HRB Special Report 106

Observations of the Performance of Concrete in Service

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Observations of the Performance of Concrete in Service

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ELI O. AXON

ELI O. AXON
A Tribute

It is entirely appropriate that this Special Report be dedicated to Eli Oscar Axon, who suffered a heart attack in early May of 1969 and died on May 7. Mr. Axon was born on July 10, 1907, in Breckenridge, Missouri, and attended elementary and secondary schools there. He attended Park College and in June 1929 was graduated with honors from the University of Missouri. He received a B. S. degree in Civil Engineering and was made a member of Tau Beta Pi, Honorary Engineering Society. He was married in Huntsville, Missouri, to Miss Eleanore E. Hunt, who survives, along with a daughter and son.

After completing college Mr. Axon was employed as an inspector of construction projects by the Missouri State Highway Commission. He spent his entire career moving up through various positions in the organization. Since 1941 he was a member of the Research Section of the Materials Department and in 1962 was named Chief of Research. His experience was vast and he enjoyed the confidence and absolute respect of a host of friends and associates.

The study of concrete, and particularly the performance of concrete in service, filled an important segment of Eli Axon's life for many years, and during the last decade he took an increasingly active part in the more formal aspects of concrete research. He belonged to the Highway Research Board and was a member of Committees MC-B2 and MC-B7, and Subcommittee 4 of MC-B1. He also served with distinction on National Cooperative Highway Research Program Advisory Panel E70, "Instrumention." He belonged to the American Concrete Institute, where he was an active and valued member of Committees 115 and 316. He also belonged to the Missouri Society of Professional Engineers and the National Society of Professional Engineers. He was the author of several papers and could be depended on for pertinent review comments on papers by others.

To those who think of prestressed concrete pavement as a recent development, at least in this country, it is of interest that Mr. Axon helped to complete a project with similar objectives over 30 years ago. Even though his experience dated back to the days of half-sack mixers and mule-drawn earthmoving equipment, his curiosity about, and knowledge of, new developments was impressive. He learned from everything he ever saw or heard, to his benefit and to the benefit of all those fortunate enough to be associated with him. Although he did not create argument simply for the sake of argument, he possessed the courage to express his opinions, even though those opinions might be contrary to long-held popular beliefs. Such people do not come along very often, and we will miss him.

In addition to his job-connected interests, Mr. Axon found time for many other activities. He was a member and elder of the United Presbyterian Church, was active in several Masonic orders, and was a Scout leader.

Foreword

The overwhelming majority of concrete performs its designed function satisfactorily throughout its intended service life. That this is true is a tribute to development and implementation of good concreting practices by the many agencies and individuals responsible for its placement. Much attention, however, should be directed toward those cases where performance differs from that expected. This attention, in the form of performance surveys, obviously represents significant investments of time and effort. If properly conducted and made available to the widest possible audience, these performance studies, regardless of scope, provide a basis by which past and current practices can be evaluated and refined.

Although much can be learned from studies of good performance, it is the study of poorer-than-expected performance that commands the most attention. Too often, however, agencies faced with a condition of premature concrete deterioration are reluctant to publicize it for varying reasons. Occasionally a study that is adequate to answer the necessary questions on a local level could, for a very slight increase in effort during its conduct, have been made applicable to a much wider audience by the collection of additional bits of data.

This Special Report is the result of a joint effort of three Highway Research Board committees, in recognition of the need for generating guidelines gained from experience by individuals engaged in performance surveys. During the course of the Report's development, liaison was maintained with the American Concrete Institute Committee 201, Durability of Concrete in Service (R. P. Vellines, chairman). At the 45th Annual Meeting in 1966, Committee MC-B1, Durability of Concrete—Physical Aspects (Howard T. Arni, chairman), Committee MC-B2, Performance of Concrete—Chemical Aspects (Howard H. Newlon, Jr., chairman), and Committee MC-B3, Mechanical Properties of Concrete (Thomas W. Reichard, chairman), sponsored a Conference Session on the Performance of Concrete in Service. This session set forth the general form and content of any substantial investigation of concrete performance, and papers from this session form Part I of this Special Report. Part II consists of several case studies dealing with individual problems of inadequate performance of concrete, some of which were presented in a session at the 46th Annual Meeting in 1967. Difficulties in formalizing material presented informally have led to the delay in its publication. In the interim, several other closely related documents have appeared that supplement this Report. References to these have been included in Appendix A.

During the revision of the conference notes for publication, death claimed one of the authors, Mr. Eli O. Axon. This volume has been dedicated to Mr. Axon. It seemed appropriate to leave Mr. Axon's discussion of the assembly of records as he presented it verbally, and therefore the editorial style of his paper varies from the remainder of the volume. Mr. Axon's assignment was to discuss in general terms the importance and limitations of records. He noted that the amount of records that could be assembled for a particular study is enormous and the type varies widely. Although not a part of his presentation, a listing of typical records in the form of a flow diagram has been prepared by Richard C. Mielenz; it appears as Appendix B.

It is contemplated that additional case study compilations will be published periodically. The benefits to be derived from a mutual sharing of both successes and failures cannot be overemphasized.

Appreciation is expressed to all of those individuals who planned and participated in the Conference Sessions and served as reviewers of either individual parts or the entire volume. Special thanks are due the task group that was responsible for organizing the Conference Sessions—Calvin C. Oleson, Paul Klieger, Peter Smith, and Dr. Mielenz.

It is the hope of all contributors that this volume will be helpful and will stimulate presentation of additional examples of "Observations of Performance of Concrete in Service."

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Part I
Guidelines for Observing the Performance of Concrete in Service

Investigation of Concrete in Service

RICHARD C. MIELENZ, Master Builders Division of Martin Marietta Corporation, Cleveland

•A REPORT on procedures, techniques, and examples of investigation of concrete in service is timely because of the increasing frequency with which such studies are undertaken and because new methods for analysis and evaluation of concrete in service have been developed recently. It is quite appropriate that the application of these methods be examined with a view toward determining their accuracy and reliability. Examples of such methods are nondestructive tests of concrete in place, new concepts on sampling of concrete, new physical tests of concrete, and new analytical techniques by which the composition of concrete can be determined or estimated.

An investigation of performance of concrete in service usually deals with failure or with some substantial inadequacy of the work. On the other hand, an investigation might be made to evaluate the condition of the concrete and concrete construction without any immediate consideration of failure. Or, a study might be made to determine why a particular concrete has performed outstandingly under severe conditions. In general, support is readily available for investigation of concrete that has failed in service, but we should not overlook the probability that examination of outstanding concrete construction will lead to improvement in standards for accomplishment of future work.

Diagnosis of real or apparent failure or unsatisfactory performance of concrete may be undertaken for any of several reasons. Concrete involved in unsatisfactory performance might be investigated to establish the ability of the concrete to continue to perform under anticipated conditions of service. The study might be made to identify processes that have caused or contributed to unsatisfactory performance. The objective may be to discover any defects in the concrete or the construction that contributed to the unsatisfactory condition. The study might be made primarily to establish remedial methods or protective measures that should be applied to provide for continuing service. A common objective of such investigations is the fixing of financial or legal responsibility for the failure or unsatisfactory performance. The scope of investigation and the procedures that are most appropriate depend to a great degree on the objectives to be achieved.

Investigations of unsatisfactory performance of concrete in service usually are based on a conclusion by the owner or his representative that unacceptable conditions have developed or are imminent. This decision is subjective to some extent because what is unsatisfactory performance in one region or in one condition of service may be acceptable elsewhere, or one owner or architect-engineer may be disturbed by a condition that is acceptable to another who is more phlegmatic, sophisticated, experienced, or cynical. That is, is the crack, the strain, the deflection, or the surface scaling sufficient to justify concern and at what point in time should an investigation be undertaken in an effort to assess the condition of the concrete and its future prospects? Such conclusions vary also over the course of time. It is to be hoped that standards or criteria of performance will rise so that we are less content to accept the "inevitable" cracks, pop-outs, scaling, or deflections.

Investigation of any substantial failure of concrete should be undertaken in accordance with an agreed-upon scope, objective, plan of attack, and prospective time schedule. At an early stage, a suitable budget of manpower and facilities should be arranged and the line of responsibility for prosecution of the investigation should be established. These decisions are based initially on preliminary observations and information available at that time. However, the investigation usually can be developed most effectively in stages, each successive stage depending on what has previously occurred. This process is most efficient in terms of manpower and facilities, but the overall elapsed time

may be so great that less efficient expedients will be dictated by the requirements of safety and maintenance. Consequently, from an engineering standpoint, the two most important objectives of the examination of concrete in service are, first, to establish the probable nature and extent of actual or imminent failure and, second, to choose appropriate remedial or protective measures. Hence, these objectives should be given primary emphasis in the plan of investigation.

It is our hope that the papers that follow will provide guidance in the development of more effective investigations of performance of concrete in service, and that the promising potential and the limitations of new investigational techniques will be more clearly evident.

Investigation of the Performance of Concrete in Service

E. O. AXON, Deceased, formerly Materials Research Director, Missouri State
Highway Commission

●IN HIS remarks Dr. Mielenz has stated that in any investigation of the performance
of concrete it is necessary to define the scope and objective of the investigation. I
most certainly want to stress the importance of this requirement. If this procedure
is not followed, the investigator will ultimately find that he has assembled a vast amount
of nonpertinent records. This, of course, is most costly, but what is perhaps more
serious is that unplanned action reduces the probability of attaining the desired results.
Because the objectives of investigations of poor concrete performance can differ ap-
preciably, identical records will not be assembled for each study. This makes it im-
possible for me to specifically list what records should or should not be assembled.

Now by mentioning investigation of poor performance of concrete, I do not want to
imply that all investigations are or should be concerned with poor performance. But,
I sincerely believe that the records will show that the vast majority of investigations
are initiated because of poor concrete performance. This does not mean that good per-
formance of concrete is not investigated, but rather that good performance is rarely
the cause for initiation of an investigation. Actually, good performance of concrete is
undoubtedly investigated as much as or more than poor performance, because it is in-
vestigated in conjunction with poor performance. In many instances the investigator
is able to pinpoint the cause or the degree of poor performance by determining wherein
the affected concrete differs from that exhibiting good performance.

I previously mentioned that the objectives of a study of poor performance of con-
crete could vary appreciably, and to illustrate this point I suggest that you consider the
following two objectives:

First: To determine the cause for the poor performance of concrete, and
Second: To determine the location, amount, and degree of deteriorated concrete in a
structure in which the concrete exhibits poor performance.

The type of records that would be assembled for these two studies would be radically
different. For the second study, one would probably not be particularly concerned with
records concerning materials, construction, or maintenance, but would be primarily
concerned with surveys and tests on the concrete. Although this type of study can be
a most complicated one, it usually is not the type that would require, and the needed
answer would not permit, a study of long duration.

On the other hand, a study of the cause of poor concrete performance can be most
lengthy. The cause of poor concrete performance is very likely to be a result of the
following:

1. The standards established by the plans and specifications were not sufficiently
high for the conditions of service.
2. A change in the conditions of service resulted in the established standards being
inadequate.
3. The concrete did not meet the standards established by the plans and specifications.

This may appear to be a short and incomplete list. However, I believe that this list
actually covers most causes of poor performance, and even if it does not, I am sure
that it is sufficient for purposes of illustration.

The term "insufficient standards" covers a multitude of possible causes for poor
performance, such as inadequate (a) design, (b) construction practices, (c) concrete

strength, (d) air requirements, (e) quality requirements for materials, and (f) protection and curing.

Then we have the possibility that the specified standards were adequate for the anticipated service conditions, but that unforeseen changes in service conditions caused them to be inadequate. Examples of such changes are unforeseen increases in traffic (particularly heavy traffic), and/or increased use of de-icers.

Finally, we have the possibility that the concrete did not meet the standards established by the plans and specifications. Here again the possibilities of nonconformance are great, as is also the degree of nonconformance.

From the preceding, it should be obvious that the amount of records that could be assembled is enormous, and many records may not be pertinent to a particular investigation. Consequently, prior to assembling records, the investigator must first determine whether an extensive investigation is warranted. Second, if it is warranted, he should then define the scope and objectives and prepare a detailed outline for future work. This outline should spell out in detail what records should be assembled, what information is needed from condition surveys and how they shall be obtained, and what samples are needed and how they shall be tested by either physical or analytical methods.

I feel sure that most organizations maintain a fairly complete file of the plans, specifications, construction and materials inspection reports and diaries, maintenance records, and traffic counts for most concrete in service. Also, official weather information is usually on hand or can be obtained. Finding records to assemble is, therefore, no serious problem. The problem is to know what records are needed and how to fill in the gaps where needed data are missing.

The initial approach to an investigation of poor performance of concrete is, therefore, to determine with what the poor performance is related or not related. This is usually accomplished by a reconnaissance survey and the assembling of the minimal amount of records. In Missouri the experienced researcher would initially do little more than check the information recorded on the proportioning plant inspector's daily reports. This would provide the researcher with such information as source of materials, concrete proportions, slump, air content, size of batch, air temperature at time of construction, whether the concrete was job- or truck-mixed, and the location of the proportioning plant. With this information and his knowledge of the materials, weather, traffic, construction and maintenance practices, and types of concrete deterioration found in Missouri, the researcher is fairly well equipped to analyze the type and scope of the problem, and to determine what additional information he needs to assemble from construction records, surveys, laboratory tests on concrete samples, or other sources.

If the survey indicates that the poor performance is related to batches or portions of batches, the researcher may well conclude that the problem is not worthy of extensive investigation. If the survey indicates a type of deterioration not associated with batches, the researcher looks for other associations. Do the manifestations of deterioration occur along joints and cracks? Is the deterioration primarily in cuts? Does it occur with a certain type of material? To get a good picture, the researcher may broaden the reconnaissance survey to include concretes in other projects and locations. This will continue until he has a rather clear picture of the type and scope of the problem.

