RAND concerns random sampling. If the variable SAMPLE represents a single random sample that is selected from the values stored in array X(N), the following coding might be used:

```
      CALL RAND(NSTART,RUX)
      IF(RUX.EQ.1.) RUX=.999999
      I=IFIX(RUX*N)+1
      SAMPLE=X(I)
```

In actual practice, it is more convenient to use subroutine SAMP (2) which is capable of selecting a random sample of up to 1000 from an array of up to 1 000 000.

When subroutine NORM is used, it is desired to generate values that have some specified mean and standard deviation. The term VALUE is defined as the random normal variable that has a mean and standard deviation of AVG and STDV respectively. Subroutine NORM returns two independent normal variates, RNX and RNY, each time it is called; therefore, a total of N of these values could be generated by the following coding, if N is assumed to be an even number.

```
      NHALF=N/2
      DO 1 I=1,NHALF
      CALL NORM(NSTART,RNX,RNY)
      VALUE(I)=AVG+STDV*RNX
    1 VALUE(NHALF+I)=AVG+STDV*RNY
```

In those rare cases in which it is inconvenient to use more than one random number each time the subroutine is called, the second value returned by NORM may simply be ignored.

SIMULATION TO DETERMINE BEST SAMPLING PLAN

For many years, it has been the practice of the New Jersey Department of Transportation to make three compression test cylinders for every 76.5 m^3 (100 yd^3) of concrete delivered to the job. Although it was not specifically required, it had become common practice to take the three cylinders from the same truck. The vast majority of the concrete was truck-mix (as opposed to central-mix), and ample data were available that indicated a great deal of variability in the strength of concrete from truck to truck (batch to batch). Because of the large degree of batch-to-batch variability, it was theorized that the sampling plan would be more effective if the three cylinders were taken from separate trucks. It was further predicted that a more accurate measure of the lot quality of the concrete would be provided by only two cylinders taken from different trucks than by three cylinders taken from the same truck.

To test these conclusions, a computer program was written to simulate the three sampling plans for many lots of concrete of varying average strengths. The best sampling plan will be the one that produces a sample mean that (a) deviates the least from the true lot mean and (b) overestimates as often as it underestimates the true lot mean. In statistical terms, this is a minimum-variance, unbiased estimator.

In the simulation of a measurement process, it is usually necessary to separate the total variance of the field data into the component associated with the measurement process itself and the component represented by the true variability of the material being measured. The variability of the measurement process is determined from repeated measurements of material from the same batch. The variability of the material is determined by subtracting the variance of the measurement process from the variance of the measurements made on many different batches of material. The measurements of many sets of cylinders taken from single trucks provided a pooled estimate of variance for the measurement process. The measurements of single cylinders selected randomly from different trucks were used to calculate an overall variance from which the measurement process variance could be subtracted to obtain the true batch-to-batch variability of the concrete.

The general techniques previously described are the basic building blocks of the simulation program. After the initial parameters are entered, it is necessary to have the computer create a large population of strengths for batches of concrete similar to the strengths occurring during construction. Subroutine NORM randomly generates the strengths by using the standard deviation that represents only the variability of the material. The batches are then divided into lots, and subroutine SAMP samples the lots as prescribed for the sampling plans being tested. An additional random component of variability that represents the sampling-and-testing error (i.e., variability of the measurement process) is added by using subroutine NORM to each simulated cylinder strength. Since the true strength of each simulated batch is known, the true mean of each lot can be calculated and compared to the mean estimated from the sample. For each lot and each sampling plan, the algebraic difference between the sample mean and the true mean is calculated and stored. After the desired number of lots have been sampled, the algebraic differences are analyzed. For a good sampling plan, the distribution of the algebraic differences will have a mean close to zero and a low standard deviation.

The steps in this program are summarized as follows:

1. Enter into the computer desired number of lots, batches (trucks) per lot, mean strength of concrete, batch-to-batch standard deviation, sampling-and-testing standard deviation, and number to start random generator;
2. Calculate the total number of simulated batches required and generate this number of "true" concrete batch strengths (use mean strength and batch-to-batch standard deviation);
3. Divide the batch strengths into lots, calculate the true lot strength for each, print a histogram of all batch strengths, and count and print percentages of batch strengths and lot strengths that are below design strength;
4. In accordance with the provisions for each plan (three cylinders from the same truck, three cylinders from separate trucks, and two cylinders from separate trucks), randomly sample each lot (adding on the variability due to sampling-and-testing error), calculate the estimated strength for each lot, and store the estimated lot strengths in separate arrays for each plan; and
5. Compare the estimated lot strength for each plan with the true lot strengths to see which plan is best and calculate and print all statistical parameters of interest.

A typical printout of the simulation for determining the best sampling plan is shown in Figure 1. The entry variables STDV(BB) and STDV(ST) refer to the batch-to-batch and sampling-and-testing standard deviations respectively. The rest of the printout is self-explanatory and provides the information necessary to determine which sampling plan is performing best.

To thoroughly test the three sampling plans over a wide range of average strengths, 17 runs of 100 lots each were made for average strengths ranging from about 20.7 to 31.0 MPa (3000 to 4500 lbf/in^2). The lot size and truck size were assumed to be 76.5 m^3 (100 yd^3) and 7.65 m^3 (10 yd^3) respectively. These measurements gave a value of 10 trucks per lot to be entered into the program. The 17 runs represented the equivalent of sampling 130 000 m^3 (170 000 yd^3) of concrete for each of the three plans.

It was known that a high value for batch-to-batch variability (σ_{BB}) and a low value for sampling-and-testing variability (σ_{ST}) would accentuate the differences in the results obtained from the different sampling plans and would bias the simulation in favor of the predicted results. Therefore, a slightly low value of σ_{BB} = 4.14 MPa (600 lbf/in^2) and a slightly high value of σ_{ST} = 2.07 MPa (300 lbf/in^2) were used in the program. As an additional check, a few runs were made with different values, and the results were essentially the same.

Table 1 gives the average differences from the true mean lot strengths for each plan and each run, the standard deviations for these differences, and the t-values for the average differences. In addition, the overall average and pooled standard deviation are given for each sampling plan.

Plan 1 and plan 2 are considered unbiased because none of the average differences is significantly different from zero. Plan 3 may be slightly biased because 3 of the 17 differences are significant at the 95 percent confidence level and have the same algebraic sign. Although there is no reason to expect that this plan is biased, since the likelihood that as many as three significant t-values with the same sign would be obtained is fairly slim (about 0.016). However, from a practical standpoint, the degree of bias is small and can probably be ignored.

The pooled standard deviations associated with the three plans are quite different because the distributions of the individual standard deviations are separate from one another. Plan 2 (three cylinders from separate trucks) is the best, plan 1 (three cylinders from the same truck) is the worst, and plan 3 (two cylinders from separate trucks) is nearly better than plan 1.

The data given in Tables 2 and 3 show the incorrect decisions that are tabulated and averaged for each plan. Although all three plans seem to have about the same percentage of incorrect decisions for individual cylinders, the lot strength results serve to confirm the conclusions obtained from the data given in Table 1. That is, plan 2 is the best because it has the fewest incorrect decisions; plan 1 is the worst because it exhibits the greatest number of incorrect decisions; and plan 3 falls between plans 1 and 2.

The computer time for this simulation was approximately 11 s for each 100 lot run. The writing of and adjustments to this program required about 5 person-days and 400 s of running time on the computer.

SIMULATION OF A STATISTICAL ACCEPTANCE PROCEDURE

To obtain a greater degree of compliance with the concrete specification, a statistical acceptance procedure with a reduced pay schedule was considered. The procedure is based on the range method of Military Standard 144 (3). Lot sizes and sample sizes are specified for each class of concrete and a quality index (Q_L) is calculated for each lot as follows:

$$Q_L = (\overline{X} - \text{design strength})/R \qquad (1)$$

where

$\overline{X}$ = sample average = $(X_1 + X_2 + \ldots + X_N)/N$,
N = sample size,
X = compressive strength, and
R = sample range = $X_{maximum} - X_{minimum}$.

Based on historical data for structural concrete, the percentages for acceptable quality level (AQL) and rejectable quality level (RQL) were determined to be 10 and 43 percent below design strength respectively. In other words, if no more than 10 percent of a lot of concrete is below the design strength, the concrete is at least equal to the quality obtained in the past and is, therefore, considered acceptable. On the other hand, if 43 percent or more of the lot is defective, the concrete is considered seriously deficient and would warrant no payment or would possibly have to be removed and replaced.

The Military Standard 144 (3) provides a procedure for determining critical Q_L limits that indicate when either the AQL or the RQL has been obtained. These levels are used to set Q_L ranges that correspond to a graduated series of pay factors. A set of ranges that might be used for sample sizes of 7 and 4 is as follows:

Q_L Range		
N = 7	N = 4	Pay Factor (%)
≥0.28	≥0.38	100
0.22 to 0.27	0.30 to 0.37	95
0.17 to 0.21	0.23 to 0.29	90
0.12 to 0.16	0.16 to 0.22	80
0.07 to 0.11	0.09 to 0.15	50
≤0.06	≤0.08	0

For any sample size, the largest Q_L value corresponds to the AQL and a pay factor of 100 percent, and the lowest Q_L value corresponds to the RQL and a pay factor of zero. Between these two extremes, there are four intermediate pay levels. These pay levels have been weighted toward the high (100 percent) end of the pay

Figure 1. Printout of simulation to determine best sampling plan.

```
ENTER LOTS, TRUCKS/LOT, MEAN, STDV(BB), STDV(ST), RANDSTAPT

      100        10       3500     600        300       9999977

DISTRIBUTION OF SIMULATED CONCRETE BATCH STRENGTHS

 UCL     FREQUENCY
1869       4 X
2091      11 XXXX
2314      15 XXXXX
2537      43 XXXXXXXXXXXXXX
2759      47 XXXXXXXXXXXXXXXX
2982      75 XXXXXXXXXXXXXXXXXXXXXXXXX
3205     137 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
3427     145 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
3650     150 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
3873     118 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
4095     112 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
4318      71 XXXXXXXXXXXXXXXXXXXXXXXX
4541      34 XXXXXXXXXXX
4763      21 XXXXXXX
4986      12 XXXX
5209       2 X
5431       3 X

MEAN = 3466          STD. DEV. =  604          STD. ERR. = 19
TOTAL OF 1000 BATCHES, 20.3 PERCENT BELOW THE DESIGN STRENGTH OF 3000
TOTAL OF 100 LOT AVERAGES,  0.0 PERCENT BELOW DESIGN STRENGTH

PERFORMANCE OF PLAN 1 (THREE CYLINDERS PER LOT FROM SAME TRUCK)

   MEAN DEVIATION OF TEST FROM TRUE LOT AVERAGE =   5
   STANDARD DEVIATION = 627     T-VALUE FOR PLAN MEAN =  0.08
   INDIVIDUAL CYLINDERS BELOW DESIGN STRENGTH = 24.3 PERCENT
   TESTS (LOT AVERAGES) BELOW DESIGN STRENGTH = 23.0 PERCENT
   CYLINDERS FALSELY REJECTED =  8.3 PERCENT
   CYLINDERS FALSELY ACCEPTED =  6.0 PERCENT
   LOTS FALSELY REJECTED = 23.0 PERCENT
   LOTS FALSELY ACCEPTED =  0.0 PERCENT

PERFORMANCE OF PLAN 2 (THREE CYLINDERS PER LOT FROM SEPARATE TRUCKS)

   MEAN DEVIATION OF TEST FROM TRUE LOT AVERAGE =  13
   STANDARD DEVIATION = 348     T-VALUE FOR PLAN MEAN =  0.37
   INDIVIDUAL CYLINDERS BELOW DESIGN STRENGTH = 21.7 PERCENT
   TESTS (LOT AVERAGES) BELOW DESIGN STRENGTH = 11.0 PERCENT
   CYLINDERS FALSELY REJECTED =  5.7 PERCENT
   CYLINDERS FALSELY ACCEPTED =  4.7 PERCENT
   LOTS FALSELY REJECTED = 11.0 PERCENT
   LOTS FALSELY ACCEPTED =  0.0 PERCENT

PERFORMANCE OF PLAN 3 (TWO CYLINDERS PER LOT FROM SEPARATE TRUCKS)

   MEAN DEVIATION OF TEST FROM TRUE LOT AVERAGE =   8
   STANDARD DEVIATION = 477     T-VALUE FOR PLAN MEAN =  0.17
   INDIVIDUAL CYLINDERS BELOW DESIGN STRENGTH = 25.0 PERCENT
   TESTS (LOT AVERAGES) BELOW DESIGN STRENGTH = 19.0 PERCENT
   CYLINDERS FALSELY REJECTED =  7.0 PERCENT
   CYLINDERS FALSELY ACCEPTED =  3.0 PERCENT
   LOTS FALSELY REJECTED = 19.0 PERCENT
   LOTS FALSELY ACCEPTED =  0.0 PERCENT
```

Table 1. Results of simulation to determine best sampling plan.