At this point I would add a word of warning about reconnaissance surveys. Although these surveys are neither detailed nor exhaustive, they must be sufficiently thorough to determine whether the concrete deterioration under investigation does or does not occur in specific concretes. Consequently, these surveys must be made with care inasmuch as the data being obtained will be used in outlining the future course of the investigation. Hastily obtained and erroneous data at this stage of an investigation can have a rather disastrous effect on its probable success. For example, when we started our bridge deck investigation in 1959, we were rather suddenly confronted with a serious problem occurring as the development of a fracture plane in the upper part of bridge decks. This problem appeared to be so acute that it was deemed necessary to hurriedly obtain an estimate of the extent of the problem throughout the state. Consequently, engineers inexperienced in the problem were asked to investigate and report as to whether a fracture plane was developing in the bridge decks in their districts. One engineer reported that fracture plane was not a problem in their bridge decks. This report, which was

erroneous, started a chain reaction that almost caused us to ship coarse aggregate entirely across the state. The reason for this almost disastrous action was that the type of coarse aggregate used in the area where fracture plane was known to exist differed from the type of coarse aggregate used in the area where fracture plane was reported to be nonexistent or not significant. So the erroneous report resulted in the erroneous conclusion that the cause of the fracture plane was the use of a particular type of coarse aggregate.

This example not only points up the necessity of making accurate reconnaissance surveys, but it is related to another problem concerning assembly of records. Construction records are often not sufficiently detailed to provide the desired information. In such instances the researcher can frequently find inspectors and workmen who observed the placement of the concrete. Talks with these individuals can be helpful, but at times the most helpful answer that can be obtained is "I don't know". In my opinion, the possibility of obtaining desirable information from such interviews is more dependent on the interviewer than on the person being interviewed. The interviewer is after facts, not conversation. The interviewer should know what information he desires; he should be so familiar with construction practices that he can fairly well visualize what may have happened; and, above all, he should be sure that the one being interviewed is kept at ease.

Actually, in such interviews the researcher is often seeking verification of facts that can be obtained by more costly means. The person being interviewed should be made aware of this possibility, as it may help to remove any reluctance he may have about freely discussing control problems that were encountered during the job but were not recorded in detail.

Having determined the type and scope of the problem and having sufficient knowledge to describe the type or types of deterioration, the researcher is ready to decide whether further investigation is or is not warranted. If additional investigation is warranted, the investigator proceeds to prepare an outline for future work. He specifically states the objectives of future studies and the procedures for attaining these objectives. In doing this, he should refer to the literature pertinent to the problem under study.

The outline could necessitate determining the population of available concretes containing the variables under investigation. Before doing this, however, much time may be saved if consideration is given to limiting the investigation to concretes having approximately the same age, environment, traffic, design, materials, proportions, air content, and de-icing controls. Many agencies have an immense amount of concrete in service, but often the amount available for a statistically sound investigation of concretes placed under normal procedures is extremely small. This is due to the fact that changes in more than one variable are often made simultaneously.

The point is, however, that adequate preplanning can save an enormous amount of time in assembling records. Inadequate planning necessitates the assembling of an overabundance of records or the making of numerous trips to the files for additional data. Good planning permits an orderly assembly of records in the minimum of time, because the outline spells out the rules for selecting the concretes to be investigated and the pertinent records needed. The outline also spells out what laboratory tests are needed, how samples of concrete are to be obtained or made, and how these samples are to be tested.

To select the concretes meeting the specified requirements and to assemble the pertinent information concerning each, it may be necessary to refer to the plans, specifications, construction records, maintenance records, and weather records. The difficulty encountered in this task will undoubtedly depend on the type of records kept and the filing system used by the organization. If we are looking for concrete pavements constructed during a specific period, located in specific areas, and containing one or more variables, we can often obtain the desired list by making several sortings of computer cards. Most of the pertinent information concerning design and construction data, for our concrete pavements built prior to 1952, have been coded and punched on these cards. This procedure greatly facilitates the assembling of records, but it entails a considerable amount of work in keeping the records up to date. We are behind in this work primarily because we have reached a point where considerable time must

be spent in revising the coding system to permit inclusion of new variables. Because there is some question regarding the economic justification for this work, it has not been continued. The disadvantage of the system is that much unneeded data are coded, whereas the advantage is that considerable time is saved in assembling needed data for particular studies.

We do, however, have summary data sheets for all concrete paving projects on file in our research section. These sheets, containing most of the information placed on the computer cards, are filed by year of construction. Because these sheets can be sorted rather rapidly, a list of concrete pavements containing specific variables can be obtained in a short time.

I might mention one more thing about records. It is very unusual for a researcher to need construction records during the first or second year following construction. Surely we would hope that most concretes would last longer than that. However, administrators are faced with an ever-increasing volume of records that they desire to dispose of as soon as practicable. Many records may be disposed of in 5 years, but in investigating concrete the need for assembling records may not occur until 10, 15, or more years after construction. At these ages one may find that a large portion of the records have been destroyed. This is where we find our summary sheets most useful. We wish now that we had summary sheets for the concretes in our bridge decks. These summary sheets have been most valuable to us in making pavement surveys because we try to record on one sheet all the pertinent information concerning the concrete, pavement design, joints, reinforcement, base, and subgrade. We have or should have a summary sheet for each concrete pavement project, and all are filed in one map cabinet. If we had to obtain these data from our files and microfilms, we would have a tremendous task and would very possibly find much data destroyed.

This system (or systems) of preassembling records points up the fact that the assembling of detailed records starts during the construction of a concrete pavement or structure. Once a job has been completed and accepted, it is most difficult to add to the recorded information in inspection reports and in diaries. Being a researcher, I would urge all construction personnel to keep the best possible records.

As has been previously suggested, information assembled from plans, specifications, construction reports and diaries, maintenance records, and other sources may be inadequate to determine the specific cause of inadequate concrete performance. In such instances, the researcher must assemble the necessary records by conducting planned studies. Methods and procedures available to the researcher for assembling the necessary records will be presented in subsequent papers.

In closing I would again stress that assembly of records for investigation of concrete performance should be an orderly process. First, the type and scope of the problem should be determined. Then, if additional investigation appears warranted, a detailed outline of the investigation should be prepared, and the records pertinent to the study assembled. Insofar as possible, the pertinence of records should be determined prior to and not after assembly.

Condition Surveys of Concrete in Service

CALVIN C. OLESON, Wiss, Janney, Elstner and Associates, Northbrook, Illinois;
formerly Portland Cement Association

●THE PRIMARY EMPHASIS in this paper is on condition surveys of concrete in service. It is sometimes difficult, and at times undesirable, to concentrate on concrete as a material to the exclusion of the structure or pavement built with the concrete, but quite often investigations deal with the material only and not with the structure. Usually, individuals or groups work in cooperation with others in making a condition survey, some being responsible for studying the concrete and others, such as structural engineers, being responsible for the survey of the structure. The two fields, though interdependent, require different skills and different investigational equipment. However, in order to study the performance of concrete in service effectively, it is almost essential that the investigation be made on the concrete as it is used. This paper deals with the procedures and techniques that are employed in the field studies.

Briefly, the steps involved in a condition survey of concrete in service may be listed as follows:

1. Define the problem.
2. Ascertain who is involved.
3. Determine the scope and participants.
4. Establish a procedure.
5. Plan the investigation.
6. Conduct the investigation.
7. Analyze and report the findings.

The application of these steps, through use of a few examples of field surveys, will be the objective of this paper.

In a way, a field research engineer is a diagnostician, a "doctor of concrete". A condition survey may be made on concrete in any condition, from new and unused to severely deteriorated concrete that is no longer useful structurally. Every investigation requires its own detailed procedure, and each investigator will approach the problem in his own way. Condition surveys may be of any extent, ranging from an investigation of a single small element to a nationwide study, such as the bridge deck survey that was undertaken by the Portland Cement Association (PCA) in cooperation with the Bureau of Public Roads and a number of state highway departments (1).

The procedures followed in conducting a bridge deck survey have been presented a number of times, but a brief review will illustrate how such an investigation is made. The major objectives of this survey were (a) to determine the types and extent of deck deterioration in selected areas, (b) to determine the causes of the various types of deterioration, (c) to develop methods for preventing deterioration of future construction, and (d) to develop methods for retarding deterioration of existing bridges now showing deterioration.

It is essential that the reasons for a condition survey be clearly understood and stated. In order to answer the first of the objectives as just stated, it was necessary to examine bridge decks to find out what was wrong with them; in other words, to define the problem.

Condition surveys were made in two ways: (a) random surveys on approximately 100 bridges in each of 12 states by local representatives of the Portland Cement Association, the Bureau of Public Roads, and the particular state highway departments, who were provided with a carefully prepared set of instructions, including definitions and appropriate photographs of each type of deck deterioration, and a form to fill in for each bridge; and (b) detailed surveys of approximately 12 bridges in each of five states

that were included in the random surveys. The detailed surveys were made by a team of observers from the PCA Structural Bureau and the Research and Development Laboratories accompanied by representatives of the Bureau of Public Roads and the selected state highway departments. For this inspection, sketches were made of each span showing location and description of every defect on the deck. Cores were taken from each deck in carefully designated locations.

The data from both the detailed and random surveys were carefully analyzed and reported. One report provides an analysis of the observations of all the bridges visited, and should answer the first objective—types and extent of deck deterioration. In spite of very careful planning in preparation of the instructions, individual interpretations were made by the different inspection teams, and it has been difficult to analyze the random data. For uniformity in reporting the random survey data, a representative of the PCA Structural Bureau or of the laboratory probably should have accompanied each of the local survey teams.

Details of the bridge deck investigation have been purposely omitted, but it clearly qualifies as an unusually comprehensive condition survey of concrete in service. It was carefully planned and conducted and the information, when available, will be authentic and valuable.

For the bridge deck survey, the answers to the seven previously listed steps would be as follows:

1. The problem: An investigation of types and causes of concrete bridge deck deterioration.

2. Who is involved: PCA, the Bureau of Public Roads, and the selected state highway departments.

3. The scope and participants: Twelve states, the Bureau of Public Roads, and PCA.

4. The procedure: Agree on the need for the investigation and secure the cooperation of the several organizations.

5. The plan: Review the literature on other studies of a similar nature, and prepare a brochure on details to be investigated.

6. The investigation: Send brochures and forms to all states included in the random survey; select personnel from PCA and the cooperating organizations for making random and detailed surveys, and carry out the surveys.

7. The analysis and report: Make petrographic studies, assemble all available data, analyze, prepare a draft report, review with the cooperating organizations, and publish.

Another example of a survey, conducted on a less comprehensive scale, resulted from the occurrence of extensive pattern-cracking observed in two southeastern states (Georgia and Alabama) about 20 years ago. The first evidence of this cracking was reported by a regional bridge engineer of the Bureau of Public Roads who was making routine inspections of bridges in his area. The pattern-cracking was then observed in numerous other bridges in this state and a preliminary analysis by the particular state highway department pointed to an incompatability between certain aggregates and cements. A meeting of engineers from the state highway department, Bureau of Public Roads, and PCA was arranged, with a field inspection of about 20 selected bridges and a discussion of what to do about the problem. Samples of concrete from one of the affected bridge elements were found to contain copious deposits identified as alkali-silica gel, the by-product of alkali-silica reaction. A full-scale investigation of bridges in two states was suggested, the author being assigned the task of making the surveys.

In order to be completely impartial and fair in making the survey, it was decided that a relatively complete coverage of the two states would be required, with an examination of every bridge encountered. For rating purposes, four classifications of condition were established: Classes I and II were considered as being without the typical pattern-cracking, and therefore unaffected; and Classes III and IV demonstrated pattern-cracking, light and heavy respectively, and were considered as being affected. In the early stages of the survey, the author was accompanied by a representative of the particular highway department, and bridge classifications were a mutually agreeable value.

It was found that if a structure was affected with the pattern-cracking, the cracking could almost invariably be observed in the concrete end posts of the handrails. Each bridge in each state was given a sequential number and was identified in any other way that would make fruitful a search for highway department records relative to that bridge. Sampling of the cracked concrete was part of the program, and it was reasoned that, because the end posts were rather hopelessly damaged anyway, it would not hurt to remove any amount of concrete necessary to find evidence of the reaction. A letter from the chief engineer of the highway department was provided, authorizing removal of such samples at the author's own discretion, in case there should be any question about someone from out of state breaking up bridges.

Some of the bridges involved were almost a mile in length and ranged in height above the ground or stream from 20 to over 70 ft. Inspection of the substructure was difficult from the top of the bridge, but this problem was solved by use of a swinging basket that was suspended from a truck with a power-operated boom. The truck moved along the bridge roadway and the basket was raised or lowered as necessary to permit the inspection. For some of these basket inspections, a portable recorder proved to be very helpful in making a running record of what was observed. The record was transcribed as time permitted.

Bridges in these two states were among the first field structures to be included in the Portland Cement Association's soniscope test program. Trucks, scaffolds, ladders, and manpower were provided to assist in the work whenever necessary.

After the field work was completed and structures had been rated, a search for construction records was made and the materials used in the concrete were determined. For the ratings, only the most severe condition found in a given structure was used, even though such a condition may have been limited in extent. For example, several of the end posts may have been severely cracked, with a rating of Class IV, while the balance of the concrete would be no worse than Class III, or even Class II, but the structure would be rated as Class IV.

Where the records showed that two cements had been used without a specific record of where each had been used, the bridge was not accepted for analytical purposes. However, where two different aggregates were shown, such as a gravel and a granite, the elements in which each had been used were easy to separate by the field inspection. The results of this work have been reported elsewhere (2).

PCA has cooperated in making comprehensive condition surveys of concrete dams in the United States. F. R. McMillan participated in an initial survey more than 30 years ago, and helped to make repeated observations of the same dams in 1957. The record for each dam included a visual description of conditions found by the observers supplemented by photographs. These observations have been included in a series of reports that are available for future reference as needed (3).

The usual procedure followed in making these surveys was for an inspection party, including McMillan and several companions, to visit the dam and make detailed visual observations. Other tests were generally not made during these inspections, but any applicable construction data, where available, were recorded. These condition surveys provide a permanent source of reference material for use at any later date.