Mean Strength of 100 Lots (MPa)	Plan 1 Avg Difference	Plan 1 Standard Deviation	Plan 1 t-Values	Plan 2 Avg Difference	Plan 2 Standard Deviation	Plan 2 t-Values	Plan 3 Avg Difference	Plan 3 Standard Deviation	Plan 3 t-Values
20.65	-0.26	3.85	-0.69	0.29	2.31	1.24	0.37	2.70	-1.38
21.33	-0.66	4.38	-1.51	-0.12	2.40	-0.49	0.50	3.20	1.59
22.26	0.44	3.85	1.14	0.03	2.32	0.12	0.03	2.91	0.10
22.56	-0.65	4.27	-1.52	0.04	2.48	0.17	0.52	3.18	1.65
23.37	0.55	4.47	1.23	0	2.56	0.0	0.49	3.03	1.65
23.90	0.03	4.32	0.08	0.09	2.40	0.37	0.06	3.29	0.17
24.15	-0.15	4.36	-0.35	-0.17	2.53	-0.65	-0.35	3.06	-1.16
24.77	-0.34	4.03	-0.84	0.20	2.52	0.78	-0.24	3.14	-0.76
25.59	0.27	3.99	0.67	0.16	2.50	0.64	0.81	2.90	2.79[a]
26.17	0.61	4.16	1.47	-0.16	2.18	-0.72	0.37	3.07	1.23
26.94	0.23	4.36	0.52	-0.10	2.38	-0.40	0.12	3.10	0.38
27.51	0.28	4.44	0.63	0.23	2.27	1.00	-0.18	2.83	-0.63
28.39	0.19	3.78	0.49	-0.16	2.17	-0.72	0.64	2.65	2.45[a]
28.96	0.43	4.21	1.02	0.13	2.50	0.53	0.46	2.96	1.56
29.52	0.21	4.26	0.48	0.19	2.32	0.82	-0.12	2.76	-0.42
30.64	-0.03	4.12	-0.08	0.17	2.30	0.73	0.60	2.63	2.29[a]
31.14	-0.21	4.12	-0.52	0.12	2.45	0.50	0.08	3.16	0.24
Avg	0.055			0.055			0.245		
Pooled		4.18			2.39			2.98	

Note: 1 Pa = 0.000 145 lbf/in^2.

[a]Significantly different from zero at 95 percent confidence level.

Table 2. Incorrect decisions for individual cylinders.

Mean Strength of 100 Lots (MPa)	Plan 1 (%) Rejected	Plan 1 (%) Accepted	Plan 2 (%) Rejected	Plan 2 (%) Accepted	Plan 3 (%) Rejected	Plan 3 (%) Accepted
20.65	7.0	8.7	9.3	8.7	8.0	10.5
21.33	8.7	5.0	8.0	5.0	7.0	8.0
22.26	6.3	5.3	7.0	6.7	10.5	7.0
22.56	6.3	7.3	8.0	4.0	6.5	5.0
23.37	6.3	3.7	5.0	4.3	6.0	3.0
23.90	8.3	6.0	5.7	4.7	7.0	3.0
24.15	6.0	4.3	5.7	3.7	6.0	3.0
24.77	9.7	2.3	5.7	4.0	9.0	1.5
25.59	5.0	0.3	4.0	1.3	5.5	2.5
26.17	2.3	1.3	3.7	2.3	4.5	2.5
26.94	2.3	0.7	4.7	1.0	2.0	0.5
27.51	3.0	1.7	1.7	1.7	2.5	1.5
28.39	3.7	0.3	2.3	0.7	2.0	1.0
28.96	2.3	0.3	2.7	0.3	2.5	1.0
29.52	0.3	0	0	0	0.5	1.0
30.64	1.7	0.7	0.3	0.3	1.0	0.5
31.14	2.7	0	0.7	0	1.5	0
Avg	4.8	2.8	4.4	2.9	4.8	3.0

Note: 1 Pa = 0.000 145 lbf/in^2.

Table 3. Incorrect decisions for lot strengths.

Mean Strength of 100 Lots (MPa)	Plan 1 (%) Rejected	Plan 1 (%) Accepted	Plan 2 (%) Rejected	Plan 2 (%) Accepted	Plan 3 (%) Rejected	Plan 3 (%) Accepted
20.65	24.0	17.0	16.0	19.0	18.0	11.0
21.33	35.0	12.0	24.0	14.0	23.0	9.0
22.26	28.0	6.0	23.0	3.0	21.0	3.0
22.56	37.0	3.0	19.0	1.0	19.0	5.0
23.37	24.0	0	14.0	0	12.0	0
23.90	23.0	0	11.0	0	19.0	0
24.15	24.0	0	13.0	0	18.0	0
24.77	20.0	0	7.0	0	14.0	0
25.59	9.0	0	2.0	0	4.0	0
26.17	8.0	0	1.0	0	6.0	0
26.94	6.0	0	2.0	0	3.0	0
27.51	6.0	0	1.0	0	1.0	0
28.39	2.0	0	0	0	0	0
28.96	2.0	0	0	0	0	0
29.52	3.0	0	0	0	0	0
30.64	0	0	0	0	0	0
31.14	1.0	0	0	0	1.0	0
Avg	14.8	2.2	7.8	2.2	9.4	1.6

Note: 1 Pa = 0.000 145 lbf/in^2.

scale to ensure that the pay for satisfactory concrete would never be severely penalized.

To produce a consistent supply of AQL concrete or better, the average strength must be considerably higher than the design strength. For example, if the design strength is 25.5 MPa (3700 lbf/in^2) and the producer possesses a 15 percent coefficient of variation, the concrete must have an average strength of about 31.6 MPa (4580 lbf/in^2) to be rated AQL. A producer must attempt to achieve this overdesign value for all batches of concrete to assure that no more than 10 percent of the batches will fall below the design value of 25.5 MPa (3700 lbf/in^2).

The computer simulation for this example served three purposes. Primarily, it was a check to see if the specification was properly accepting lots of AQL and better and rejecting lots of RQL and poorer. It also emphasized the importance of using an adequate overdesign value when work is performed under this specification. And finally, it demonstrated that a contractor who overdesigned properly would, on the average, achieve a pay factor close to 100 percent.

This simulation program is similar to that of the first example except that two separate components of variance were not required. In this second example, the computer must generate a distribution of strength tests that have a specific mean and coefficient of variation. These parameters are used to determine the standard deviation and then subroutine NORM is used. A Q_L value is calculated for each lot. These values are then compared to the ranges previously given, which have been programmed into the memory of the computer, to determine the pay factor. The simulation also contains a minimum strength requirement for studying the effects of this provision.

The steps in this program are as follows:

1. Enter into the computer desired number of lots, mean strength of concrete, coefficient of variation, design strength, minimum allowable strength, number of tests per lot, and number to start random generator;
2. Calculate the total number of simulated strength tests required, generate this number of tests by using the mean strength and coefficient of variation, and print a histogram of these values;
3. Calculate and print the mean, standard deviation, and coefficient of variation of these tests;
4. Calculate and print the number of tests below the minimum level, the number of lots having one or more tests below the minimum level, and the overall percentage of material below design strength; and
5. Calculate the Q_L value and pay factor for each lot, print a table showing the distribution of lots by Q_L and pay factor, and calculate and print the overall expected pay factor for all lots.

A typical printout of a simulation of a statistical acceptance procedure is shown in Figure 2. The overall expected pay factor in the printout is simply the average pay factor for all lots. Table 4 gives the results of 20 runs made

Figure 2. Printout of simulation of statistical acceptance procedure.

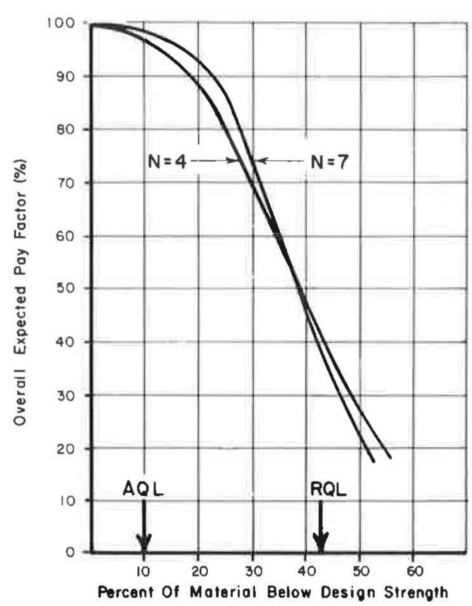
```
ENTER LOTS, MEAN, CV, DESIGN, MINIMUM, TESTS, RANDSTART

      100   4580  15   3700    2900      7     9999993

DISTRIBUTION OF SIMULATED CONCRETE STRENGTH TESTS

 UCL     FREQUENCY
2600     2 X
2879     1 X
3158     9 XXXX
3437    21 XXXXXXXXXX
3716    31 XXXXXXXXXXXXXX
3995    78 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
4274    79 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
4553   104 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
4832   108 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
5111   110 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
5390    74 XXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXX
5669    43 XXXXXXXXXXXXXXXXXXXX
5948    31 XXXXXXXXXXXXXX
6227     7 XXX
6505     1 X
6784     1 X

MEAN = 4595      STD. DEV. =  675      COEFF. OF VAR. = 14.7

TOTAL OF  700 TESTS WITH   4 BELOW MINIMUM LEVEL OF 2900

TOTAL OF 100 LOTS WITH  4 HAVING ONE OR MORE TESTS BELOW 2900

AMOUNT OF MATERIAL BELOW DESIGN STRENGTH =  9.1 PERCENT

     DISTRIBUTION BY PAY FACTOR OF SIMULATED QL VALUES

QL RANGE                  PAY FACTOR                  FREQUENCY
 >.27                        100                          91
.22-.27                       95                           2
.17-.21                       90                           4
.12-.16                       80                           1
.07-.11                       50                           1
 <.07                          0                           1

OVERALL EXPECTED PAY FACTOR FOR ALL LOTS =  97.8 PERCENT
```

Table 4. Results of simulation of statistical acceptance procedure.