A final example illustrates a condition survey made on a single structure. Early in 1965 an investigation of dams in the Los Angeles area by a team of consulting engineers disclosed that a relatively small concrete arch dam (165 ft high by 620 ft long) in Ventura County had developed signs of pattern-cracking near the top, and there was some evidence that one of the abutments might be shifting. Further studies by a consulting engineering firm resulted in three alternatives with respect to the dam:

1. Remove defective concrete in the dam and replace with better concrete;
2. Remove most of the top 20 ft and permanently lower the spillway crest; or
3. Remove the dam completely.

This much of the study was reported in the April 22, 1965, issue of Engineering News-Record.

Articles in the Los Angeles papers and one technical journal then reported that the Ventura County Board of Supervisors had ordered that the dam be demolished. PCA

was requested to make additional tests of the concrete using its soniscope. Arrangements for the tests, including the furnishing of equipment for going down both faces of the dam, were made by the Los Angeles office of the Portland Cement Association in cooperation with the Ventura County engineers in charge of the dam. Two engineers from the PCA Field Research Section visited the dam, which had been dewatered pending a decision as to its future.

A visual inspection showed that several of the upper lifts of concrete, under the spillway crest, were definitely, although not severely, pattern-cracked. For the evaluative tests, locations were selected at each end of the dam where there was no evidence of cracking, and at other intermediate locations that included pattern-cracked sections. The soniscope tests were made from upstream to downstream faces, radially through the arch concrete. Pulse velocity through undamaged concrete was established at about 13,500 fps plus or minus a few hundred feet per second. The tests also showed that pulse velocities dropped to as low as 9,300 fps in the cracked sections. The soniscope tests clearly showed that the concrete, except for the cracked sections, was in excellent condition and that there should be no fear concerning the safety of the structure so far as the undamaged concrete was concerned.

The Ventura Star Free Press carried an account of our tests and a statement that the dam would be saved but that two notches 30 ft deep and 140 ft long would be cut from the top of the arch, thus lowering the spillway crest and the pressure against the dam. This treatment was later reported (6).

In this case, the condition survey was limited to measuring the quality of the concrete in the dam and had nothing to do with other factors such as foundation stability. However, the tests provided assurance about the concrete that would have been very difficult if not impossible to learn in any other way.

SUMMARY

1. A survey may be made on concrete in any condition, from new and undamaged to completely deteriorated.

2. The objectives of the survey should be clearly defined.

3. Preliminary plans for the survey should be thoroughly laid out.

4. The field work will almost invariably be cooperative in nature, with the owner providing necessary clearances and authorization for making the investigation.

5. Whenever possible, and to such an extent as they are available, records of construction, including any data on materials, should be consulted and studied.

6. Sampling is generally necessary in order to provide information on any abnormalities of the concrete. An on-the-site investigation will sometimes reveal the presence of deteriorating influences, but usually a more thorough laboratory study is required.

7. Soniscope tests have been very useful in making nondestructive field examinations of concrete.

8. Complete impartiality and objectivity is of utmost importance in making a condition survey.

9. The records obtained in any survey must be kept in an orderly manner and in such a way that they may be filed for future use. A bound field book is excellent for permanent filing, but loose leaves are sometimes more suitable, depending on the circumstances.

REFERENCES

1. Klieger, Paul, and Fountain, Richard S. A Cooperative Bridge Deck Study. HRB Bull. 323, 1962, pp. 23-25.
2. Oleson, C. C. Abnormal Cracking in Highway Structures in Georgia and Alabama. ACI Jour., March 1963, pp. 329-352.
3. Symposium on Mass Concrete. ACI Spec. Publ. 6, 1963.
4. Concrete Arch Must Undergo Surgery. Engineering News-Record, April 22, 1965.
5. Dam Owners Hope Notch Cut Today. Engineering News-Record, Dec. 23, 1965.
6. Swaybacked Dam Passes Its Exam. Engineering News-Record, Sept. 7, 1967.

Sampling of Concrete in Service

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●THE SAMPLING PHASE of the investigation of concrete in service, for the evaluation of its serviceability or the diagnosis of its ills, cannot be separated from the other phases of the overall investigation. This is the case because, in order to sample efficiently, one has to decide what the samples are to be used for and what possible tests or measurements are required to provide the needed information. And, one cannot realistically know these things until after one has developed the history of the particular concrete, assembled all available data that could have a bearing on the end result being sought, and carefully examined the concrete, the site, and the environment in detail. Only after these items have been studied in relationship to one another, and with a tentative plan for the tests that could provide dependable answers, can one develop a realistic sampling plan that will provide the most information for the minimum number of samples at the lowest overall cost consistent with the dependability of the answers to be obtained.

In effect, the investigation of concrete in service requires the systems approach to provide dependable answers at minimum cost. Sampling is only one unit of the system and has to be given its proper place, keeping in mind an overall perspective. This discussion will, however, limit itself to the sampling phase of the system, leaving the remaining facets to be discussed by other authors. It is also limited to concrete in service only. In other words, the discussion is limited to hardened concrete in place in a structure, even though the structure may not have been put to use at the time of sampling.

BASIC PRINCIPLES

What is a sample? A sample may be defined as a unit or portion of a unit that forms part of the universe or population; that is, part of the total structure or portion of the structure or volume of material under consideration. Such a sample is taken as being evidence of the properties, characteristics, or composition of the universe at the particular point or location sampled. A sample taken a few feet away may have different properties, characteristics, or composition, simply because of the inherent variations in materials, workmanship, process, and exposure. Variation is the law of nature, and the closest to uniformity one can come in engineering work is when dealing with a relatively small volume of liquid that is being continuously stirred to keep all portions the same.

One notes in specifications, in various standards, and in test methods a requirement that a representative sample be obtained. Is such a thing possible? In the theoretically uniform liquid described above, a sample taken at any point may be assumed to be representative of the whole. But when it comes to concrete, one soon finds that samples taken from two successive batches differ in their characteristics. On closer observations, one determines that samples taken from two parts of the same batch will also exhibit differing characteristics. When such concrete is placed in a structure, additional variations in placement, consolidation, curing, protection, and exposure tend to increase the original variability of the plastic concrete. This means that it is impossible to obtain a representative sample of a significant volume of concrete or of a significantly large portion of a structure. Therefore, instructions to obtain a representative sample of concrete are unrealistic, to say the least.

All a sample represents is itself and that it is part of the universe or structure or portion of the structure under study or investigation. How then can one take a sample that represents itself only as part of the overall picture and determine what the characteristics of the whole might be? The process is not as difficult as might appear.

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Engineers are accustomed to making a series of observations and plotting each obser-vation as a point. After enough points have been plotted, a curve representing the trends of these points can be drawn. The curve can then be expressed in terms of a mathematical equation that accurately reflects the relationships developed by the sum total of the observations.

Each point, representing a pair of observations, represents itself as being a point on the curve or close to the curve, but never representing the whole curve. In the same manner, a series of samples, each representing itself as being part of the larger whole or universe, will provide observations that, taken together and properly analyzed, permit the estimation of the parameters or characteristics of the whole. This is be-cause the variations among the samples (when properly and randomly taken) will reflect the variations of the whole or universe.

To achieve this, with any reasonable degree of reliability or confidence, requires first that the series of samples be taken in a random manner; that is, that the location or time of sampling of any one sample not be determined by the decision or selection of an individual, but rather that such locations be determined strictly by a process of chance not influenced by any individual. Second, it is necessary to take a sufficient number of samples in the series to provide the desired reliability of estimating the characteristics of the parent whole or universe.

SAMPLING PLANS

General Plans

To apply these basic principles to sampling by a producer or contractor for the con-trol of his production or by the owner for determining acceptance characteristics of the product he is buying, it is relatively simple to assume the level of variability either from past experience or from actual records, to decide on the confidence limits or reliability with which one desires to estimate the parameters of the universe or whole, to assume acceptable levels of the producer's risk and the purchaser's risk, and then simply to use the applicable statistical calculations to develop a sampling plan. But the application of this procedure to concrete in service becomes very expensive and, for most purposes, unnecessary. Each sample of concrete in service is more expen-sive to obtain and test than samples of concrete at the production and placing stages. And sampling plans, developed as previously indicated for control or acceptance, will result in many more samples than needed to solve the problems encountered in inves-tigating the behavior or service record, or to solve a problem regarding distress of con-crete in service. If one applies strict statistical procedures, one has not only more expense in sampling, but also unnecessary sampling, thus compounding the costs. Thus, for concrete in service, the selection of a sampling plan in strict compliance with a sound statistical procedure is not practical, either physically or economically.

How can one then adapt these basically sound sampling procedures to the investiga-tion of concrete in service, keep costs within reason, and at the same time have con-fidence in the results? The first step is to determine the purpose of the sampling. In general, there are two reasons for sampling concrete in service:

1. Sampling to guide control, or to learn the reason a given structure is performing properly and adequately; and
2. Sampling to diagnose trouble or signs of distress, learn what the causes may be, and reconstruct the responsibility.

Sampling a Properly Functioning Structure

Funds for studying a "healthy" structure are rarely available, and when they are, they are very limited. Ostensibly, there are no differences in the outer appearance of the con-crete in the structure; otherwise it would immediately be classed in the second category. Here one can simply take the funds available, decide on the tests to be made to provide definitive answers, figure out the cost of taking each sample and testing it, and thus arrive at the number of samples feasible under the circumstances. It is then easy enough to devise a random sampling procedure by dividing the structure into subsections.

Samples can then be taken at points determined at random within each section or random section by the use of random numbers, drawing cards, numbered chips, pieces of paper that have been shuffled in a container, spun in a toy roulette, or whatever the investigator's ingenuity can come up with that will ensure pure chance.

A good example of this type of work was the examination of cores on an Illinois toll highway. In this case, cores were taken regularly on a routine basis for the measurement of thickness of the concrete, which was standard practice to determine thickness acceptability. A core was taken for each 2,000 linear feet of each lane. On a routine basis, a specific fraction of these cores was picked at random and examined petrographically for such things as percent of entrained air and its distribution, aggregate segregation, laitance, carbonation, evidence of bleeding or excessive water content, possible reaction rims on aggregate particles, clay balls, and any other characteristics of interest that could be observed by the petrographer. The purpose was to call attention to any irregularities and to check the effectiveness of the field control, even though there was no evidence of any problem areas that needed to be investigated specifically. This also formed a record to correlate with in case future problems developed, the tested cores being such that they could be related to future distress areas in case these occurred.

Sampling a Poorly Functioning Structure

The "sick" structure seems to be the financially favored child; someone is always willing to supply the funds to find the cause or causes of the trouble, where the responsibility lies, and what to do to repair it. It is always much more expensive and less effective than the other approach, because some damage has already been done that cannot be undone and that remains there as evidence. A reliable sampling plan is more difficult to develop in this case, yet rarely is it practical to apply the full statistical methods developed for control or acceptance purposes.

The first step is to study the records to see if one can pick up any irregularities thay may produce the symptoms on hand. The second step is to examine the structure carefully and mark out the areas or portions in which the symptoms or distress occur in varying degrees of intensity. A numerical scale to describe degrees of distress that has proved useful to the author is as follows:

1. Absent,
2. Faint,
3. Moderate,
4. Marked,
5. Pronounced, or
6. Severe.

But any other scale that serves the purpose of the investigator can be used. If the structure was constructed in two seasons or in cold weather and hot weather, then areas of varying severity of the symptoms should be marked out in each construction unit for which the conditions were reasonably the same to determine if variations in environmental conditions had a significant bearing on the problem.

Once areas have been thus delineated, then it becomes a simple matter of random sampling within each area to provide samples that represent individual points in the structure and that appear to have the same ranges in a variety of characteristics, symptoms, or problems. The number of samples, here again, is a matter of judgment and realism in the expenditure of funds. One can spend justifiably much more money in diagnosing a $100 million structure than in investigating a $100,000 piece of construction.

An example of this kind of sampling is a case where, a few weeks after construction, random overall "boils" or "blisters" that looked like burned spots appeared on the surface of a pavement that had been constructed in winter. The popular reaction of management was that the contractor should not have been permitted to cut reinforcing steel with an acetylene torch while the steel was resting on the concrete. It so happened that there was no steel used in the pavement and none around the project. An examination revealed that these blisters occurred randomly, and the areas could not be separated

into areas of varying severity. The typical surface of the blister was raised slightly over the concrete surface, looked whiter than the concrete, and was surrounded by a darker ring where it blended into the concrete that appeared normal. When the blister was tapped, it sounded hollow and the center was friable, whereas the normal-appearing concrete sounded solid, like dense, hard concrete normally sounds. Samples were taken from several of these blisters at random and analyzed chemically. It was found that these blister areas contained calcium chloride in the proportion of approximately 20 percent of the cement.

The specifications had permitted 2 percent calcium chloride in the form of a solution for this winter work. The resident engineer and the contractor had felt that this restriction was just a hindrance to construction progress, so they added the calcium chloride dry. The supply of this salt was apparently badly lumped because of poor storage under moist conditions, and some of the lumps had not broken down in the mixer. These, being lighter than the rest of the mix, had floated to near the surface of the slab. To test this hypothesis, slabs were made in the laboratory, and handfuls of calcium chloride were inserted at different points just below the surface. The slabs were then placed outdoors to weather. Within a few weeks the same blisters appeared. When one arrives at a hypothesis in diagnosing concrete symptoms, one knows he is on the right track if he can duplicate the symptoms experimentally at will.

SAMPLING METHODS

In general there are three methods of taking the actual samples of concrete in service:

1. Coring;
2. Cutting prisms with a diamond saw; and
3. Using nondestructive testing.

Coring is the simplest, quickest, and least costly of all methods of actually taking samples from concrete in service. The breaking of samples by use of a crowbar or sledge hammer is most undesirable, as the process is likely to damage the sample so that one can never be sure that something being observed is a characteristic of the concrete in the structure or was inflicted on it in the sampling. The digging into the concrete to see what is wrong usually destroys the evidence being sought and should never be undertaken.

As a practical matter, a sample may constitute more than one core or piece of concrete so as to provide the necessary quantity of concrete for use in the several tests and analyses that may be contemplated. So far as possible, each test or analysis should be performed on previously untreated and undisturbed concrete, because one test may invalidate the specimen for use in ensuing tests. Where, for some reason, a larger sample than a core is desired, prisms can be cut from slabs by means of diamond saws, and serve the same purpose as cores except for size.

The determination of the proper locations at which nondestructive tests (Schmidt hammer, resistivity, seismic, sonic, etc.) are made is essentially a process of sampling. Such locations should be selected by the same principles outlined for core or prism samples.

SUMMARY AND CONCLUSIONS

This discussion of the sampling of concrete in service may be summarized as follows:

1. There is no such thing as a representative sample of concrete in service. There are only samples that represent individual points.

2. It is neither practical nor economical to utilize the full statistical approaches to reliability, as used in sampling plans for control of quality in production or in acceptance sampling, when one is sampling concrete in service.

3. The most practical approach is to classify the symptoms according to severity of occurrence and take random samples within each such class.

4. A random sampling plan in the various areas of similar symptoms is necessary to provide a series of samples that reflects the variations in the overall total structure.

5. This means that judgment is essential to the success of such investigations. It is not a field for a novice. One must know concrete, construction, design specifications, materials, and foundations, and must have developed skill in relating cause and effect. In other words, one needs experience that develops insights into these interrelationships as a system. In fact, concrete itself is a system within the larger system of the construction of the structure.

ACKNOWLEDGMENT

Physical Tests for Investigating Performance of Concrete

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●A THOROUGH INVESTIGATION of concrete in service usually includes some physical
or analytical testing in addition to visual examination. It is true that many of the de-
teriorating influences that affect concrete leave characteristic marks that can be used
as visual indicators of the condition of the concrete. However, when involved in a for-
mal investigation of concrete in service, the participating investigators are not likely
to be satisfied with a diagnosis based on appearance alone. Because concrete is a com-
plicated material, any single visual feature can usually result from a number of differ-
ent causes. Thus, records are assembled, samples are taken, and tests are conducted
in order to provide a better basis for reconstructing the history of the concrete, for
comparing concrete from different structures or from different locations of the same
structure, or for judging the adequacy of the concrete to continue to fulfill its function.
The test results are presented in numerical terms, and this usually makes a more
convincing report than opinion or logic alone.

For the purposes of this discussion, three classes of physical tests that can be made
in connection with an investigation of field concrete are considered: (a) nondestructive
tests, (b) standard tests on cored or cut sections, and (c) tests of concrete constituents.

NONDESTRUCTIVE TESTS

In recent years, considerable attention has been given to the nondestructive type of
test. Such tests have advantages in that they can be made directly on the concrete in
situ, thus eliminating the need for coring or sawing. When desired, however, most
nondestructive tests can also be applied to specimens of the usual size. In situ test
results are generally free from the influence of boundary effects, which could be a fac-
tor when specimens of limited size are tested. Nondestructive tests can usually be
performed rapidly, and therefore can provide a large number of readings of the prop-
erty of interest in support of a statistical average for any selected mass of con-
crete.

Sonic Testing

Much work has been done toward determining the condition of concrete in service by
measuring the velocity of pulses of sonic or ultrasonic vibrations transmitted through
it. In theory, the speed of sound in a material is related to its dynamic modulus of
elasticity. Thus, any condition in concrete that results in a change in this property,
such as most forms of deterioration, should be reflected by changes in the velocity of
sound waves.

Numerous instruments (1, 2) for measuring pulse velocity have been developed,
such as the well-known soniscope, the electronic interval timer, and combinations of
the two. In essence, these devices provide a means for transmitting and receiving
sound pulses, and for measuring the time required for them to travel through a known
thickness of the concrete, thus making it possible to calculate sound velocity.

The reliability of sonic data is largely determined by the skill and experience of the
operator in choosing appropriate test points and in observing those conditions that might
influence the measurements. Such factors as changes in density, moisture, aggregate-
paste ratio, and the presence of voids and cracks should all be given weight in the final

analysis. Properly interpreted, sonic techniques have been found useful for acquiring the following types of information regarding the condition of concrete:

1. Measure of uniformity within a structure, particularly with regard to defining the extent of observed deterioration.

2. Measure of changes in the condition of concrete with time by repetition of tests.

3. Measure of general level of quality. For example, velocities of more than 12,000 fps have been suggested as indicating a sound condition, whereas values of less than 10,000 fps generally indicate poor condition (3). Such an evaluation can be useful in determining whether specimens submitted in connection with an investigation do actually represent different levels of quality as intended.

4. Measure of depth and extent of cracks, by taking into account the fact that sound waves pass around cracks or voids and therefore travel a longer path than if the discontinuity were not there.

Sonic techniques can also be applied to determining the thickness of concrete sections such as pavement slabs. Muenow (4) has reported on a technique that requires the determination of both the wave velocity in the concrete and the longitudinal resonant frequency of the slab vibrated in the direction of its thickness. When allowances are made for surface cracks or other irregularities, thickness determinations by this method have been found to agree within 2 or 3 percent of that determined by core measurement.

Impact Testing

Many devices or techniques for determining the approximate strength of concrete by its reaction to impact have been used, particularly in European countries. These include the blow of a standard hammer, a pistol shot, use of a hydraulic jack, a steel ball and hammer, pendulum devices, and various ball impact tests. The test measurement is either the size of the indentation resulting from the impact or the rebound.

An instrument that has received a great deal of attention in this country is the Schmidt hammer, which determines the compressive strength of concrete by measuring the rebound of a spring-driven steel plunger. Because of its obvious virtues of rugged construction, simplicity, and rapidity of operation, and the fact that it seemed to fill a critical need, this device has been used extensively in the field. However, many users of the Schmidt hammer have expressed disappointment in the results obtained. Not only do individual determinations usually indicate widely varying compressive strengths, but also average strengths may not correlate well with actual strength determinations of cores or cylinders. Such results are not particularly surprising when proper consideration is given to inherent limitations of the instrument and the nonhomogeneity of concrete.

A common source of difficulty is the reluctance to calibrate the instrument for each source or type of aggregate. The practice of using the manufacturer's curve for converting impact readings to compressive strength, regardless of the aggregate involved, can lead to considerable error and frequently accounts for the difference in strengths indicated by the impact hammer and the strengths of cores. A serious limitation of the instrument is the fact that each individual test is influenced by only a very limited mass of concrete near the point where the impact is applied. Much of the variability between individual results is related to the position of large aggregate particles with respect to the point of impact. When the test is made on a formed or finished surface, it should also be recognized that the quality of the concrete near such surfaces is often different from that in the interior of the concrete. When so used, there may be difficulty in interpreting the results, but the test hammer may not be in error when it indicates strengths differing from core strengths.

To illustrate these points as well as to show how the Schmidt hammer can be used to gain information not readily obtainable by other methods, data obtained during the course of an investigation of a concrete pavement failure are shown in Figures 1 and 2. This pavement was showing surface raveling after only 2 weeks of use. Figure 1 shows the variation in impact readings in one of the cores representing concrete still in good
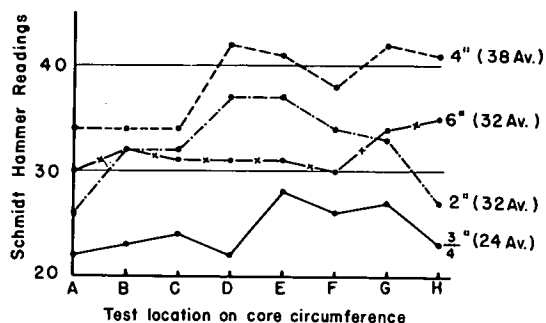
Figure 1. Variation of strength within 8½-in. pavement core.

condition. Not only did the indicated strengths vary around the core at the same depth, but the average level of strength varied greatly at various depths, being lowest near the surface of the pavement. Figure 2 shows comparison data on cores taken both from raveled and adjacent sound areas. In addition to showing variability, the several comparisons indicate that surface raveling was associated with an impact reading of 24 or less.

Locating Reinforcing Steel

Information is often desired on the location and depth of cover of reinforcing steel. At least two instruments of European manufacture have been developed to make such determinations in a nondestructive manner. The position and depth of steel are indicated by a meter deflection according to the effect of the steel on an electromagnetic field developed by the instrument. One instrument has been used extensively by the Georgia State Highway Department and is reported to provide very accurate readings for up to 1 in. of embedment and fairly close readings between 1 and 2 in. The Portland Cement Association (5) has reported on the use of another make of instrument that comes in two sizes—one for use where the cover is less than $1\frac{5}{8}$ in., and a more powerful model for cover measurements up to $4\frac{3}{4}$ in. The depth of cover could be measured to an accuracy of $\frac{1}{16}$ in. up to a 2-in. cover and to $\frac{1}{8}$ in. between a 2- and 3-in. cover.

Density by Radioactive Methods

The British have experimented with the measurement of the density of concrete by radioactive methods. When X-rays or gamma rays pass through concrete, they are partly absorbed and partly scattered, depending on the density of the concrete. When both sides of a concrete member are accessible, the density can be determined by the degree of radiation absorption through a known thickness up to thicknesses of 3 ft. When only one surface is accessible, as in the case of a pavement slab, the amount of backscatter is used as an indication of density.

Because the density of concrete is affected by many factors, such as air content, kind of aggregate, moisture content, and mix proportions, there is considerable difficulty
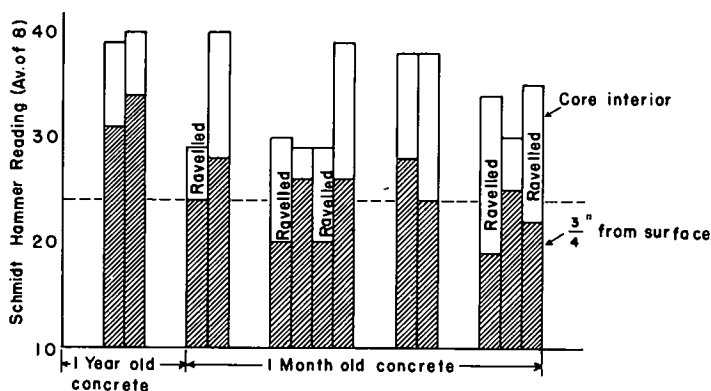

Figure 2. Strength vs raveling of pavement.

to be expected in using this property of concrete as in indicator of deterioration. Preiss (6) states that an accuracy of 0.5 percent is possible by the transmission method and suggests that no greater accuracy is necessary in view of the heterogeneity of concrete. He indicates that the method can be useful for location of steel reinforcement, voids in grout, segregated zones, and poorly consolidated concrete.

STANDARD TESTS ON CORED OR CUT SECTIONS

The tests that are made on cores or beams taken from a structure are in many cases the same tests that are made on molded specimens. When interpreting test results, however, there are differences between molded and cut specimens and between both the types of specimen and the concrete in the structure to be considered. One important feature of specimens taken from a structure is that they have the same curing history as that of the structure. In this respect, at least, tests on such specimens are more truly indicative of the condition of the concrete in the structure than are tests of specimens molded from the same concrete, because it is virtually impossible to provide curing equivalent to that provided by the large mass of concrete. On the other hand, both molded and cut specimens of concrete are subject to boundary effects that may cause them to act differently from the concrete in the structure. This is even more true of cut specimens than molded specimens because many aggregate particles in cut specimens are not completely surrounded by cement paste and therefore may tend to act as discrete particles rather than as an integral part of the concrete.

Strength

Strength tests are usually performed in accordance with the ASTM Method C 42, Obtaining and Testing Drilled Cores and Sawed Beams of Concrete, and are particularly useful for determining relative strengths in different parts of a structure. However, when such tests are used to indicate the actual strength of the concrete in situ, or are compared with the strengths of standard molded specimens, certain limitations should be recognized. For example, Method C 42 requires specimens to be immersed in water for at least 40 hours prior to the test or in a moisture condition other than 40 hours as directed by the testing agency. Such condition is desirable for the purpose of standardizing the test procedure, but it also has the effect of lowering the determined strength below the strength that would be obtained if specimens were tested in a drier condition, such as would be likely to exist in a structure. The drilling or sawing of a specimen, as opposed to molding, can have a sizable effect on the strength of the specimen, as can variation in size and shape of specimen.

Compressive Strength—The relationship between the compressive strength of 6-in. drilled cores and molded cylinders made with the same concrete ($^{3}/_{4}$-in. aggregate) and similarly cured is shown in Figure 3 (7). It is observed that the ratio of strengths varies with the strength of the concrete. However, it is fortunate that at the lower strength levels, which would be of greatest concern in an investigation, the strengths of the two types of specimen are not greatly different. The method provides for the use of cores of different sizes and length-diameter ratios. But if the provisions regarding these factors are observed, the differences in strength as a result of these variables are also not large enough to require special consideration when interpreting test results.

ASTM Method C 42 also mentions the determination of compressive strength by testing
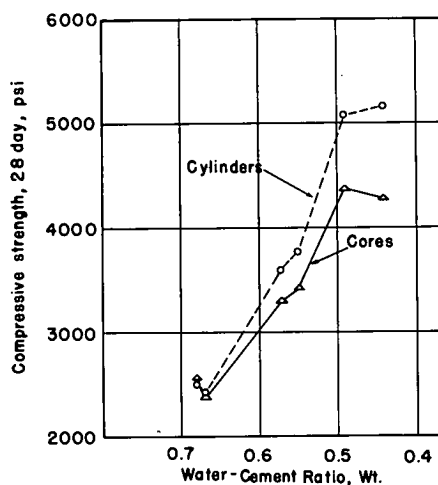

Figure 3. Relation of compressive strength of 6 by 12-in. cylinders and cores similarly cured.
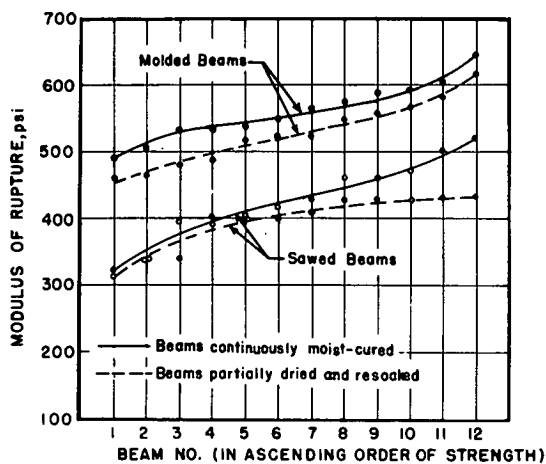
Figure 4. Relationship between flexural strengths of sawed and molded beams of same concrete.

portions of beams as modified cubes in accordance with ASTM Method C 116, Test for Compressive Strength of Concrete Using Portions of Beams Broken in Flexure. Strength values obtained by this method differ significantly from those obtained on the standard 6 by 12-in. cylinder according to the strength of the concrete. Kesler (8) developed a statistical relationship between results on the two types of specimen that indicated that in general the cube strengths exceeded the cylinder strengths by more than 10 percent. Although Kesler's work was based on molded specimens, it is assumed here that the same relationship would apply to sawed beams.