Mean Strength (MPa)	Sample Size	Coefficient of Variation (%)	Material Below Design Strength (%)	Overall Expected Pay Factor (%)
34.23	7	21.1	10.9	98.3
31.68	7	14.7	9.1	97.8
29.65	7	16.1	20.3	93.3
27.77	7	15.0	30.6	71.7
26.92	7	20.2	40.1	44.8
26.48	7	19.6	42.0	44.3
26.20	7	14.3	42.3	38.0
26.07	7	15.2	44.0	36.8
25.29	7	15.1	51.6	19.8
34.55	4	20.4	9.5	96.5
31.85	4	14.6	8.8	97.6
29.54	4	16.0	20.5	85.3
27.83	4	15.3	30.5	68.6
27.01	4	15.2	37.0	54.0
26.82	4	19.3	40.8	48.0
26.68	4	20.2	40.8	47.6
26.60	4	15.9	39.5	46.8
26.42	4	15.0	42.0	41.3
26.17	4	15.5	43.8	44.3
25.24	4	15.6	52.5	21.0

Notes: 1 Pa = 0.000 145 lbf/in^2.
Design strength = 25.50 MPa; overdesign strength = 31.60 MPa at CV = 15 and 34.34 MPa at CV = 20.

Figure 3. Operating characteristic curves for statistical acceptance procedure.



for a single class of concrete that has a design strength of 25.5 MPa (3700 lbf/in^2).

The operating characteristic curves shown in Figure 3 were determined from the data given in Table 4. These curves show the relation between the percentage of material below design strength and the overall expected pay factor. For AQL concrete (10 percent of material below design strength), the expected pay factor should theoretically be 100 percent. In reality, it is approximately 98 and 97 percent for sample sizes of 7 and 4 respectively. At the other extreme, RQL concrete (43 percent of material below design strength) was intended to have an expected pay factor close to zero. However, because many rejectable lots actually receive partial payment, the overall expected pay factors at the RQL are approximately 36 and 41 percent for the two sample sizes. If the sample sizes remain unchanged, any attempt to lower the expected pay factors at the RQL by modifying the pay levels previously given will have the adverse effect of also lowering the expected pay factors at the AQL. It remains a matter of judgment to decide whether certain refinements should be made. The computer simulation has served its purpose by providing the information necessary to make these decisions.

The computer time for this simulation was approximately 5 s for each 100-lot run. About 3 person-days and 200 s of running time were required to develop and adjust this program.

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What Is a Quality-Assurance System?

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An overall approach to quality assurance that is practical and realistic is outlined. This approach takes into consideration and gives weight to the various factors that constitute quality. This overall or system approach has two main subsystems, namely, the technical or engineering subsystem and the nontechnical or social subsystem. The approach recognizes that for the technical subsystem the level of quality is set by the designer and is expressed in plans and specifications. Plans on the whole have served their purpose adequately; however, specifications have given rise to many problems in the execution of the work. Reasons and suggestions for avoiding such problems are given. The interactions among the various facets of the system that continually take place are discussed. A formalized basis for feedback is suggested so the experience gained can be used in future work.

Quality is defined as any predictable characteristic of a material, ingredient, product, or object. In engineering, the word by itself is practically meaningless. A specification for quality materials or quality workmanship means very little since quality to one person may not be quality to another. However, quality level is definable in engineering terms. Therefore, the word quality implies a quality level of some sort, and the contract document needs to define that quality level.

The quality level is something more than simply the quality of the physical materials that constitute a structure or project. The quality level is judged by how well the finished project or structure serves society—physically, functionally, emotionally, environmentally, and, of course, economically.

Quality assurance deals with obtaining the quality level of construction needed for a project to perform the functions intended and to do so within the various human, social, environmental, and economical requirements and limitations. Quality encompasses determining the needs and will of the people, political considerations, and social and environmental factors and how these interact to influence design, specifications, contractual relationships, production, quality control, sampling, testing, charting, inspection, decision making. The key to success for a quality-assurance system is quality-minded people at every step of the process or system—and this requires training. Without attitudes of positive quality at every step, it is impossible to achieve or assure quality no matter how good the design or specifications. Such attitudes are initially needed to develop the proper design and specifications.

QUALITY-ASSURANCE SYSTEM

Figure 1 (1) is a diagram of a quality-assurance system. The system is divided into two main subsystems: the technical and the nontechnical or social subsystems. The technical subsystem is represented by a polygon in the figure, while the nontechnical or social factors are listed in the center of the figure. Chronologically, the nontechnical or social activities usually precede the technical activities.

The nontechnical activities start with the needs and wishes of the people that are read and interpreted by politicians. Planners, architects, and lawyers then become involved. Engineers have usually not become involved in the nontechnical activities, but should do so because the success of their work depends to a large extent on how well they communicate in these early stages with the persons engaged in the other disciplines.

Once the work is assigned to the engineer, the designer sets the quality levels of the various parts of the project through plans and specifications. The contractor then makes an estimate and submits a bid. If the bid is realistic, one can expect a satisfactory quality level. However, if the contractor is desperate for work and bids unrealistically low, the foundation is laid for a troublesome job for which quality will be difficult to attain.

Another important facet of the system is the preparation and training necessary for the construction phase of both the owner and the contractor. What is expected of each person, and what special features of the project are different from similar features of other projects?

Control, sampling, testing, and acceptance during construction are the critical activities within the system. The finest designs and the most fair and equitable specifications can be completely thwarted if the designs and

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specifications are improperly implemented, or if weak designs and specifications cannot be salvaged and implemented into a satisfactory structure by an alert construction manager.

The relation and interaction are complex and vary from job to job so that they are difficult to analyze in relation to specific jobs. Therefore, this discussion focuses on the specifications and on the field work that deals with the training, production control, and acceptance of the physical elements used in the structure or project.

SPECIFICATIONS

An anonymous writer defined specifications as follows:

> A voluminous and painstakingly dry document designed to harass, hamper, and confuse the manufacturer or contractor, disturb the digestion and emotional stability of Congressmen, gnaw at the very foundations of democracy, and provide simultaneous discrimination against both big and little business. It is written as a masterpiece of incoherency by a man who never saw the commodities specified and for bidders who won't read it anyway. The item detailed bears no relationship to reality as it cannot be produced as specified and would be worse than worthless if it could. It is hopelessly incompatible with current production techniques, utilizes materials that are not available, was three years out of date when published, and it cost more to write than the items described therein.

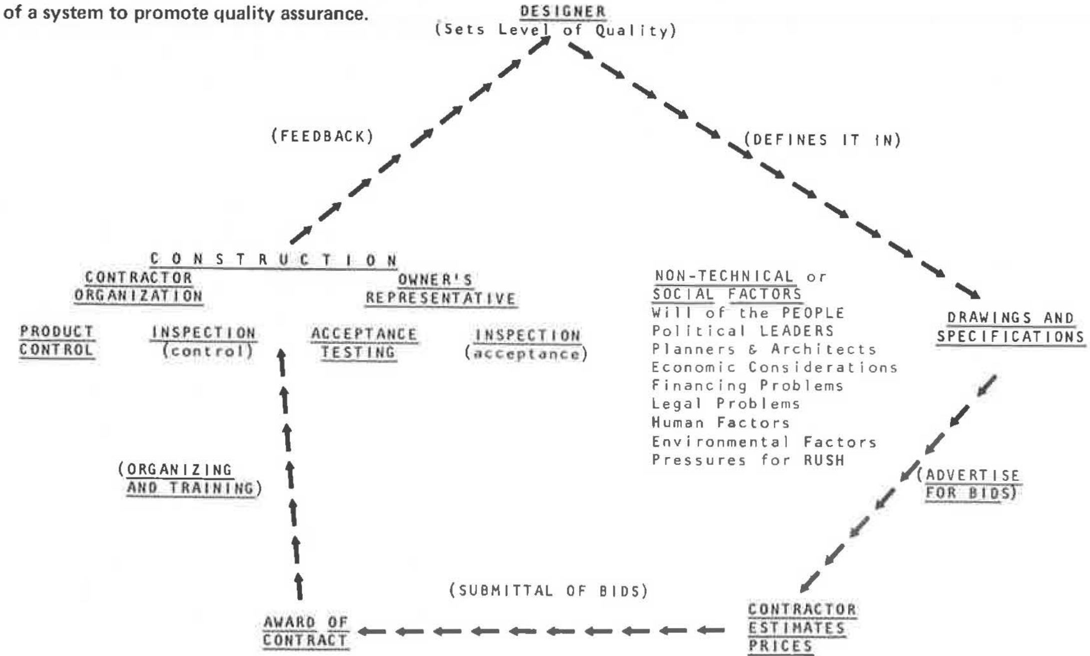
This definition may be exaggerated, but serves to emphasize that a specification should say what it means and mean what it says. Very few specifications meet that criterion, and this is why they have not served their purpose. Much of the bickering between engineer and contractor during construction and many of the claims after completion of the work can be traced to inadequate, unrealistic, or arbitrarily enforced specifications.

One of the main weaknesses of most specifications is that they are assembled mostly by copying from other documents that were previously used on other work, sometimes of a different type, in a distant geographical area. Most often, the writing of specifications is given to someone who is satisfied with sitting at a desk and writing or copying whatever looks satisfactory for a finished document. The writer is rarely acquainted with the significance or philosophy of the design, design criteria, field conditions, field procedures, and contractual relationships. Although the plans may be studied by the designer before they are signed and issued, the specifications are rarely seen or studied. The specifications are as important as the plans since what is on the plans is the result of a careful design. In the same manner, specifications should be designed just as carefully (2).

Another weakness is that, in the past and to an appreciable extent currently, specifications have mostly provided a sharp line of demarcation between passing or failing to pass. This is perhaps due to the engineer's educational background regarding precision and accuracy but is not realistic. For specifications to be realistic, tolerances must be provided. Under the circumstances, the best approach to acceptance is the probability approach. Everything known or observed in nature and in our environment is found to be definable, and its occurrence and variability can be predicted through the various mathematical probability laws or systems. It is fortunate that engineers can observe variations and can approximate, predict, or express them through normal distribution (3, 4). This distribution is represented graphically by a bell-shaped curve (the mathematical expressions are found in many statistical publications) and can be handled with little, if any, difficulty. It is unfortunate that specifications or control based on the probability approach has been labeled statistical. This labeling has delayed the acceptance of the probability approach because many engineers are not familiar with statistics. At the same time, this approach has interjected a sophistication and complexity that are neither needed nor justified in construction. The engineer's sampling is crude; the tests are relatively crude; and the results from these tests provide little assurance that the parameters are directly related to the service provided by the facility. The computer is probably to blame—it is accessible and engineers use it and fall in the trap of unnecessary sophistication and statistical complexity. Some 150 years ago Henry Clay said that statistics are no substitute for judgment.

Still another weakness is that most specifications are written to keep the engineer in control; therefore, in effect, the engineer is designated the responsibilities of the

Figure 1. Diagram of a system to promote quality assurance.



contractor. This turnaround upsets the normal legal relationships, and in effect the engineer becomes responsible rather than the contractor. It is time that the contractor is allowed to assume the responsibilities (5).