Flexural Strength—Unlike the approximate equivalency found to exist between the compressive strengths of cores and cylinders at certain strength levels, it has been found that sawing significantly reduces the flexural strength of beams below that obtained on the same size of molded specimens at all strength levels. Figure 4 shows data developed by Walker and Bloem (9) on beams having a 3 by 4-in. cross section and containing a quartz gravel aggregate. For similar curing conditions, the modulus of rupture of the sawed specimens averaged about 25 percent less than the molded beams. It is further shown that intermittent drying, such as might be experienced by specimens from a structure, is more damaging to sawed specimens than to molded specimens. Further studies by Bloem (10) showed that the effect of sawing was not related to type of aggregate or size of beam but was related to the location of the sawed surface with respect to the tension side, the strength reduction being greatest when a sawed surface was in tension.

The use of the splitting tensile test apparently has not been proposed for determining the tensile strength of drilled cores. However, certain aspects of this test appear to recommend it over the flexural test for determining tensile strength of concrete where cut specimens are involved. Because the zone of maximum stress in the splitting tensile test does not occur at the surface of the specimen, it is unlikely that there would be any effect on strength as a result of the cutting of aggregate particles during drilling. However, because the bearing edges of such a specimen would be along a cut surface, the effect of such an edge, or the need for special leveling treatment of the specimen in this regard, would have to be determined.

Elasticity

Although concrete cannot be considered a perfectly elastic material, it is enough so that the theory of elasticity can be applied to it within limits of stress and time. Thus, ASTM Method C 469 outlines a procedure for determining the static Young's modulus of elasticity of cores or cylinders in compression as a chord modulus where the maximum stress does not exceed 40 percent of the ultimate load. By this limitation on stress and the prescribed rate of stress application, the conditions under which creep can come into play during the test are standardized. The modulus of elasticity can also be determined from data obtained dynamically. ASTM Method C 215 describes the procedure for determining the fundamental transverse, longitudinal, and torsional frequencies of concrete specimens from which the dynamic moduli can be calculated. Because the specimen is essentially in an unstressed state during the test, the influence of creep is not a factor and, as a result, the modulus of elasticity determined dynamically tends to be higher than that determined statically.

Static determinations of the modulus of elasticity provide values that are useful for design purposes, such as determining deformation and stress distribution between concrete and steel in reinforced or prestressed concrete members. The static modulus of elasticity is also useful for calculating the stresses resulting from shrinkage, settlement, or other distortions. Thus, for investigative purposes, the elastic constants determined statically will be of interest primarily where some aspect of structural performance is in question.

Dynamic elasticity tests, on the other hand, yield values that are widely used as an index of concrete quality. Although there is no generally applicable relationship between the strength and elastic properties of concrete, these properties are affected by the same factors in the same manner; that is, a deteriorating influence that lowers strength will also lower the modulus of elasticity. The dynamic modulus of elasticity by resonant frequency, therefore, is useful for investigating the quality of concrete in specimens of known size and weight in the same way that sonic tests are useful in studying concrete in situ.

Air Content

An investigation of the performance of concrete, particularly where durability with respect to freezing and thawing is a factor, often requires information on the air-void system of the concrete. Determination of all the important air-void parameters of hardened concrete requires analysis by optical methods, but if only the total air content is required the so-called high-pressure method can be used (11).

In this method, a known volume of concrete in a saturated condition is subjected to hydraulic pressure of 5,000 psi. The apparatus measures the change in volume of the entrapped and entrained air in accordance with the principles of Boyle's law as the pressure is increased from atmospheric pressure to 5,000 psi. The method is convenient in that any size or shape of specimen can be used that will fit into the pressure chamber, no special treatment of the specimen is required other than moisture conditioning, only about 15 minutes are required for the actual air determination, and the procedure is relatively free from the effects of personal technique.

Air contents of hardened concrete by the high-pressure method have been found to be in approximate agreement with air contents determined on the same batches of concrete plastic, within the range of air content normally specified for air-entrained concrete. For air contents of less than 3 percent, the high-pressure method generally indicates a slightly higher value than that determined on the same concrete in a plastic state. The high-pressure method has also been compared with optical techniques by several investigators. Table 1 (12) gives data developed by the Corps of Engineers that indicate reasonably good agreement between the two methods. Comparisons by other investigators have not shown such good agreement, variously indicating both lower and higher values by the high-pressure method. Conceivably, these different findings may be related to the poor reproducibility of the optical method itself when the work of different operators is being compared. The Corps of Engineers data also point to one of the limitations of the high-pressure method—that it indicates total air and does not distinguish between air present as small beneficial voids, that present as entrapped

TABLE 1

AIR CONTENT—HIGH PRESSURE VS POINT COUNT

Batch	Percent Air Content Plastic	Percent Air Content, Hardened[a]			
		High Pressure	Point Count		
			Entrained	Accidental	Total
1	3.9	4.2	3.0	1.1	4.1
2	7.5	8.0	7.0	1.2	8.2
3	11.5	13.2	12.2	1.2	13.4

[a]Each value is an average of three tests (Corps of Engineers data).

air, or that present in aggregate particles. Even with this limitation, the high-pressure method is generally adequate for quickly answering the question of whether concrete in service has insufficient or excessive air content. Because the high-pressure method tends to indicate too high a value at low levels of air content, it is reasonably certain that a low air content by the high-pressure method indicates that the concrete is not adequately protected against freezing and thawing damage.

Effect of Environment

Many investigations of concrete are initiated because the concrete has evidenced deterioration as a result of some environmental influence, such as frost action, temperature or moisture changes, or chemical attack. In such cases, it may be desired to demonstrate that concrete from the structure is affected by one or more of these factors under controlled conditions. Tests for this purpose are difficult to interpret at best, but if attempted should be made on specimens that have not yet begun to deteriorate. The ideal specimen would be one containing sound aggregate, free from cracks, and usually taken from an interior location that has been protected from the suspected environmental influence. The significance of the tests is often greatly enhanced if concrete from another structure of proven good performance under the same conditions of exposure is included in the test program as a reference.

Deterioration of concrete from environmental effects usually results from internal pressures occurring in cycles, such as from freezing and thawing, or as a constant buildup, as in the case of alkali-silica reaction. The test procedure used, therefore, should consist of a series of cycles or a constant condition, depending on which is most conducive to producing the observed deterioration. The condition of specimens may be evaluated during the test by visual observation for evidence of deterioration, weight loss, length change, change in resonant frequency, or loss of strength.

Freezing and Thawing—Freezing and thawing tests, even when performed on carefully fabricated specimens, are recognized as being difficult to interpret in terms of probable performance under specific conditions of service. The difficulty of interpreting such tests performed on cut specimens is even greater, because the exposure of aggregate particles to absorption of water without the protection of cement paste can be expected to increase the severity of the test to an unknown degree, and thus to remove even further the conditions of the test from the reality of performance in a structure. The consideration that should be given to this factor in interpreting results is probably dependent on the pore characteristics of the aggregate. An aggregate of very low absorption, for example, would not be expected to be affected, and therefore specimens containing such an aggregate logically could be evaluated by the same criteria as molded specimens. Where aggregates with an appreciable absorption are involved, it may be impossible to determine the extent to which exposure of the aggregate particles may have increased damage to the specimen above the damage that would have occurred had they been surrounded by cement paste.

Two methods for rapid freezing and thawing of concrete specimens are presently available as ASTM Standards. As indicated in the scope of these methods, they are not intended to answer the question of how a particular concrete will perform in a structure, but rather to make possible a comparison of the freezing-thawing resistance of different concretes. For the purposes of an investigation, these procedures can be used to determine if concrete in the subject structure is more or less durable than concrete in another structure, or does or does not meet some arbitrary standard used to judge concrete for similar structures in the area.

In any case, the quantitative measure of the test can be in terms of changes in resonant frequency or length changes. Weight loss is sometimes used but can be very unreliable because even obviously deteriorated specimens may show a gain in weight due to increased absorption. Resonant frequency is generally not as sensitive an indicator of deterioration as length change. Total length change after a specified number of cycles, however, may be unreliable because it is strongly influenced by temperature and moisture conditions. Dilation of specimens during a single cooling cycle has been proposed by Powers (13) as a theoretically more appropriate measure of frost action,

and has been used as a basis for accepting aggregates for concrete (14). For this work, a dilation of more than 50 millionths in. per in. above the length at the freezing point was considered as being indicative of unsatisfactory durability under the conditions of the test.

Wetting-Drying, Heating-Cooling—Some concretes may be subject to deterioration as a result of excessive volume change caused by temperature and moisture fluctuations. Excessive volume changes of this type are usually related to thermal or compositional properties of the aggregates. Although the effects of moisture and temperature changes might be studied separately, the maximum volume change, and therefore the maximum effect, will usually be produced by a cycle such as wetting at an elevated temperature, drying, and then cooling. The extremes of temperature used should bear some relationship to those temperatures expected in structures.

Both resonant frequency and length-change measurements can be used to measure the effect of the treatment. Total length change is probably more useful for this type of test than for freezing and thawing tests because the specimens can be evaluated in a dry condition without disturbing the testing routine.

Chemical Action—Deterioration from chemical or physicochemical action may occur from reactions between constituents of the concrete, such as between cement and aggregates, or between a constituent of the concrete and an outside agent, such as sulfate solution. In either case, the course of the reaction can proceed under constant conditions and does not depend on a cycling treatment to cause damage. The test conditions should be selected so as to best promote the suspected type of reaction, such as the use of 100 F moist storage for the alkali-silica reaction. Length change is usually the best type of measurement for indicating the effect of the treatment on the specimens. Like freezing and thawing tests, a positive reaction of specimens to a particular test may only indicate that the concrete is susceptible to damage from a specific cause. The damage may not actually take place under the environmental conditions in the structure. One actual case history (15) of a structure that had remained sound for over 30 years showed that concrete specimens taken from the structure gave quick evidence of alkali-silica reactivity in 100 F moist storage as indicated by profuse formation of silica gel.

Absorption

The absorption of concrete is considered to be related to its overall quality and for this reason is frequently included as one of the properties to be determined on concrete whose performance is being studied. Of primary interest is the absorption that takes place rather rapidly when concrete is immersed in water at normal pressure and temperatures. Except where highly absorptive aggregates are involved, such absorption is primarily a filling of capillary pores of the paste and is known to vary inversely as the cement content and directly as the water-cement ratio and mixing water content of the concrete. Absorption also decreases as curing proceeds. Although each of these factors has an influence on the quality of concrete, it has not been found feasible to use absorption, per se, as an index of durability. The property of absorption is of value primarily for comparing different concretes, particularly with regard to their original water-cement ratio.

The absorption value, as well as its interpretation, will depend on the conditions of the test. A common procedure is to dry the concrete to constant weight at 105 C and immerse it for 24 to 72 hours at room temperature. Absorption of structural concrete on this basis is not particularly useful unless other background information is available on concretes being compared. For example, before it can be inferred that the higher absorption of two concretes indicates a higher mixing water content, it should be known that the curing histories and cement contents are the same. On the other hand, if one has evidence that the cement and original water contents of two concretes were essentially the same, then large differences in absorption should be indicative of different curing treatment.

Axon (16) has proposed a procedure for determining the original water-cement ratio of hardened concrete that utilizes an absorption value based on vacuum saturation. Under this condition of test, absorption is considered to represent total porosity, including

air voids and aggregate pore space, and is used in conjunction with linear traverse measurements of aggregate, paste, and air content to determine the relative volumes of the various components of the concrete.

Permeability

Permeability refers to the movement of water through concrete under pressure. This property is primarily of interest in connection with hydraulic structures or other structures, such as basement walls or storage vats, that might be subjected to percolation of liquids under head. Because permeability occurs primarily in the paste portion, it is a function of the paste pore structure and is influenced greatly by the water-cement ratio and by curing.

The usual method of testing concrete for permeability utilizes equipment such as that described by Cook (17), which provides a means of forcing water through a specimen under a known head. Results are usually expressed as permeability coefficients in terms of cubic feet per second across 1 sq ft of area under 1 ft of head through 1 ft of thickness. Data developed by the U.S. Bureau of Reclamation (18) show that permeability increases rapidly for water-cement ratios higher than 0.55 by weight. Permeability may also vary according to the direction in which it is measured. Thus, permeability measured in the vertical direction, as originally cast, would be expected to differ from that measured in the horizontal direction through the same specimen of concrete.

Thermal Expansion

The properties of thermal conductivity, specific heat, thermal diffusivity, and coefficient of thermal expansion may each have significance to specific engineering uses of concrete. The property considered here as having greatest significance to outdoor structures is the coefficient of thermal expansion.

The thermal expansion of most structural concrete lies between 3.5 and 6.5×10^{-6} in. per in. per deg F, and for all practical purposes can be considered a weighted average of the thermal coefficients of the paste and aggregate portions. However, the coefficient of the paste portion, and therefore of the concrete also, varies according to its moisture content, being a minimum when either oven-dry or saturated.

Because of the difficulties of closely controlling temperature and moisture conditions when concrete is supposedly in an oven-dry state, tests for thermal expansion are usually made when the concrete is in a saturated condition even though this does not yield as high a coefficient as would be obtained at intermediate conditions of moisture. An attempt to develop a standardized procedure for determining coefficient of thermal expansion on a saturated basis is being made by an ASTM subcommittee. Tests to date demonstrate that the coefficient varies according to the particular temperature range over which it is determined. For example, a typical set of data obtained in the Bureau of Public Roads laboratory gave these results on the same specimen:

Temperature Range, F	Thermal Coefficient, $\times 10^{-6}$
40 to 70	3.0
70 to 100	3.3
100 to 130	3.6

Thus, a suitable procedure must define the temperature range and, as a minimum, should provide for temperature control within 0.2 F. A further requirement is that specimens must be given an opportunity to reach volume equilibrium at each temperature point.

Shrinkage

The shrinkage of concrete from a structure may be of interest for explaining observed cracking, warping, or movement of structural members. The term shrinkage

usually refers to the change in volume of concrete that takes place as it loses moisture from its initial wet condition to some specific condition of dryness. The investigator of concrete in service cannot make this determination because the concrete has already experienced some drying. Thus, he can only determine the shrinkage taking place from some subsequent condition of wetness, such as that obtained by immersion in water. This is normally less than the original shrinkage. Nevertheless, shrinkage determined in this manner can be used to compare different concretes or to determine whether a concrete shows excessive shrinkage characteristics, such as that caused by the use of aggregates having a low modulus of elasticity or exhibiting high dimensional changes within themselves.

ASTM Method C 341, Length Change of Drilled or Sawed Specimens of Cement Mortar or Concrete, was revised to cover only the determination of volume change of concrete specimens taken from a structure as opposed to formed concrete products. Length changes are measured between gage points set into the specimen. Because the easiest condition of wetness to standardize is immersion in water, this treatment is prescribed for 7 days. The drying condition is at 50 percent relative humidity and 23 C. Under these conditions, concrete having normal shrinkage characteristics would not be expected to show greater than about 0.05 percent contraction even if allowed to reach equilibrium.

Unit Weight

The unit weight of hardened concrete varies according to the density of aggregates, mix proportions, and air content. Within a structure, it is unlikely that any of these factors other than air content will vary sufficiently to cause significant variations in density. Thus, the unit weight of concrete in a structure usually varies directly as its air content and is therefore a good indicator of the uniformity of this important property. Density can be used to estimate the actual air content of hardened concrete only with a great deal of difficulty, because the density of the aggregates, proportions, and water contents must be determined.

TESTS OF MOLDED SPECIMENS

In addition to visually examining and testing the concrete in a structure, it may be desirable to fabricate and test molded specimens in connection with an investigation of performance. The need for such tests may arise either because it has not been possible to determine the cause of the observed performance, or it is desired to demonstrate that the condition of the concrete resulted from a suspected cause by reproducing the same condition under controlled conditions.

Depending on the specific situation, it may be desirable to use samples of the original constituents if they are available, to use samples from the same sources from which the original constituents were obtained, or at least to use materials having similar properties as the original constituents. The comparisons that are necessary to develop the desired information usually fall into one or a combination of the following categories:

1. Comparison of the same or similar constituents with other constituents of known performance;

2. Comparison of various percentages of the same or similar constituents; and

3. Influence of exposure by subjecting similar specimens to a range of environmental conditions.

A prime example of this type of approach to understanding the performance of concrete in service was the manner in which the alkali-silica phenomenon was discovered and proved. Having observed that the particular type of deterioration in question could not be explained by any known factor, and that it occurred only with certain aggregates, but not necessarily always with those aggregates, specimens were fabricated using the suspected aggregates with different cements. It was finally observed that an expansion occurred only when cements of high alkali content were used, thus leading to the cause of the deterioration.

As in the case of the alkali-aggregate reaction, tests of this type are of the nature of research and, in fact, do often lead to extensive research investigations when the specific performance cannot be fully understood or explained by any known theory.

REFERENCES

1. Kesler, C. E., and Change, T. S. A Review of Sonic Methods for the Determination of Mechanical Properties of Solid Materials. ASTM Bull., Oct. 1957, p. 40.
2. Effects of Concrete Characteristics on the Pulse Velocity—A Symposium. HRB Bull. 206, 1959.
3. Whitehurst, E. A. Soniscope Tests Concrete Structures. PCA Bull. 36, Feb. 1951.
4. Muenow, Richard. A Sonic Method to Determine Pavement Thickness. Jour. PCA Research and Development Laboratories, Vol. 5, No. 3, Sept. 1963, p. 8.
5. Locating Reinforcing Steel in Concrete. Jour. PCA Research and Development Laboratories, May 1965, p. 76.
6. Preiss, K. Measuring Concrete Density by Gamma Ray Transmission. ASTM Materials and Research Standards, June 1965, p. 285.
7. Investigation of Compressive Strength of Molded Cylinders and Drilled Cores of Concrete. U.S. Army Engineer Waterways Experiment Station, Tech. Rept. 6-5222, Aug. 1959.
8. Kesler, C. E. Statistical Relation Between Cylinder, Modified Cube, and Beam Strength of Plain Concrete. Proc. ASTM, Vol. 54, 1954, pp. 1178-1187.
9. Walker, S., and Bloem, D. L. Studies of Flexural Strength of Concrete—Part 3: Effects of Variations in Testing Procedures. Proc. ASTM, Vol. 57, 1957, pp. 1122-1139.
10. Bloem, D. L. Effects of Sawing Specimens From Hardened Concrete on Measured Flexural Strength. Distributed with NSGA Technical Information Letter 145.
11. Lindsay, J. D. Illinois Develops High Pressure Air Meter for Determining Air-Content of Hardened Concrete. HRB Proc., Vol. 35, 1956, pp. 424-435.
12. High Pressure Test for Determining Air Content of Hardened Concrete. U.S. Army Engineer Waterways Experiment Station, Misc. Paper 6-286, Oct. 1958.
13. Powers, T. C. Basic Considerations Pertaining to Freezing-and-Thawing Tests. Proc. ASTM, Vol. 55, 1955, pp. 1132-1155.
14. Tremper, B., and Spellman, D. L. Tests for Freeze-Thaw Durability of Concrete Aggregates. HRB Bull. 305, 1961, pp. 28-50.
15. Memorandum from H. Allen to A. C. Swain, Bureau of Public Roads, Feb. 20, 1957.
16. Axon, E. O. A Method of Estimating the Original Mix Composition of Hardened Concrete Using Physical Tests. Proc. ASTM, Vol. 62, 1962, pp. 1068-1080.
17. Cook, H. K. Permeability Tests of Lean Mass Concrete. Proc. ASTM, Vol. 51, 1951, pp. 1156-1165.
18. Concrete Manual. U.S. Bureau of Reclamation, 7th Edition, 1963, p. 38.

Analytical Techniques

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•AN IMPORTANT GOAL of any concrete investigation is to correlate the performance of concrete with the chemical and physical properties of its components. Information in construction records, data made available by field surveys, or other types of exploratory study may reveal what was proposed (and supposed) to have been done and how the concrete should have performed. An evaluation of the resulting information may point out trends or relationships that can be interpreted subjectively. But direct proof of what has transpired lies in the concrete itself, and analytical investigations must be made to determine the facts that will explain the behavior of the concrete in its environment.

Analytical techniques for isolating many of the parameters of concrete are available. These parameters may often be correlated with certain concreting operations—from proportioning through curing—or with chemical and physical changes that affect concrete stability. The emphasis in this paper is not primarily on an understanding of concrete behavior, but rather on techniques that may provide information from which an understanding may arise. These techniques will be demonstrated by following a core through those various processing and investigative steps that might be employed.

Before processing the core, features on formed, finished, broken, or cut surfaces should be noted—markings from finishing tools or forms, and cracks and secondary deposits. A rough measure of porosity or hardness could be obtained by wetting and scratching the aggregate and paste; an opinion of the quality of the concrete could be formed by noting the manner of any breakage. Meaningful analyses are made possible by such observations on the core itself, especially when combined with general microscopical examinations. For example, Figure 1 shows the underside of a core from a bridge deck. Patterned imprints occur in aggregate sockets. These imprints, imposed by ice crystals that formed when the concrete was plastic or semirigid, can be traced for 2 in. along the core side. They are direct evidence that the concrete was subjected to freezing before setting of the matrix.

After such preliminary observations, the core can be sectioned and each section designated for a particular category of testing. A practical problem, and often one of economy, involves the earmarking of those tests that will provide data significant for the investigation. Direction can best be obtained from microscopical studies. These often reveal relationships that should be delineated by certain analytical methods; superfluous data, time-consuming to obtain, can thus be avoided. Figure 2 shows three categories of general testing that can be applied to the core: microscopical, chemical, and selective-area chemical and microscopical. Each of these categories is discussed separately.

MICROSCOPICAL ANALYSIS

A longitudinal slice prepared by procedures given in ASTM Standard C 457-66-T may, in addition to providing surfaces for measurement of characteristics of air-void systems, display features of or related to grading, proportioning, and distribution of aggregate; size and location of embedded metals; altered components, including their degree of alteration and location; the morphology and location of compounds that resulted from either the alteration of components or deposition from solutions originally external to the concrete; and volume instability evidenced by the pattern and location of cracks within the concrete and their location relative to components. Figure 3 shows some of these features, many of which are individually described in the following and shown in Figures 4 through 12. Figures 4 through 8 show specific examples of

Figure 1. Formerly present ice crystals were responsible for the patterned imprints in aggregate sockets (magnification 1.8 X).

features related to concreting procedures; Figures 9 through 12 show features associated with incompatibility of components.

Figure 4 shows a cross section of a core taken from a pavement where surface raveling had occurred. An area in the middle of the section was impregnated with wax (using the procedures of ASTM C 457-66 T) and appears darker than the concrete above and below. Air contents determined separately for the upper, middle (Fig. 5), and lower areas were 6, 15, and 6 percent respectively. Surface raveling occurred when high air contents, such as those in the middle of the section, were at the wearing surface.

Figure 6 shows a cross section of a core taken from a pavement that contained a nonuniform distribution of entrained air. The localized concentration of entrained air shown occurred in a small stringer of essentially neat paste that was flanked by non-air-entrained mortar.
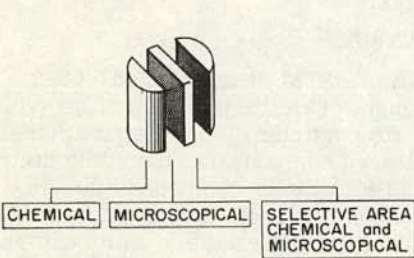

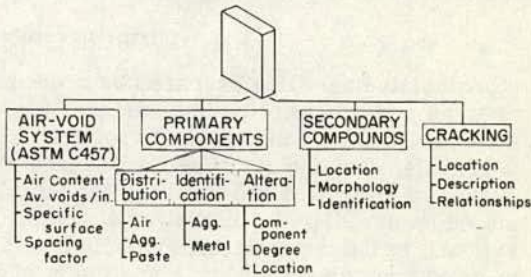Figure 2. Major analytical techniques used to analyze concrete.

Figure 3. Data or features obtained by microscopical analyses of lapped surfaces.
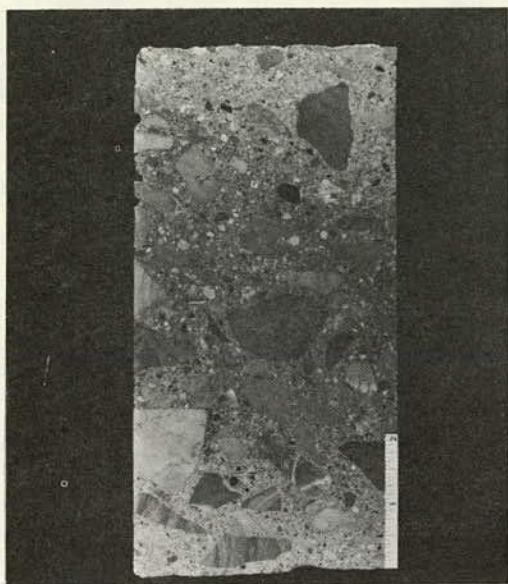
Figure 4. Cross section of a core from a pavement where surface raveling occurred. The dark area in the central portion was treated with wax to facilitate processing and is shown enlarged in Figure 5. Scale is in inches.

Figure 5. Magnified portion of the central area shown in Figure 4. The air content was determined to be 15 percent (magnification 3X).

Figure 7, a cross section of a core taken from a pavement, shows a $\frac{1}{2}$-in. zone that is devoid of coarse aggregate and is located at the wearing surface. The proportions of air, aggregate, and paste in this zone and in the remaining concrete can be determined by use of linear traverse or point-count methods of measurement.

Figure 8 shows the middle portion of a cross section of a core taken from a pavement made using a two-course method of construction: a base course was placed first, mesh was then "walked" into the base course, and the top course was placed. An abnormal concentration of entrapped voids occurred in areas flanking the mesh.

Figure 9 shows a cross section of a core taken from an airport runway that had cracked both before and after resurfacing. The 1-in. thick overlay has bonded excellently. In the overlay is a vertical crack that extends into the base concrete. The coarse aggregate in the base concrete is fractured, with the fractures frequently extending into the mortar. Freezing and thawing of moisture-saturated aggregate was interpreted as being the mechanism responsible for the fracturing in the base concrete; cracking in the overlay is probably a reflection of the instability of the base concrete.

Figure 10 shows an upper cross section of a core from a bridge deck. Associated with the reinforcing bar, which is located $\frac{5}{8}$ in. below the wearing surface, are two prominent fractures that angle obliquely and intersect the surface. Minor fractures that extend radially into the mortar are also visible. The steel is corroded along its
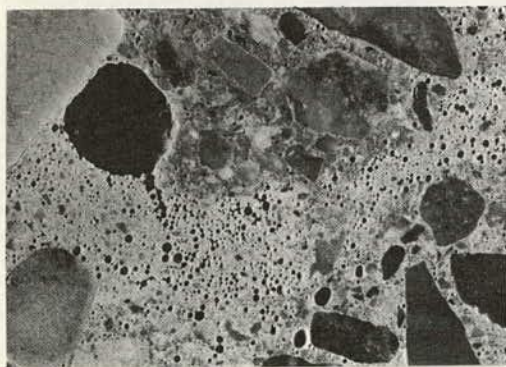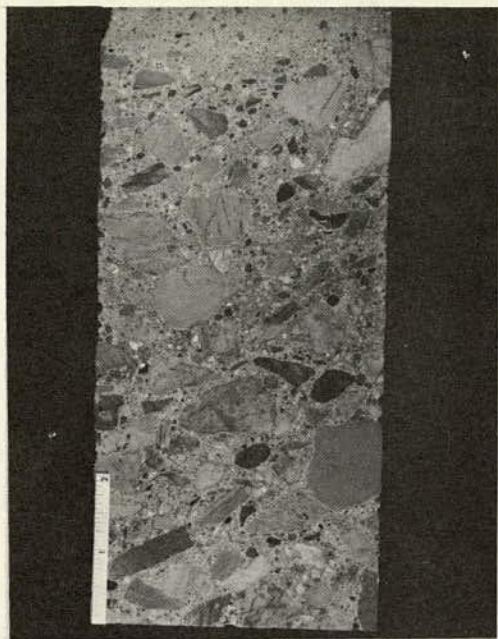

Figure 6. Cross section of a portion of a core from a pavement where a nonuniform distribution of entrained air occurred (magnification 3X).