Finally, the most serious weakness is that the engineer and lawyer who write the owner's specifications have managed to give these a tone that foments adversary relationships between the owner's representatives and the contractor. The reason behind this antagonism was expressed succinctly in a journal article:

> A remarkably high percentage of respectable, God-fearing consulting engineers, men who are generally at peace with themselves and who love their neighbors, are convinced that all contractors are crooked and would substitute rubber bands for rivets if they could save 15 cents on a $10 million job.

This attitude raises costs, frequently delays the work, and negatively affects the desired quality. What is needed is contract documents that engender cooperation and teamwork and that are above all equitable to both sides. This cooperation is the only way to reduce costs, expedite the work, and assure the attainment of quality.

CONTROL OF QUALITY BY TRAINING

Training for the construction phase for both owner and contractor organizations is important for the attainment of a quality-assurance program. Each training program has to be designed for the specific situation, and the training is needed to indoctrinate experienced persons into the special problems of the project. Training is also needed to develop the persons who lack experience.

There are several tools that can be used to assist in the planning and conducting of such training (7, 8). However, this field is changing and is being upgraded so fast that the most current practices are likely to be found in the last job rather than in the literature. I no sooner finish a job than I already have ideas about how to do the same job differently. The important point is that there is rarely enough time for training, and this does not permit adequate control to be obtained at minimum cost.

CONTROL OF QUALITY DURING CONSTRUCTION

The person who actually does the work controls the quality during construction. An inspector or a supervisor can stand over a worker, but cannot move the hands of the worker or manipulate the brain that directs the worker's hands. This is why training is so important for developing not only the proper skills to attain the desired quality, but also for attaining the proper attitudes.

In addition, control of quality during the construction phase depends on control during earlier phases. Controlling quality by testing the cylinders, slump, or air content in concrete is too late since the quality desired is either there or it is not. If it is not, the material may have to be rejected.

With these two points in mind, how does one go about controlling quality during construction? Control in the field should be divided into two aspects that are parallel with the two basic responsibilities during construction:

1. Product control and inspection by the contractor to assure that the work will meet the requirements that have been contracted, and
2. Acceptance at various stages by the owner's engineer to assure that the contractual agreements have been met.

Both activities use the same principles, philosophy, and skills, but for different purposes and, therefore, require different approaches. Another interesting approach to the control of construction activities in the field is outlined by Riley (9).

Control by the Contractor

By the provisions of the contract, the contractor is responsible for providing a controlled product that will meet the requirements of the plans and specifications. The contractor purchases materials, assigns equipment and personnel, and oversees work being done by the workers. The contractor is also responsible for the results obtained. But contractors are not prequalified to be quality minded—the lowest bidder receives the job (10).

Besides training, there are two basic aspects of a program that will permit the contractor to control the product for assurance of acceptance and to do the job for minimum cost. First, the contractor has to set up a sampling (11), testing, and charting program (12) that will predict what the performance will be (based on the information developed at the beginning of the work or based on historical data that have been developed from similar work). Second, the contractor has to set up inspection to check on items that cannot be sampled and tested (13, 14). The contractor must, therefore, have a control group responsible to management and not responsible to the superintendent or production people. The control group must have testing facilities on the site and must develop sampling plans, sampling procedures, testing, and charting for use in predicting and in finding problem areas and solutions. The control group develops data and relays information and perspective to both production and management. The production group decides whether to use the information (15). The control group and the production group should not interfere with the work of each other, but management must see that the production group takes advantage of the information developed by the control group.

For the system to work, there must be give and take at the working level. The control group supervisor or manager must establish a friendly relationship with the production superintendent. If such a relationship is not established, the program cannot succeed. Therefore, the personalities of the persons heading the two groups have to be compatible, and management has to insist that they work together even though neither is the boss of the other. The best way to achieve this is to promote a person who is already in the organization, who is accepted by everyone, and who can be trained to head the control work. This person has a better chance of establishing a rapport with the production group than would a stranger who is brought in from the outside. Sometimes a small start like this will enable the program to be accepted, and a person from the outside who is more highly trained can be brought in without difficulty. However, the program must be sold first.

Many other factors of quality control are not subject to codification or formalization, and these are perhaps more important than the formal sampling, testing, and charting. For the most part, these factors are items that are peculiar to each job. For example, there is one sieve size for a particular aggregate that is a critical factor. This factor can be checked very quickly in the middle of the process, and future complications can be avoided. Another example is to be aware of a noise or action that gives a warning signal that an adjustment is needed. There are also some simple informal tests that guide workers and supervisors, assure them that the process is going smoothly, or warn them that it is not.

The control activity by the contractor cannot be delegated to an outside group because such a delegation will fail to imbue the contractor's organization with attitudes about control that must permeate the entire organization from top management down to the lowest ranking laborer.

Control by the Owner

If the contractor is responsible for results, the owner has no legal right to tell the contractor how to work, what equipment to use, or how to administer the job. If the owner gives the contractor directions and the contractor can show that he followed directions, then the results become the owner's responsibility. The owner has the privilege of either accepting or rejecting the work at various stages. Depending on circumstances, the owner can check for acceptance either through sampling, testing, charting, or inspecting. The same type of sampling, testing, and inspecting should be done by the contractor but for a different purpose. Actually, the contractor's facet of the control program requires more frequent sampling and testing and more detailed inspection than the owner's acceptance activities. If the owner can rely on the data developed by the contractor, then the owner's work for acceptance becomes minor and simple. On the other hand, if the contractor fails to carry out proper control, then the owner's sampling and testing should be intensified, and inspection will be in order (16). The contractor is in the position of the owner when he tests and inspects supplies and subcontractors for acceptance.

Incentives

Control during construction can be more easily achieved if the specifications have incentives for the contractor. The specifications should also include provisions for varying the levels of the owner's testing and inspection; such levels will automatically operate depending on the contractor's control performance. This check by the owner will add incentive for the contractor to properly control because intensified sampling, testing, and inspecting by the owner will make it tougher on the contractor and raise costs (17).

When a program of this type is operating properly, the contractor saves cost by control since rejections are reduced and efficiency is improved. Normally, this procedure should more than pay for the contractor's share of the program. If the owner employs minimal sampling and inspection, savings are also made. On future work some of the saving obtained by the contractor is bound to be passed on to the owner in competitive bidding, so the saving is not only in the control and acceptance costs but ultimately in the bid prices.

It is essential that there be cooperation between the field forces of the owner and the contractor; otherwise, costs go up, control is lost, the owner becomes a policeman, and the contractor tries to get by with anything the policeman will not catch.

INTERACTIONS AND FEEDBACK

The various facets of the quality-assurance system interact among one another not only in the technical but also in the nontechnical areas. These interactions vary from situation to situation and according to the persons involved.

For such a system to function effectively, there must be feedback from the various facets to permit adjustments or to learn lessons for the future. The most important areas of feedback are from the field to the design office and from the field to the contractor's organization (mainly between quality control manager and estimators). Other important feedback areas are the planning teams and the environmental and ecological staff. All such feedback should be preplanned and should be regularly scheduled if it is to be effective. This planning also helps to tie together the various groups and disciplines involved.

DISCUSSION AND SUMMARY

This approach to quality assurance has proved itself, even when applied in a limited manner for as long ago as 20 years on the Illinois Toll Highway. It is simple. It is based on fundamentally correct approaches. It improves efficiency and reduces costs. It maintains the integrity of contractual relationships. It makes contract administration much simpler. Basically, it is an adaptation to construction of the same principles used successfully in industry for many years. Why then is it not being widely used? There are several reasons.

People accept change slowly and reluctantly. Because the persons in charge of policy have a vested interest in the status quo and because they had a hand in developing the status quo, they hate to see it change. It has been said that truth does not triumph because its opponents are made to see the light but rather because they eventually die and a new generation arises that recognizes the truth.

Although this outlook may be realistic, it is pessimistic and offers little hope of progress through the people in key policy positions. Every once in a while, one does find a key person who has a sufficiently progressive outlook to try new approaches, and this is how progress takes place.

Quality assurance is not achieved through testing and inspection by the owner's engineers. The control of quality is an overall activity that starts with the designer who sets the level of quality and expresses this level of quality in the plans and specifications. The contractor exercises control of quality when he plans how to build a project and estimates the cost for the bid.

After a contract is signed, the contractor and owner's engineers develop organizations that must be properly trained and indoctrinated if the quality desired is to be attained and attained at minimum cost. During construction the contractor needs to establish a control group that is independent of the production group. The functions of this control group are to set and carry out sampling plans, test the samples, keep control charts, and inspect and report both to management and to production. The owner's engineers also need an acceptance group that will evaluate the data and charts developed by the contractor's control group, and carry out additional sampling, testing, charting, and inspection as needed. Finally, the experience from the field operations must be made available to the designer so realistic quality levels for future projects can be attained. This feedback is the most important. Failure in any one of these facets can cause failure in performance. It is by the use of this overall concept that the quality desired can be achieved at minimum cost.

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An Integrated and Computerized Quality-Assurance System

S. C. Shah, Louisiana Department of Highways

A quality-assurance program within the highway department involves, besides material sampling, testing and inspection, review, and update of these procedures based on performance of the finished product. Conceptually, it is a feedback system and emphasizes the interrelations among various disciplines within the highway department. Louisiana has expanded its quality-assurance program to provide such a feedback system to management, design, construction, and specification personnel. The system is computer-oriented and is capable of providing (a) assistance in monitoring construction projects for compliance with specifications, (b) a log of major construction materials and tests, and (c) a record of the pattern of variation due to contractor, materials, and equipment for subsequent statistical evaluation of quality control and specification review and update.

Interest in a quality-assurance system was initially aroused in the early 1960s. This awakening led to a research effort by a number of states, including Louisiana, to obtain information relative to the variability and quality level of the current construction practices. This information on variability was then translated into specifications by using standard statistical procedures for quality control and acceptance. The initial effort was finally completed by implementing these statistically oriented specifications for highway construction and materials.

Now, in the 1970s, the emphasis is on a systems approach to quality assurance. Application of statistical techniques to writing highway construction and material specifications has been well established. We must now develop a system to provide continual feedback of data with respect to materials, tests, performance, and specifications for all disciplines within the highway department. Figure 1 shows quality assurance as this closed cycle. Tests and inspection of construction and materials lead to acceptance (or rejection) of the finished product. Performance characteristics of this finished product are then related to the materials and construction inspection and tests originally incorporated into the product. Such review may eventually lead to revisions and updates of design procedures and specifications, and the cycle repeats itself.

The above system is truly a data feedback system and is a prerequisite for any decision-making process. Such an integrated feedback system in the manual mode would almost be an impossibility. For an optimum system, automation is the only approach. Louisiana is developing a computerized system to report material and construction test data that would best satisfy the feedback requirements for the quality cycle to operate effectively. This paper attempts to discuss the conceptual design and development of such a system.

CURRENT MANUAL SYSTEM

One of the major limitations of the present manual system is its inability to respond rapidly and accurately to a wide variety of user requests. This means that, if a change in the output is desired, considerable time and effort are required to respond to the new request. The second deficiency in the manual system is its inability to cross reference passing and failing samples, tests, and locations. This ability to cross reference is most desirable when it is time to compile data for final certifications. The third deficiency of the current system, which ties directly to the first two deficiencies, is the format in which the data are organized and stored. The format does not lend itself to adequately analyzing quality control.

In most cases, material sampling and testing for job control or conformance are performed by one or all of the following sources: project engineer, district laboratory, materials laboratory, and material producers. The volume of sampling and testing depends on the number of ongoing construction projects in each district. Past records have indicated that an average of 15 000 tests are performed by each district during the course of a year. The figure is twice as much for the materials laboratory. Seventy-five percent of the data generated by the laboratories require typing for distribution. The typed documents are filed and stored in filing cabinets. Extraction of user-requested information is done manually, and this effort can become extremely tedious, if

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not impossible. The plight of the project engineer (or any person) when it is time for final certification is shown in Figure 2.

Figure 1. Cycle for quality assurance.

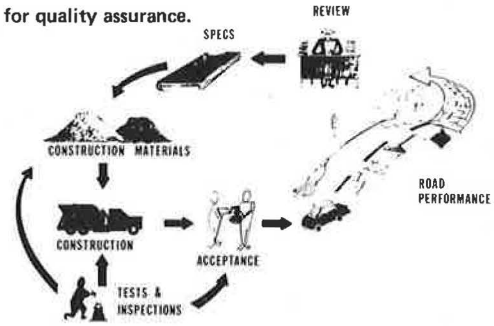


Figure 2. Manual retrieval of information requested by the user.

INTEGRATED AND COMPUTERIZED SYSTEM

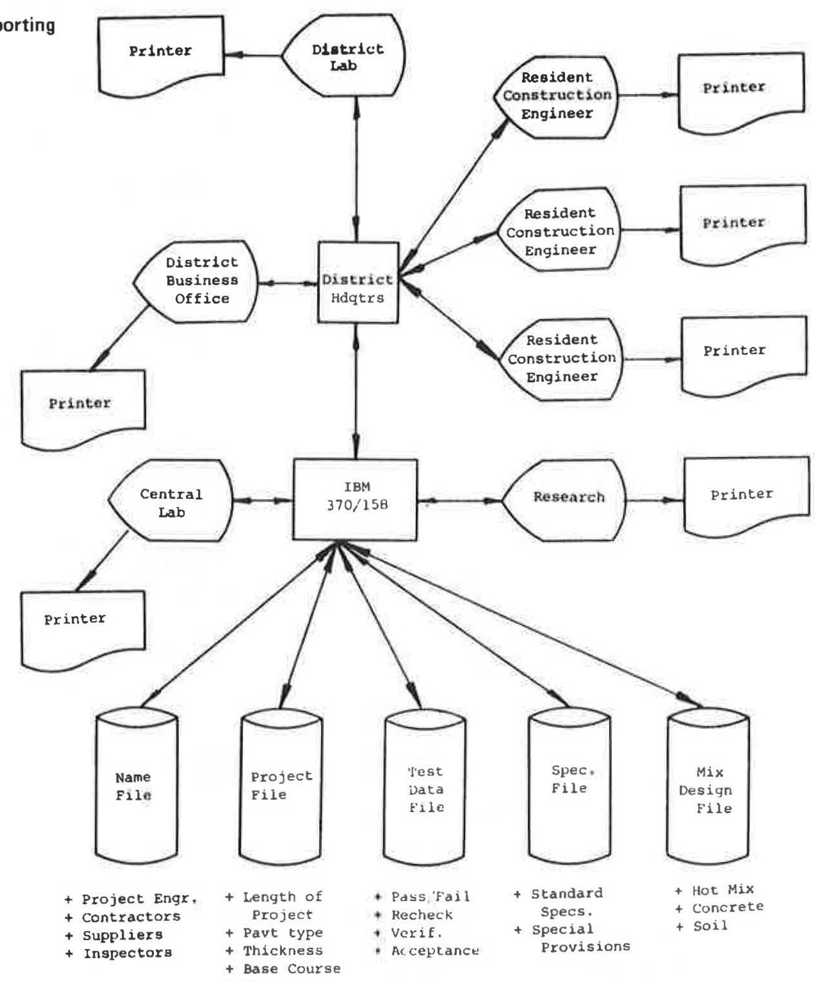
The purpose of the new automated system is to integrate the current system of sampling, test reporting, certification, and evaluation of construction projects. The system operations necessary to achieve these functional goals can be listed as follows:

1. Provide input data that can be easily transmitted and stored;
2. Provide output reports according to specific categories such as materials, project, source, and districts;
3. Provide specification check for each material and test;
4. Cross reference passing and failing tests for a given sample, test, and location; and
5. Provide timely information to all disciplines within the department.

System Design

Figure 3 shows a flow chart for the materials testing system. The system will be a teleprocessing system in which all data entry and retrieval will be through a type of television terminal. When the system is fully operational, a total of approximately 30 screens will be provided to the user for data entry, inquiry, and retrieval. The following is a list and description of the various screens.

Figure 3. Overview of network system for reporting material test data.



1. The project screen will have all the information pertinent to the project. The major thrust will be to provide information relative to project identification, cost, cross section, and other similar information relative to the project.
2. The material and test data screens will include information regarding aggregate, asphalt cement, liquid asphalt, cement, steel, hot mix, paving concrete, structural concrete, soil for base course, compaction of base course, mix design, and miscellaneous.
3. The specification screens will be standard specifications for each of the above materials and tests.

Figure 4. Test data entry and processing system.

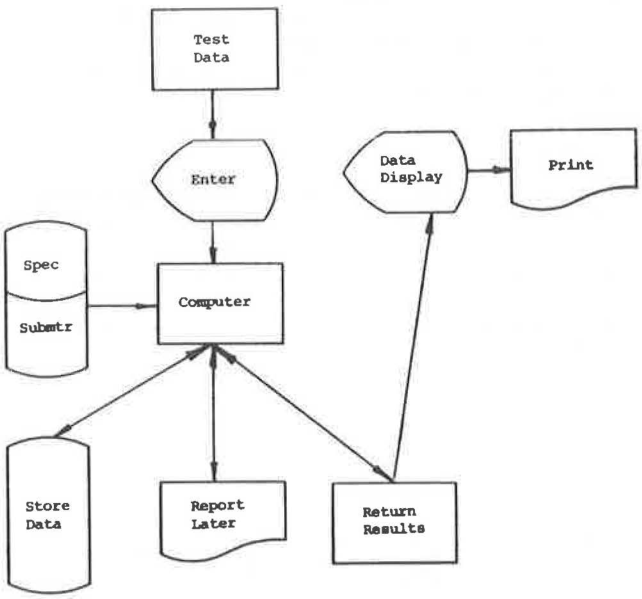
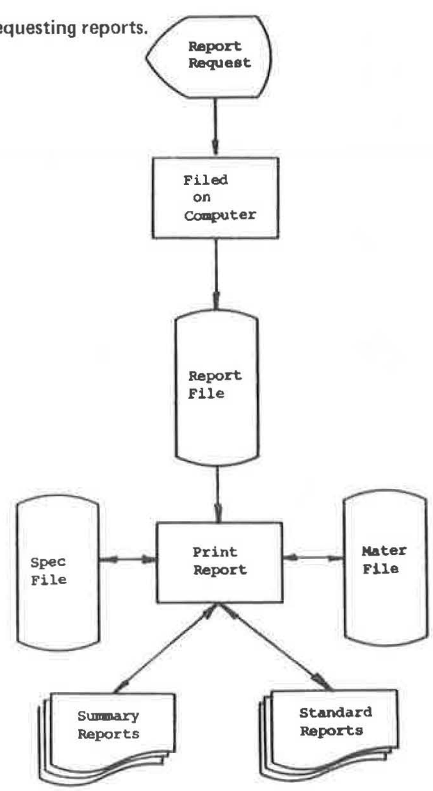


Figure 5. System for requesting reports.



Provisions will also be made for entry of contract specifications.
4. The submitter screen will project the names of all submitters including the contractors and materials producers.
5. The table of contents screen will list all the screens available for the system and the identification of transaction code necessary for retrieval.
6. The report request screen will project all the reports available through the system. Thus, report 1 may be a standard report for aggregate data and report 10 may be a summary report for base course compaction.
7. The browsing screen will allow the requester to retrieve any information that is relative to the construction data.

Data Input

The input of data will be through the terminals and from input forms designed specifically as work card and data entry card. The screen for each material or test will be an exact replica of the input data forms. This similarity will make the data entry easy and flexible. Figure 4 shows an overview of the system operation after the data are entered.

Data Output

When the selected information is needed, most of the reports will be provided on demand. For entry of a report request, the system will provide a request display on the terminal (report request screen). The requester will identify the report needed that may be a standard report or a summary report. A flow chart of the report request system is shown in Figure 5.

System Requirements and Implementation

The system is expected to be operational by the end of 1977. The system is designed for IBM 370, model 158, and IBM 3270 communication terminals.

SUMMARY

The brief overview of the expanded quality-assurance program in Louisiana through the use of computerized storage-retrieval techniques is expected to provide relief from two important problems:

1. The continuing increase in time and effort required by personnel in collecting, recording, and processing the data on a variety of documents; and
2. The difficult, if not impossible, task of retrieving these data for use in ongoing operations with respect to construction, design and maintenance, specifications, research, and other problem-solving efforts for adequate planning of a future system for pavement feedback data.

ACKNOWLEDGMENTS

This study is being conducted in cooperation with the Federal Highway Administration. The views expressed are mine and not necessarily those of the Federal Highway Administration.

Role of and Need for Maintenance Feedback

Milton S. Greitzer, New Jersey Department of Transportation

This paper discusses (a) the communications gap among maintenance, design, and quality-assurance personnel, and its causes; (b) highway maintenance—what it is, its cost, and its increasing share of the total highway budget; (c) cost of general or fiscal maintenance and its possible relation to the original design and quality-assurance program; (d) proposed maintenance research projects, and especially one project that seeks to identify particular features that affect normal maintainability of a highway facility; (e) the improved expertise of maintenance personnel and the current maintenance management system that can improve maintenance feedback; (f) recurring maintenance problems that require consideration by design and quality-assurance personnel; and (g) the importance of maintenance feedback and how to improve it.

The maintenance engineer can play a key role in a good quality-assurance program by reporting on the performance of the end product. Unfortunately, many times the maintenance engineer does not discharge this important duty and does not provide the necessary inputs because of a communication gap among the personnel in maintenance, design, and quality assurance. It stems from the fact that the maintenance personnel feel they are involved in the world of practical application, and they must accomplish their mission in an expeditious manner so that a needed facility is always available to the public. The maintenance personnel also believe that design and quality-assurance personnel live in an ivory tower, are full of unnecessary technicalities, and are slow to respond to maintenance needs. I can best illustrate this point by an incident that occurred a few years ago.

As a result of several serious accidents, including at least two fatalities that were apparently due to the slipperiness of the pavement on a traffic circle, it was decided to immediately overlay the pavement. The quality-assurance personnel were to provide the maintenance forces, who were to do the work, with a specification for a bituminous concrete overlay composed of crushed-gravel aggregate. The time of the year when the work was to be accomplished was late December. Among the items stipulated in the specifications provided by the quality-assurance personnel to maintenance personnel was the following weather stipulation.

> The crushed gravel bituminous concrete mixture shall not be placed when the atmospheric temperature is below 4.4°C (40°F), or has been below 0°C (32°F) during the preceding 8 h, or is expected to go below 0°C (32°F) during the 8 h following start of paving.

The choice was a simple one: Design the specification and do the job in December because of the paramount needs of the motorists, or delay doing the work until spring to comply with the specifications. From the point of view of maintenance there was no choice. It was decided to proceed at once, and the pavement was laid at temperatures ranging from -6.7 to -1.1°C (20 to 30°F). The quality-assurance personnel monitored the project by checking ambient temperatures, existing pavement temperatures, and the temperature of the bituminous concrete. The quality-assurance personnel made mild protestations and were generally critical of the entire operation.