Figure 7. Cross section on a core taken from a pavement. A ½-in. thick zone at the wearing surface is devoid of coarse aggregate. Scale is in inches.

upper periphery and corrosion products have migrated into the fractures. The major cracks define an incipient spall. Relationships between the thickness of cover over the steel, the cracking patterns, the disposition of the corrosion products, and other features were useful for interpreting the mechanism responsible for the cracking.

Figure 11 shows a portion of a cross section of a laboratory prism containing a coarse aggregate known to shrink excessively. The peripheral cracking associated with the aggregate particles was used to determine the relative volume changes of the components.

Figure 12 shows part of a cross section of a laboratory prism that contained a paste designed to expand. Peripheral separations occurred at coarse aggregate-paste interfaces. Unfortunately, the cracking patterns in this figure and in the preceding figure were similar. Thus, when encountering field concretes, the unstable component must be determined through supplementary tests. For example, one might make measurements of dimensional
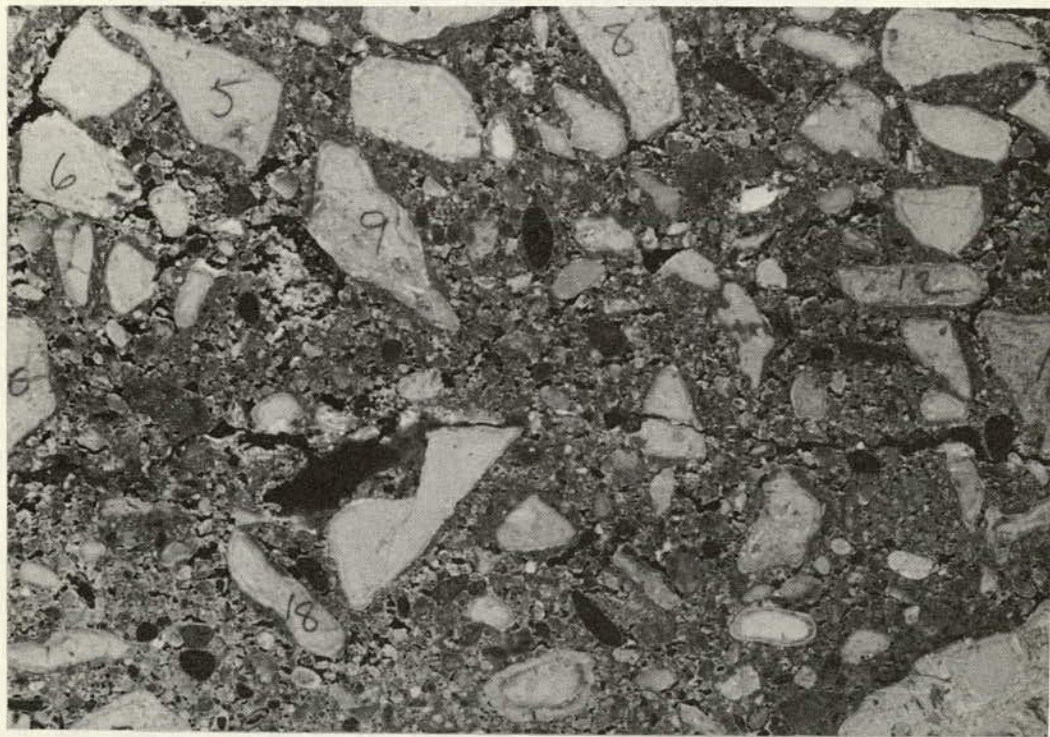
Figure 8. Enlargement of the central portion of a core taken from a pavement placed using two-course construction. An abnormal concentration of entrapped voids is present (magnification 1.5X).
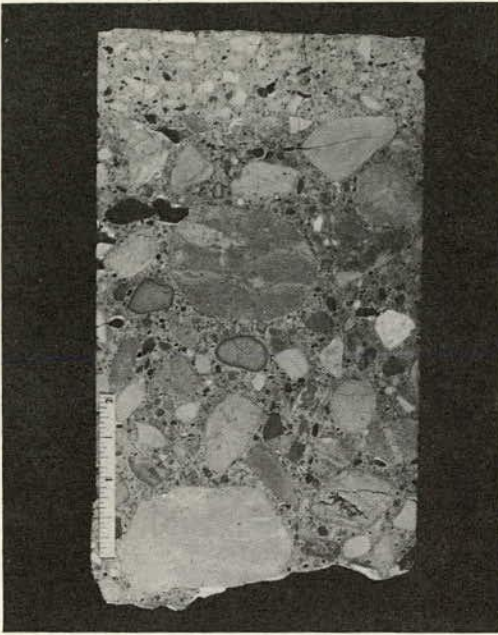
Figure 9. Cross section of a core taken from a resurfaced airport runway. Cracking in the 1-in. resurfacing is a reflection of continuing distress in the base concrete. Scale is in inches.

Figure 10. Cross section of a core taken from a bridge deck. An incipient spall, radial cracking, and rusting are associated with the reinforcing bar.

Figure 11. Enlargement of an area from a cross section of a prism made with a coarse aggregate that shrank excessively. Peripheral cracking was associated with the coarse aggregate particles (magnification 57.4X).

Figure 12. Enlargement of an area from a cross section of a prism made with a paste designed to expand. Peripheral cracking was associated with coarse aggregate particles. Scale is in $1/16$-in. increments.

changes that occur when the coarse aggregate is removed from the concrete and immersed in certain solutions. Identification of secondary components associated specifically with volume changes of aggregate, or of paste, might also be attempted.

CHEMICAL ANALYSIS

Chemical analysis covers a broad field and is here categorized as including "wet chemical" and instrumental methods used to analyze a relatively large portion of concrete. These analyses can be useful for identifying organic admixtures and additives, for identifying contaminants initially present or subsequently absorbed, and for establishing approximate cement contents.

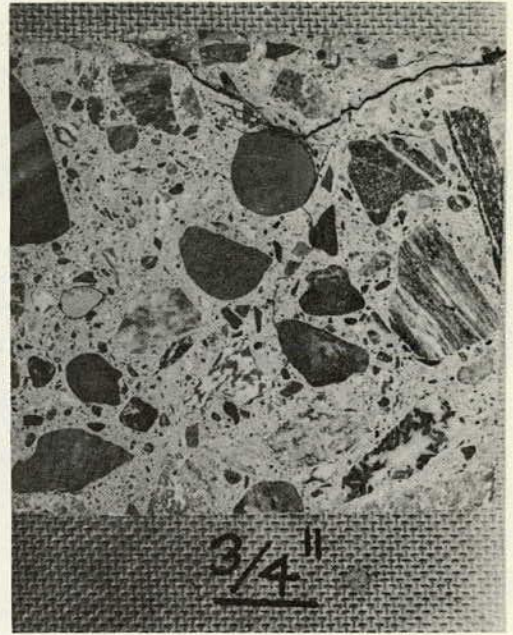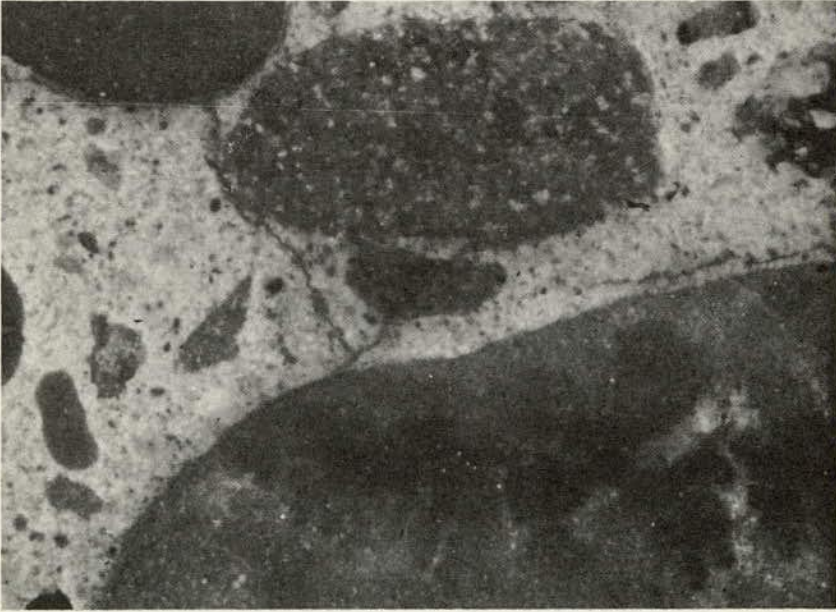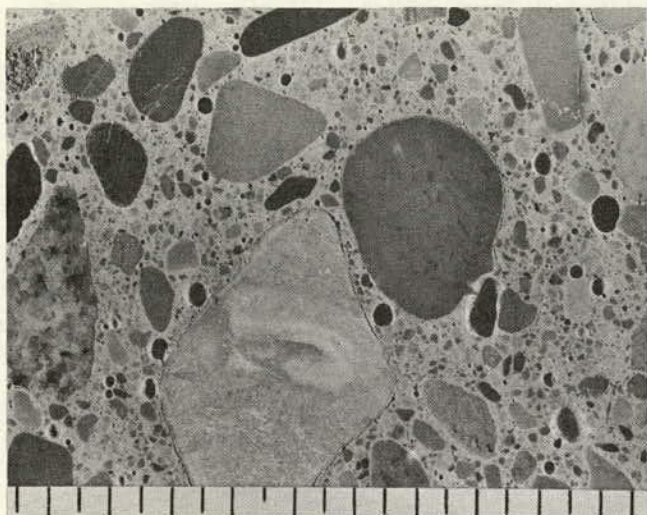Infrared spectroscopy has been used to detect air-entraining agents, retarders, plasticizers, water reducers, and other types of admixtures used in concrete. Classical wet chemical or colorimetric methods may be used to detect sugars and lignosulfonate; the lignosulfonate identification should be confirmed by ultraviolet or infrared spectroscopical methods. A failure to detect specific organic components should not be taken as a definite indication of the absence of organic matter. A positive identification, on the other hand, may often provide semiquantitative data.

Wet chemical methods are employed in ASTM Standard C 85-66 to provide quantitative data for lime and silica that can be recalculated in terms of cement content. The results, however, are affected by aggregates containing lime and silica that are soluble in the test.

A chemical test may be designed to allow detection of any element of interest. A frequent problem is to determine what is of interest! Figure 13 shows some of the useful information that can be obtained through chemical analyses.

SELECTIVE-AREA ANALYSES

A selective-area analysis may involve quantities of material smaller than a mosquito's eye or larger than half a core. A pinpoint of material may represent several strained ettringite crystals that grew within

Figure 13. Types of data obtainable by chemical analyses.
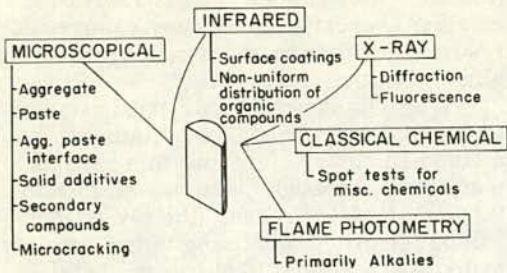
Figure 14. Data that can be made available by analyzing selective areas using a variety of analytical techniques.

the complex paste; 45 percent of a core half may consist of secondary gypsum. Selective-area analyses may involve any techniques that an analytical mind can devise. Those techniques shown in Figure 14 are significant because they can be used to detect, pinpoint, or identify particular components, components that have been altered, alteration products, solid materials as small in size as cement, surface coatings, nonuniform distributions of secondary compounds, soluble additives or admixtures, and microcracks. Sampling and specimen preparation techniques will vary with the investigator and may be devised as required—often through his innate animal cunning.

Microscopical methods can be used to evaluate aggregate (one of the easiest components to identify); the size and number of residual cement particles (which can be used to assess cement fineness and to estimate cement content and water-cement ratio); the compound composition of residual cement particles; particles having the fineness of cement such as slag, slaked lime, fly ash, pozzolans, limestone fines, and quartz

Figure 15. Entrapped air void containing needles of ettringite and platelets of calcium hydroxide (magnification 7.5×).
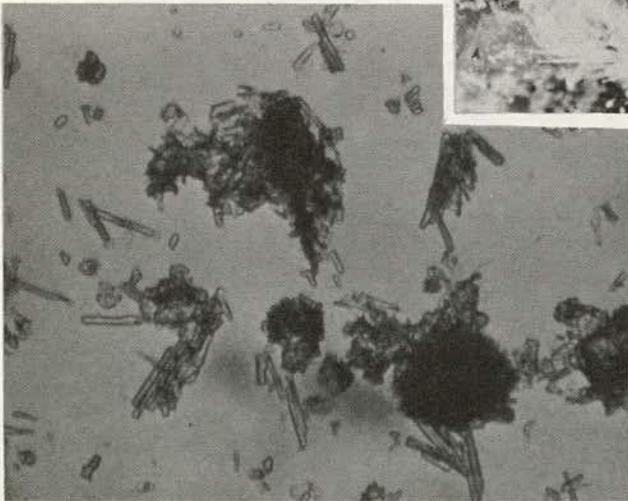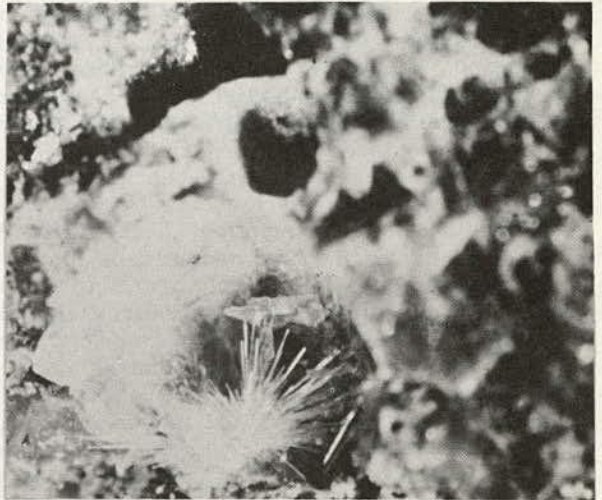
Figure 16. Powder mount of altered paste containing ettringite (magnification 375×).