Admittedly, the finished product in December was not up to standard. The longitudinal and transverse joints were of questionable quality; yet, the motorists had a much safer traffic circle. The advent of summer temperatures the following year, combined with the high volumes of traffic using the circle, knitted the joints together and improved the appearance and ridability of the resurfacing project. The maintenance personnel felt they had accomplished their role, in spite of the specifications, and provided a needed service to the public. I can think of no better example of the divergence of views and needs based on individual disciplines that tends to separate quality-assurance and maintenance personnel.

There is at least one common denominator in which we all share an interest: the annual budget and how the transportation dollar is spent. Maintenance costs throughout the nation have been on a steady increase that is irrespective of the inflationary spiral. Morgan (1) reported that an estimated 16.7 percent of the total annual state highway disbursements in 1974 was spent on maintenance and operations. This disbursement amounts

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to an estimated expenditure of $2.7 billion out of a total state highway disbursement of $16.3 billion.

Highway maintenance is defined as the act of preserving and keeping a highway, including all of its elements, in condition as close as is practical to its originally constructed condition, or its subsequently improved condition; and the operation of a highway facility and services incidental thereto, to provide safe, convenient, and economical highway transportation. The highway maintenance role is divided into two major activities: general or physical maintenance and traffic services or operations.

The first category consists of the physical upkeep of the highway and includes the repair of pavement failure, maintenance of shoulders, repair of guardrail, sealing of joints, and cleaning of drainage. The repair and maintenance of bridges and structures are also included in the physical upkeep. For the last 15 years, approximately 66 percent of the total maintenance expenditure has been directed to the general or physical maintenance of the roadway and structures. Since this expenditure may bear a direct relation to the original design and quality-assurance program, it should be of special interest. How much of this maintenance cost can be traced to the original design and quality-assurance program is uncertain. Whatever this figure may be, maintenance feedback can reduce it.

Feedback has always been a hope of maintenance personnel, and they have also had the expectation that design and quality-assurance personnel would at least equate low possible initial construction costs with ultimate maintenance cost. There is considerable doubt in the maintenance community that maintenance costs are given any weight at all in the original determination. Is their belief correct? Or, do design and quality-assurance personnel consider the impact of that oft-quoted expression, "It's not the initial cost, it's the upkeep."

In August 1976, the Federal Highway Administration (FHWA) distributed a report (2) that describes the conclusions reached by 60 maintenance experts in regard to a study for maintenance research needs. Twenty-eight major problems in nine maintenance areas were highlighted. It is interesting to note that one of the proposed research projects was "Integration of Maintenance Needs Into Preconstruction Procedures." The research problem statement reads as follows (2, pp. 97-98):

The successful completion of a highway project is dependent on the satisfactory accomplishment of the various phases leading to the end product. Planning, design, construction, operations, and maintenance play equally important and interrelated roles. The opening of a highway facility to traffic should not be construed as completion of the project. The operation and maintenance of the facility are of paramount importance to the safe and efficient movement of people and goods.

Decisions reached during preconstruction activities such as planning, right-of-way location, environmental deliberations, and design decisions may have detrimental effects on maintenance costs, operations, and effectiveness. These effects may be one-time or continuing, corrective or uncorrectable. In too many cases, these effects last the life of the highway facility.

On the other hand, properly coordinated planning, location, and design features and decisions may result in positive effects on maintenance costs and operations. From the standpoint of maintenance, the facility needs to be planned, located, and developed so as to enhance maintainability without adverse effects on the other functions.

The scope of this project is intended to be restricted so as not to include the structural design of pavements. That problem area is, of course, critically important but is being handled under other research projects. The scope of the project is, however, intended to include geometric and other design features.

Objectives

1. To identify particular features that add, detract, or otherwise affect the normal maintainability of the highway facility.
2. To develop information that will measure the increase or decrease in maintenance requirements and expenditures generated from the features identified above. It is suggested that the output of this task be structured in a format common to the preconstruction process.
3. To recommend adjustments in the preconstruction functions and processes so that proper consideration of the results obtained in 1 and 2 above can be incorporated.

As one can see, the maintenance feedback problem is real and nationwide.

It has only been within the past 15 years that maintenance personnel have graduated from common laborers to maintenance engineers and maintenance technicians. During this same period of time, there has been developed a more sophisticated and disciplined approach to maintenance work, which is today generally known as a highway maintenance management system.

In effect, this system is simply a use of management tools such as organization, planning, scheduling, budgeting, performance measurement, control, and reporting. The matter of reporting is directly related to this subject of maintenance feedback. An effective reporting system that uses data processing techniques can and now does provide detailed information regarding the types of maintenance work accomplished on any given section of highway—its frequency, hours spent in repair, materials and equipment used, and detailed and total cost figures. These data, together with the increased amount of engineering and technical expertise of the maintenance personnel, can provide an invaluable source of information to design and quality-assurance personnel on the performance of the end product.

The local maintenance foreman can also provide a wealth of detailed information. The local foreman literally knows every bit of the roadway and can cite the defects, virtues, what repairs have been accomplished, and where and what repairs are needed on the roadway. However, the local foreman does have difficulty communicating this information upward.

The engineers and inspectors for the New Jersey Department of Transportation administer the construction contracts from plans and specifications to reality. They provide the information to maintenance personnel about problems encountered during the construction process that can have a direct bearing on resultant maintenance problems. They are also in continual contact with the quality-assurance and design personnel, pointing out conflicts and errors or omissions in plans and specifications, feeding back information, and seeking prompt answers to questions so the contract administrators can provide the contractors with information to keep a project moving. Unfortunately, since the construction personnel are geared to a contractor's fast-moving and profit-oriented operation, they are usually seeking quick responses. Therefore, their feedback often comes through as a criticism of the design and quality-assurance personnel. This is not their intention; it is only a difference in needs and work scheduling of the disciplines involved.

What are some of the recurring maintenance problems that require special consideration by design and quality-assurance personnel? The Highway Maintenance Research Needs Report previously mentioned (2) notes several nationwide maintenance problems that may or may not have been fed back, but have not as yet been resolved. I will note only two that have been included as research projects (2, pp. 103-105, 115-118):

Research Project Title
Reduce Number of Expansion Joints in Existing Bridges

Research Problem Statement
Bridge expansion joints normally require frequent periodic maintenance to clean and reseal. Those structures having bituminous concrete surfacing magnify the joint at the surface by spalling through the overlay and thus

require additional maintenance. Most of these maintenance operations must be performed while the bridge is open to traffic. Also, many operating joints are difficult to seal and may permit water to drain on critical facilities below. After initial shrinkage and shortening have taken place, selected bridges may require fewer expansion joints.

Objective
Determine those bridges by type, length, temperature range, and overlay that may be amenable to eliminating selected expansion joints.

Research Project Title
Improved Pavement-Shoulder Joint Maintenance

Research Problem Statement
Many miles of highways have been constructed and are being designed and constructed with portland cement concrete roadway pavements and bituminous-surfaced shoulders. The resulting longitudinal joint between the roadway pavement and the shoulder contributes to costly maintenance and inconvenience to motorists. Keeping this joint sealed to prevent the intrusion of water and other objectionable material is very difficult, but of prime importance to pavement performance. If the joint is not completely sealed, surface water will eventually get beneath the roadway pavement and the shoulder and will contribute to pumping, faulting of transverse pavement joints, and shoulder settlement.

Objective
Develop maintenance procedures and operations that will alleviate the problem associated with the joint between a portland cement concrete roadway and a bituminous-surfaced shoulder.

A polling of several maintenance supervisors in New Jersey indicated full concurrence with the problems just outlined. They also enumerated several other troublesome areas that may only be common to New Jersey, but could well deserve quality-assurance and design attention Some of these problems are as follows:

1. The surface treatment of shoulders with light-colored gravel or stone to simply provide delineation or contrast from bituminous concrete riding surfaces [a great deal of the aggregate soon finds its way from the shoulder onto the riding surface or into the drainage system or is plowed off the shoulder during winter snow operations, and it is suggested that the use of a 20.32-cm (8-in) wide edge line or intermittent diagonal stripe be painted on the shoulder to provide delineation by indicating the nonriding surface];
2. The standardization of bridge railing and fencing to minimize the inventory of the types that must be stored and to reduce delay time when replacements are ordered;
3. The difficulties involved in plowing and storing snow and its subsequent melting and freezing on superelevated road sections (a special drainage design or a break in grade on the superelevated cross section of the high side would provide an area for snow storage);
4. The problem of plowing snow on bridges that traverse other roadways without depositing snow on vehicles traveling below because of the open bridge railing and lack of wide catwalk areas for snow storage;
5. The umbrella drainage that becomes inoperative with a buildup of turf and is the cause of considerable erosion to slopes on steep grades (curb or berm would direct flow to specific paved gutter or inlet areas);
6. The placement of trees or shrubs that makes machine mowing of grass difficult and requires costly maintenance handwork; and
7. The planting of trees or shrubs to provide a fence for snow at ineffectual locations (experienced maintenance personnel can provide information concerning the direction of snowstorms, accumulations, and prevailing winds to help in determining critical sites where a snow fence is needed).

These are only some of the problems that the maintenance personnel feel are being built into roadways. How do we correct them?

If maintenance feedback is to work, the parties involved must have a mutual respect for the role each person plays and be able to discuss problems as equal partners. My experience is that, on projects or committees that are not related to maintenance feedback, the design, quality-assurance, planning, maintenance, and other disciplines will work together to resolve a problem. The problem with maintenance feedback is that, of necessity, it implies a criticism of the work of others and, therefore, often results in a negative response. By nature, designers are sensitive about their designs. If maintenance feedback is going to work the design and quality-assurance personnel will have to become better listeners. They will have to get out into the field, look at the end product, and actively solicit comments from the maintenance personnel.

On the other hand, the maintenance personnel must become more effective in communicating their problems and more constructive in making their criticisms. If we work together as a team, we will be able to build a better transportation product, and we should use every tool at our disposal. Maintenance feedback is a relatively untapped source of information about the end product and has the potential of playing a very important role.

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Quality-Assurance System for Cement-Aggregate Concrete

Charles D. Fehnel, Lone Star Industries, Inc., Greenwich, Connecticut

The concrete construction industry is unique because it involves quality-control systems that must be exercised on the production and performance of individual materials (cement and aggregate) and also of concrete, the ultimate product in which these materials are used. Therefore, specifications written for one material will generally affect the performance and ultimate consumer cost of all three materials because of the degree of quality control required for each material. To demonstrate the effect specifications may have on the ultimate cost to the consumer, examples from a concrete supplier's quality-control and quality-assurance program are presented. The equipment and manpower required for carrying out these programs and the ultimate effect of these requirements on the economics of product processing are described.

Prior presentations have dealt with the significant aspects of quality-assurance programs for individual components. As a producer of cement and other building materials, we are keenly aware of the problems associated with the production of these components. As a supplier of these materials to the construction industry, we have likewise been keenly aware of the performance requirements necessary to make these products acceptable in the marketplace. For this reason, we look at quality-assurance programs from a broader viewpoint. The relation between the components and the system plays a significant part in our ability to produce a quality product that can be sold competitively in the marketplace.

Techniques for quality control have been consistently improved through the years. Improved production, process control, monitoring, testing, and data processing equipment and techniques have been the basic factors in our ability to establish new process and product performance standards. In cement plants, for instance, quality-control systems control and monitor the functions of quarrying, proportioning and blending at two stages, raw grinding, burning and cooling, finish grinding, and storing and withdrawing cement. For each function, specific sampling and control testing procedures are carried out and documented. Our particular control system involves almost 23 000 man-hours of work annually. Backup technical assistance involves at least 10 000 man-hours of work by our research laboratory personnel.

Depending on the degree of sophistication built into the system control, the data are collected, recorded, and analyzed either manually or automatically through the use of computers for maintaining control over the operating parameters. In addition to the traditional cement-testing equipment, X-ray, atomic absorption, DTA, microscopic, and mineralogical tests are also conducted.

A diagram of the cement process flow and the quality-control function exercised at various stages is shown in Figure 1. We have established separate standards for each production phase. In effect, each department has its own quality-assurance or quality-control program. Standard deviations and coefficients of variation have been the basic statistical tools for evaluating the variations in the chemical and physical properties of the cement. The control data and graphics shown in Figure 2 are used for visual-trend studies. At each plant, quality-control personnel observe these trends and are responsible for controlling them.

In the aggregate industry, we have similar controls over the production process with particular emphasis on size reduction (crushing), beneficiation, and size gradation. For concrete, the emphasis is placed on the proper blending of materials of a given quality so that a concrete of a defined quality is produced.

Concrete control data and graphics, similar to those based on the American Concrete Institute (ACI) code, are prepared. Examples of such information are shown in Figures 3 and 4. Our desire is to maintain a low standard deviation from the norm. We believe that greater flexibility can be allowed in specifications without sacrificing quality standards.

To further strengthen these quality-assurance procedures, test programs are established to evaluate the testing personnel and equipment, since the evaluation concerns the testing agreement between testing laboratories and testing personnel. Many laboratories, like ours, are certified by the Cement and Concrete Reference Laboratories of the National Bureau of Standards.

Therefore, quality-assurance programs for individual

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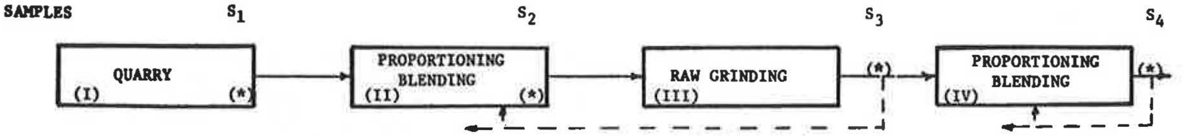
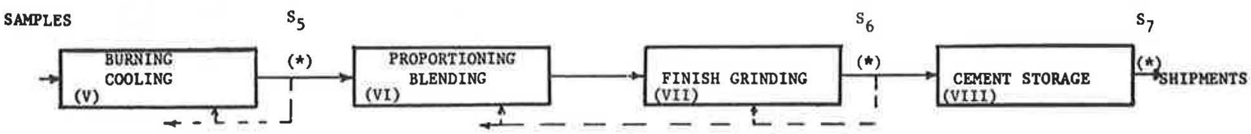
Figure 1. Quality-control systems for process flow, sampling, and control loops.



	(I)	(II)	(III)	(IV)
MATERIALS	1. Limestone 2. Useable Overburden (Alumina & Iron)	1. Quarry Rock 2. Purchase Materials a. Sand b. Iron Ore c. Useable Overburden (Alumina & Iron)	Slurry Raw Mix	Slurry Raw Mix
CONTROL	Chemical Composition by blending materials in rough predetermined proportions.	Chemical Composition Volumetric proportioning and blending of materials in predetermined amounts.	Fineness Water Content Chemical Composition	Chemical Composition Volumetric proportioning and blending of slurry to predetermined set point.



	(V)	(VI)	(VII)	(VIII)
MATERIALS	Clinker	1. Clinker 2. Gypsum 3. Additives	Cement	Cement
CONTROL	Chemical Composition and clinker quality	SO_3 content and chemical composition with volumetric and gravimetric proportioning.	1. Gypsum Addition 2. Fineness 3. Chemical Composition 4. Physical Characteristics	Random checks of shipments for specification conformance.

________ Material Flow
-------- Control Loop
(*) Sample Point

Figure 2. Computerized production analysis report of type 1 cement.

* CHEMICAL ANALYSIS *

MONTH	SIO2	AL2O3	FE2O3	CAO	MGO	SO3	LOSS	TOTAL	F	B	A/F	C3S	C2S	C2F C3A	C4AF	CASO4	FREE CAO	INS RES	NA2O	K2O	T ALK	NO TST
JULY	*21.71	5.08	2.31	64.72	1.05	*2.96	1.14	99.0	2.46	2.94	2.20	52.6	22.5	9.6	7.0	5.0	.67	.42	.09	.72	.56	19
AUGUST	21.42	5.10	2.31	64.53	1.09	3.22	1.35	99.0	2.48	2.89	2.21	53.2	21.3	9.6	7.0	5.5	.72	.38	.10	.73	.58	17
SEPTEMBER	21.33	5.09	2.26	64.63	.95	3.27	1.43	99.0	2.49	2.90	2.25	54.2	20.2	9.7	6.9	5.6	.79	.47	.10	.74	.59	30
OCTOBER	21.33	5.05	2.41	64.67	1.06	3.28	1.25	99.1	2.49	2.86	2.10	54.4	20.0	9.3	7.3	5.6	.63	.30	.11	.77	.62	23
NOVEMBER	21.40	5.04	2.38	64.52	1.06	3.31	1.11	98.8	2.48	2.88	2.12	53.3	21.1	9.3	7.2	5.6	.48	.29	.10	.78	.61	14
DECEMBER	21.26	5.06	*2.55	*65.00	.99	3.21	.96	99.0	2.52	2.79	1.98	56.2	18.5	9.1	7.8	5.5	.46	.43	.11	.76	.61	11
JANUARY	21.26	5.12	2.44	64.87	.97	3.24	1.16	99.1	2.51	2.81	2.10	55.4	19.1	9.4	7.4	5.5	.47	.43	*.15	.76	.65	7
FEBRUARY	21.25	5.14	2.38	64.65	1.03	3.31	1.15	98.9	2.50	2.83	2.16	54.3	19.9	9.6	7.2	5.6	.63	.40	.11	.77	.62	9
MARCH	21.34	5.06	2.46	64.76	1.13	3.27	1.06	99.1	2.50	2.84	2.06	54.6	19.9	9.2	7.5	5.6	.41	.36	.07	.73	.55	11
APRIL	21.30	5.09	2.41	64.67	1.19	3.24	1.14	99.0	2.50	2.84	2.11	54.5	19.9	9.4	7.3	5.5	.46	.33	.07	.71	.54	18
MAY	21.31	5.09	2.30	64.72	1.19	3.27	1.20	99.1	2.50	2.88	2.21	54.7	19.8	9.6	7.0	5.6	.52	.37	.08	.72	.55	18
JUNE	21.31	*5.23	2.37	64.57	1.17	3.25	1.30	99.2	2.48	2.80	2.21	53.1	21.0	9.8	7.2	5.5	.50	.38	.08	.76	.58	14
HIGH	21.71	5.23	2.55	65.00	1.19	3.31	1.43	99.2	2.52	2.94	2.25	56.2	22.5	9.8	7.8	5.6	.79	.47	.15	.78	.65	
LOW	21.25	5.04	2.26	64.52	.95	2.96	.96	98.8	2.46	2.79	1.98	52.6	18.5	9.1	6.9	5.0	.41	.29	.07	.71	.54	
AVERAGE	21.36	5.09	2.36	64.67	1.07	3.23	1.22	99.0	2.49	2.87	2.16	54.1	20.4	9.5	7.2	5.5	.59	.38	.10	.74	.59	
STD.DEV.	.12	.05	.08	.11	.08	.09	.13	.1	.01	.04	.07	.9	1.0	.2	.2	.2	.12	.06	.02	.02	.03	

* PHYSICAL ANALYSIS *

MONTH	FIN #325	WAGNER	BLAINE	V-I-C-A-T INIT	V-I-C-A-T FIN	G-I-L-M-O-R-E INIT	G-I-L-M-O-R-E FIN	FALSE SET	NORM CONS	%H2O	AIR ENT	COMPRESSIVE STRENGTH 1DAY	3DAY	7DAY	28DAY	AUTO CLAVE	NO TST
JULY	86.2	1785	3300	85	180	2.20	4.15	88	25.0	48.5	11.0	1950	3240	4230	5410	.005	19
AUGUST	85.1	1780	3275	80	175	2.20	4.15	65	24.5	48.5	11.4	1965	3275	4125	5245	.001-	17
SEPTEMBER	87.3	1875	3535	75	175	2.10	4.10	82	24.5	48.5	11.2	2090	3380	4270	5565	.003-	30
OCTOBER	86.3	1855	3290	75	175	2.15	4.15	87	24.5	48.5	11.8	*2260	3510	4450	5485	.008	23
NOVEMBER	87.2	1820	3315	80	180	2.15	4.10	86	24.5	48.5	11.1	2075	3420	4300	5490	.010-	14
DECEMBER	86.9	1770	3180	85	*190	2.25	4.15	84	24.5	48.5	11.5	2130	3370	4365	5400	.012-	11
JANUARY	86.7	1810	3145	80	180	2.15	4.10	86	24.5	48.0	11.7	2080	*3555	4350	5525	.002-	7
FEBRUARY	85.4	1730	3230	80	175	2.15	4.15	85	24.5	48.5	10.8	2090	3290	4215	5395	.002-	9
MARCH	83.8	*1705	3210	75	175	2.20	4.15	88	24.5	48.5	11.1	1945	3195	*4025	*5030	.002-	11
APRIL	84.4	1780	3265	85	172	2.25	4.15	83	24.0	48.5	11.3	1910	3315	4280	5491	.015-	18
MAY	84.8	1870	3370	85	175	2.20	4.15	88	24.0	48.5	11.2	1940	3330	4245	5276	.017-	18
JUNE	86.1	*1950	3369	83	176	2.15	4.12	83	24.3	48.5	10.9	1920	3287	4323		.018-	14
HIGH	87.3	1950	3535	85	190	2.25	4.15	88	25.0		11.8	2260	3555	4450	5565	.008	
LOW	83.8	1705	3145	75	172	2.10	4.10	82	24.0		10.8	1910	3195	4025	5030	.018-	
AVERAGE	85.9	1824	3371	80	176	2.17	4.13	85	24.4		11.2	2034	3348	4271	5410	.005-	
STD. DEV.	1.1	59	108	4	4	.04	.02	2	.3		.3	111	94	103	141	.008	

* VALUES WHICH EXCEED THE AVERAGE BY MORE THAN TWO STANDARD DEVIATIONS

Figure 3. Computerized concrete quality-control report.

CONCRETE QUALITY CONTROL REPORT

PROJECT
CONTRACTOR
ARCHITECT
ENGINEER
SUPPLIER

SPECIFIED 28 DAY STRENGTH 4,000 (10)
CEMENT TYPE LONE STAR I LB/CU YD 564 (6SK)
W/C 5.25G/SK .47/WT AGGREGATE 1" GRAVEL
ADMIXTURE: A-E NONE OZ/SK OTHER 3OZ POZ
1905 GRAVEL/YD, 1336 SAND/YD

	SLUMP		% AIR		7 DAY STRENGTH		28 DAY STRENGTH	
DATE	TEST VALUE	MOVING AVE 3	TEST VALUE	MOVING AVE 3	TEST VALUE	MOVING AVE 3	TEST VALUE	MOVING AVE 3
04-11-73	4.75				4545		6320	
04-12-73	3.75				4570		5650	
04-13-73	4.00	4.17			3150	4121	4635	5535
04-20-73	5.25	4.33			4315	4045	5255	5180
04-24-73	4.50	4.58			4605	4023	5615	5168
04-25-73	5.25	5.00			4305	4408	5160	5343
04-26-73	5.00	4.92			4395	4435	5800	5525
04-27-73	3.75	4.67			3430	4043	4845	5268
04-30-73	4.00	4.25			4810	4211	6270	5638
05-01-73	4.75	4.17			3785	4008	4990	5368
05-09-73	2.75	3.83			4505	4366	5330	5530
05-11-73	5.00	4.17			3995	4095	5605	5308
05-15-73	5.25	4.33			4505	4335	5905	5613
05-16-73	4.75	5.00			4155	4218	5280	5597
05-17-73	4.25	4.75			4040	4233	5475	5553
05-18-73	5.00	4.67			3510	3901	4200	4985
05-22-73	4.50	4.58						

02-28-74	5450	5340	110	5395	5735	5224
02-28-74	4700	4550	150	4625	5297	5218
03-05-74	4520	5100	580	4810	4943	5214
03-05-74	5080	5110	30	5095	4843	5213
03-08-74	4620	5040	420	4830	4912	5209
03-08-74	5070	5360	290	5215	5047	5209
HIGH				6535	6247	
LOW				3620	4077	
RANGE				2915	2169	
NO OF TESTS				101		
AVERAGE				5209		
STAND. DEVIATION				606		
COEF. OF VAR.				11.6		
AVE. RANGE				274		
WITHIN TEST STAND.DEV.				243		
WITHIN TEST COEF.OF VAR.				4.7		

APPROXIMATELY 2.3% OF THE TESTS ARE EXPECTED TO FALL BELOW THE DESIGN STRENGTH OF 4000

components are already in effect. Primarily, they are designed to provide the producer and consumer with a system by which process control and product quality optimization can be measured and cost reduction can be realized. These programs do, however, require many man-hours, sophisticated test equipment, and computerization.

However, is the quality-assurance system itself optimized? Does it assure that the most favorable quality and cost conditions under specific circumstances are realized? Or, are quality-assurance systems for individual components compatible with the objective of the ultimate quality-assurance system that requires a combination of several related systems? Too often, we zero in on one condition and perhaps later on we find we are headed in the wrong direction.

Within our construction-related activities such optimization is difficult to achieve. What are the variables that affect our quality-assurance systems and that relate to quality and cost control? Some of these are type and source of raw materials, type and availability of fuels, processing machinery and equipment, control and test equipment, type and cost of transportation, placing equipment, personnel, and specifications or design criteria. Each of these items obviously plays an important part in establishing the ultimate product quality and the cost of the product.

Some quality-control experts say that one should never consider cost when establishing criteria for optimum quality of the product. For this discussion, we will consider these factors jointly. Usually, there are many ways to achieve the same objectives; therefore, we must ask some basic questions. To what degree is the ultimate use of the concrete product flexible in chemical and physical performance characteristics? What do the specifications allow? To what degree can chemical and physical characteristics of each product (cement and aggregate) vary without affecting the performance criteria of the concrete? What flexibility can be allowed in processing, transporting, and placing without affecting the performance criteria? I have stressed flexibility rather than rigidity because we think a quality-assurance program should have flexibility.

Obviously, these questions indicate the need for correlation analysis. The chemical and performance characteristics of each component must be correlated with the chemical and performance characteristics of every other component to determine what variables have the greatest impact on achieving the ultimate objective, and how the variables can be controlled. Otherwise, ill-conceived quality-assurance programs may be too costly and cumbersome to maintain.

The programs often call for control efforts and information that have an insignificant effect on the final objective and are costly. Mainly these efforts include what we might call cookbook criteria such as organization structure, job descriptions, and internal plant communication forms. Do such requirements make the con-

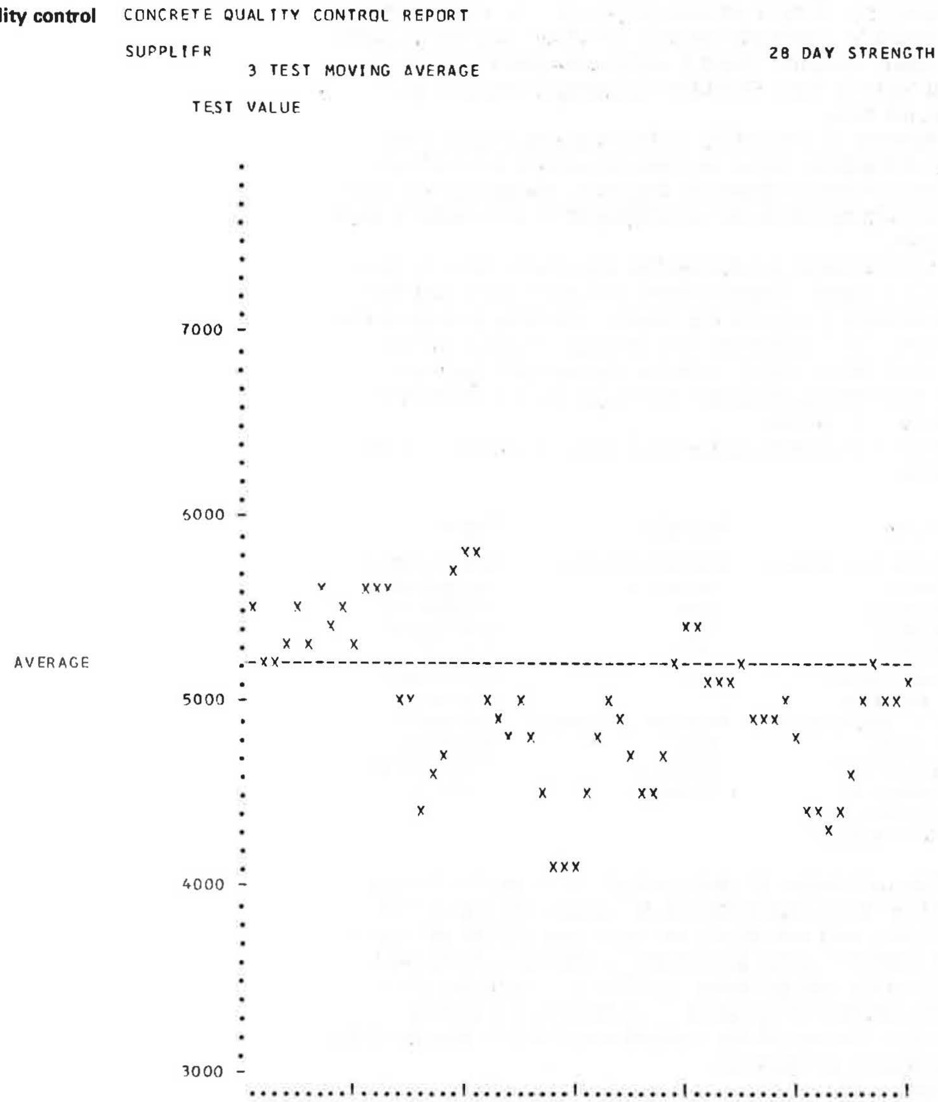


Figure 4. Computerized graph for quality control of concrete.

crete better achieve the performance objectives, or do they perhaps cause us to miss some of the more important points?

While reviewing the cement and aggregate requirements for highway construction in our marketing areas several years ago, we found that most of the states would call for AASHO standard specifications and then add specific exceptions to the specifications for use in their own areas. At that time, we were sure that each state had a legitimate reason for adding other restrictions to ensure product quality and desired field performance. However, as control and evaluation techniques are improved, the need for added restrictions is no longer supported by fact. Today, the need for the costly practice of silo testing and sealing has virtually disappeared. Inventory, sampling, and testing costs were reduced for all parties without sacrificing quality control.

The experience with aggregate is similar. There were 215 dissimilar coarse aggregate gradations specified throughout the United States. AASHO specifications called for 19 gradations. However, in this case, the flexibility of state specifications was based on local aggregate availability and on the knowledge that, although the gradation was different, its impact on the ultimate desired quality of the concrete was not significant.

Quality-assurance programs should, therefore, be structured to primarily flag those characteristics that have a significant impact on product performance, product costs, and placement costs.

It is now possible through the use of computer techniques to make the necessary correlation analysis to determine the critical characteristics that have a significant impact on the ultimate performance and cost of the products. One of the best examples of such a correlation effort is found in the three-part Building Science Series (1). We suggest a similar method for correlating product performance interrelations, concrete performance interrelations, and finally the performance interrelation between both. Initially, this would be a tremendous task because of the many variables that would have to be considered. However, this task would not be impossible. We should first determine what the high-risk, high-cost factors are.

Alkali in cement and aggregate is an important factor. Quality aggregates will become in short supply in many areas. Environmental considerations and raw material availability are influencing the level of alkali in cement. Historical data indicate that cement alkali should be low when used with an alkali-reactive aggregate.

As a result of several specific case histories in which low alkali may have been required, we now find a general demand for low-alkali cements in many areas where its use would have no significant effect on the purpose or performance of the concrete produced. In other areas, the available aggregate or crushed stone may not require low-alkali cement. Good quality-assurance programs would build in such flexibility by using historical performance facts.

Fineness of cement is another area of controversy. Some demand a coarse cement and others demand fine cement for better strength; however, the actual supporting correlation data are not available to substantiate such demands.

I have already mentioned the numerous sizes of gradation required. Suppose each gradation used with the same cement produced the desired strength and durability required. Would the specific gradation specifications have been necessary? A properly designed quality-assurance program would determine if such flexibility could be acceptable.

Some of the criteria for each component relate to the following:

Cement	Aggregate	Concrete
Chemical composition	Chemical composition	Cement content
Silicates	Limestone	Aggregate content and size
Aluminates	Basalt	Sand content and size
Alkalies	Slag	Water content
Sulfates	Gravel	Admixtures
Rare element impurities	Sand	Setting time
Physical characteristics	Physical characteristics	Workability
Fineness	Size	Volume change
Setting time	Gradation	Durability
Workability	Soundness	
Strength		
Volume change		

This list makes it obvious that, if we are to design a quality-assurance program to satisfy the hopes that variability will not occur, we must use all the information available from producers, testing agencies, and laboratories and put them together in a well-designed quality-assurance program. Certainly, even these variables are not all the variables related to the chemical and physical properties.

Has anyone conducted a correlation analysis based on the performance characteristics of these three components? Such analyses have to be conducted to determine what significant effect the variables of the components will have on the ultimate concrete. Has anyone also considered the intercomponent relations to the concrete performance? We feel the industry or specification writers still have an important step to take before establishing the true relation of each variable to the final product. We may well find that some of our specifications, as written, could be made more flexible and still protect the consumer. At the same time, producer and user would realize significant cost savings in mineral resource conservation, energy requirements, and capital costs. Quality-assurance programs need not be cumbersome or costly to protect the consumer and still minimize process and product costs.

REFERENCE

1. Interrelations Between Cement and Concrete Properties. National Bureau of Standards, U.S. Department of Commerce, Series 1, 2, and 3, 1965-1968.