Figure 17. Polished section that shows residual and relic cement particles and microcracking.

flour; the morphology and distribution of calcium hydroxide; secondary compounds (their identification and location); and other almost fugitive features.

Figure 15 shows an entrapped air void that contains fine needles radiating from a common center, plus two thin hexagonal platelets. Although there is not much of these materials present, the two phases can be identified as ettringite and calcium hydroxide. Figure 16 shows material, selectively picked from a soft area of paste, identified as ettringite and altered paste. The location of the ettringite, as shown in the preceding figure, would be normal for many concretes exposed in a moist environment. In Figure 16, however, its occurrence typifies concrete damaged through sulfate attack. Figure 17 shows a photomicrograph of an area of paste (highly polished) from a concrete curb that expanded excessively. Recognizable are three minus-200-mesh aggregate particles and several residual and relic cement particles surrounded by hydration products. Although the bulk of the cement particle located in the center of the figure has hydrated, the original size of the particle can be estimated closely. The dark circular areas within the particle were crystals of free lime, now hydrated to calcium hydroxide. Radiating from the particle are several microfractures that resulted from the in situ hydration of the free lime. This single observation was insignificant by itself, but when coupled with many other similar observations demonstrating the free lime-cracking relationship, plus field survey notes that showed the curb had expanded excessively, the cause for the abnormal behavior was indicated.

Infrared spectroscopical methods can be used to analyze selected portions of concrete, such as areas containing unusual concentrations of air voids, abnormally hydrated paste, or coated surfaces. Such areas can then be compared with areas that do not display these characteristics. X-ray diffraction can be used to identify extremely small quantities of secondary compounds, and X-ray fluorescence can be used to determine the presence of elements that may escape detection by other analytical methods. Classical chemical spot tests can be fruitful in detecting small localized quantities of elements or oxides that would be masked if a large specimen were analyzed.

Figure 18 shows how selective analyses may be used to establish relationships between types of deterioration (e. g., scaling, spalling, and cracking), such as observed on bridge decks, and specific elemental contents at different concentrations. In this instance, chloride levels in cores taken from deteriorated decks were compared with those in cores from sound decks.

SUMMARY

Some of the analytical techniques that may be useful for examining concrete have been presented. A wealth of data may be obtained through use of these techniques: data related to the cement; to the paste; to the aggregate; to the air-void system; to embedded metals; to proportioning, mixing, placing, consolidation, finishing and curing; to the alteration of components; to cracking; and to the effects of environment on concrete.
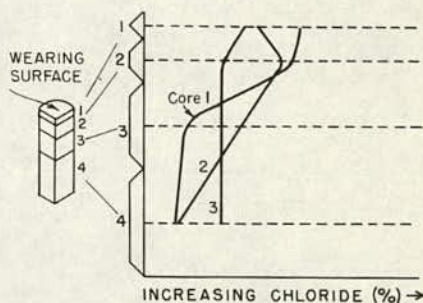
Figure 18. Analysis for chlorides in selected areas in cores may be useful for correlating particular types of deterioration and chloride ion concentrations.

Such data, when evaluated in the light of the sampling methods and when combined with the information that records, field surveys, and physical tests reveal, can provide connecting links between performance and responsible agents.

REFERENCES

1. Brown, L. S. Petrography of Cement and Concrete. Jour. PCA Research and Development Laboratories, Vol. 1, No. 3, Sept. 1959, pp. 23-34; also Research Department Bull. 111, PCA.

2. Erlin, Bernard. Methods Used in Petrographic Studies of Concrete. Analytical Techniques for Hydraulic Cement and Concrete, ASTM STP 395, 1966, pp. 3-17; also Research Department Bull. 193, PCA.

3. Hime, W. G., Mivelaz, W. F., and Connolly, J. D. Use of Infrared Spectrophotometry for the Detection and Identification of Organic Additions in Cement and Admixtures in Hardened Concrete. Analytical Techniques for Hydraulic Cement and Concrete, ASTM STP 395, 1966, pp. 18-29; also Research Department Bull. 194, PCA.

4. Mather, Katharine. Petrographic Examination (Hardened Concrete). Significance of Tests and Properties of Concrete and Concrete-Making Materials, ASTM STP 169-A, 1966, pp. 125-143.

5. Mielenz, R. C. Petrography Applied to Portland Cement. Reviews in Engineering Geology, Geol. Soc. of America, Vol. 1, pp. 1962, pp. 1-38.

6. Mielenz, R. C. Diagnosing Concrete Failures. Lecture No. 2, Stanton Walker Lecture Series on the Material Sciences, Univ. of Maryland, Nov. 18, 1964.

Part II
Case Studies

Measurement of Volume Changes of Concrete in Service

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●THE PRESENCE of cracking on the surface of concrete is generally accepted as an indication of some kind of a volume change in the body of the concrete. A correct interpretation of the nature of the volume change is not a simple matter, and more extensive observations are usually necessary before it can be said with certainty whether the cracking is the result of expansion or contraction or a combination of both. Where a reaction between alkalies in the cement and some component of the aggregates is suspected, there may be a gross expansion within the body of the concrete, but the surface cracks may be due in part to a shrinkage or to a lack of expansion of the surface layers. The cracks will open wherever the stresses exceed the tensile strength of the surface layer, and if there is no restraint in any direction, a typical pattern-cracking appears. However, the same kind of pattern-cracking may be due entirely to a shrinkage of the surface with no expansion or comparable shrinkage of the interior. Shrinkage of concrete is a common phenomenon and cements are now being marketed that are shrinkage-compensated. At the other extreme, there are many examples of so much expansion in concrete pavements that blowups occur, with total failure of areas of the pavement.

In order to eliminate the need for speculation as to the nature of the movement in concrete bridge decks and other bridge elements, a program of precise length-change measurements was undertaken by the Portland Cement Association in 1954 in cooperation with several state highway departments and other organizations. The equipment, assembled in the PCA shops, consisted of a 100-in. Invar steel tape, with means for attaching to brass inserts in the concrete and for applying a tension of 30 lb. Ten accurate subdivisions of 0.10 in. are provided on each side of the 0 and 100-in. marks, and readings are made with a shop microscope that further subdivides each 0.10 in. into 100 parts. Two different tapes are used each time and the calculated measurements are checked within a tolerance of 0.002 in. Finally, the internal temperature of the concrete is obtained and the measurements are corrected to 74 F, the temperature at which the tape is calibrated in the laboratory. This equipment is shown in Figures 1 and 2.

In two southeastern states, 32 installations were made in 12 bridges, the usual
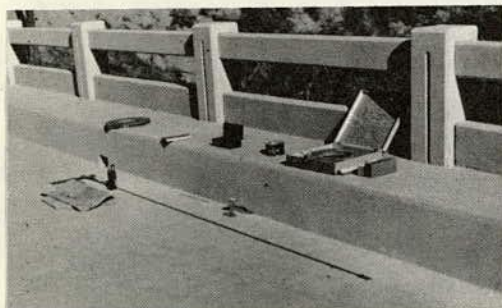
Figure 1. Precise measurement equipment showing typical installation in gutter line.
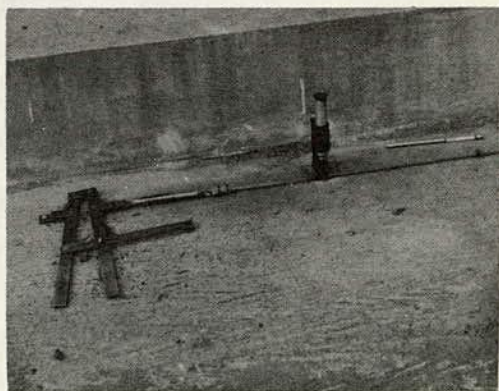
Figure 2. Tensioning device and shop microscope reading to thousandths of an inch.
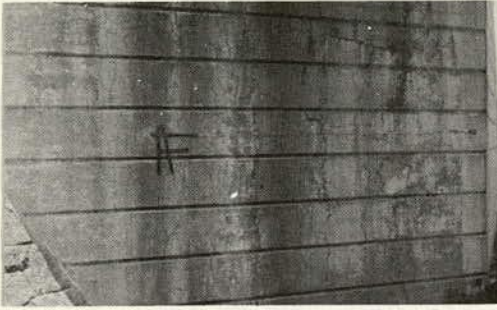
Figure 3. Precise measurement installation in severely cracked abutment.

Figure 4. Setting brass insert with lead wool. This concrete is expanding at 0.03 percent per year.
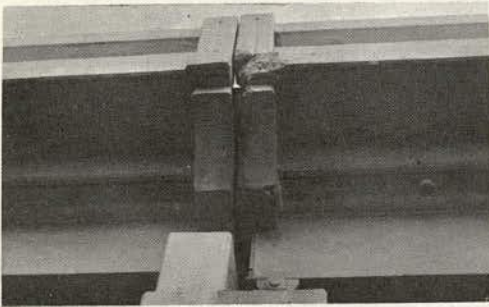
Figure 5. Expansion in bridge deck results in tight joint and broken end web.

location being in the gutter at 6 to 12 in. from the face of the curb. Other locations were on the top of the curb or safety walk, or on a wing wall or abutment. The concretes included a wide range of cements and aggregates, and the structures were new bridges in most cases, although a few were several years old. After 2 years of observation and measurements with no appreciable change in any of the concretes, another bridge, in which the concrete was definitely pattern-cracked, was added to the program (Figs. 3 and 4).

RESULTS OF THE STUDY

A few of the brass inserts have been destroyed or damaged beyond use, but many of the original points have remained undamaged for over 10 years. In the 32 original installations, the maximum contraction
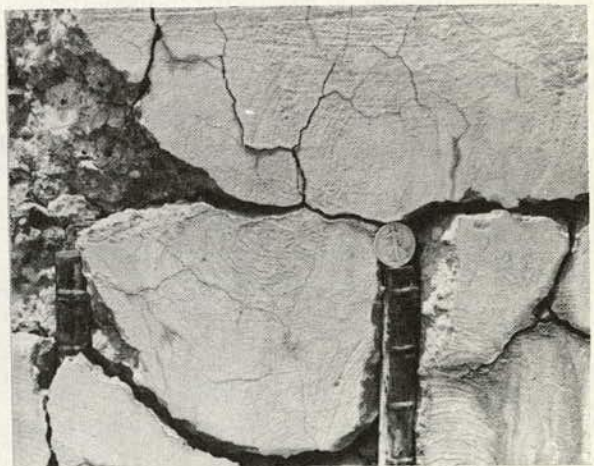
Figure 6. Hand rail post in 1956 (left) and 1964 (right) vertical expansion is 1 $\frac{3}{16}$ in. in spite of reinforcing.

has been 0.021 in. or 0.02 percent. The maximum expansion has been 0.017 in. or 0.017 percent. However, it is recognized that these installations were made in more or less heavily reinforced concrete and there was a possibility that any potential volume change might have been affected by the reinforcement. That this is not necessarily the case is shown by a bridge deck (Fig. 5) that has expanded sufficiently to shear reinforcement for end webs and to spall off the concrete cover for the bars. The steel girders are the reference media in this case.

Another example of expansion of a reinforced concrete is shown in Figure 6. This post was observed to be severely cracked in 1956, with one of the heavy vertical bars being partially exposed. By 1964, the concrete had expanded vertically $1\frac{3}{16}$ in. This is the clear distance between the top of the bar and the concrete that originally was in contact with the top end of the bar. In this case, the reinforcement had no significant effect in preventing expansion.

For the bridge that was added to the program because it was already cracked, two installations in the abutments have revealed expansion at an annual rate of about 0.03 percent for a total of 0.185 and 0.171 in. between the reference points. This example provides the assurance needed to show that the precise measurement program does indeed measure expansion or contraction in concrete if it is present.

CONCLUSION

The techniques and equipment developed by PCA for measuring volume changes in concrete in service are convenient, easy to use, and reliable. The concrete in 12 bridges, measured over periods of as much as 11 years, has shown no appreciable dimensional change in either direction. During the same observational period, concrete that was cracked, and known to be affected by alkali-aggregate reaction, has shown a steady rate of expansion.

Deterioration of Concrete Sidewalks and Curbs

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   Experiment Station

Twenty-four 4-in. diameter concrete cores, three 6 by 12-in. concrete
cylinders, and samples of fine and coarse aggregates were examined to
determine the cause of heavy surface scaling that occurred in some of
the concrete. Eighteen of the cores came from six locations in the af-
fected area. The locations from which they were taken had been selected
in pairs separated by expansion joints to represent concrete in relatively
better and relatively worse conditions. The other six cores represented
new concrete from another nearby area and from a second nearby area
that was in good condition after one winter. The three cylinders repre-
sented interior concrete from the fourth floor of a new building in the
area.

Air content and cement content determinations and petrographic ex-
aminations were made on the cores from the affected area. Air content
determinations were made on the cores and cylinders from the other
areas. These examinations yielded evidence that supported the hypoth-
esis that the deterioration of the affected concrete was caused by freez-
ing and thawing, which produced damage because the concrete was not
sufficiently air-entrained, and that future damage to similar concrete
that was still intact could be expected.

•SCALING AND DETERIORATION of concrete in sidewalks and curbs were noticed
after the winter of 1962-1963 (Figs. 1 and 2). The concrete had been placed in 1960
and 1961, and no such deterioration or scaling was expected. A representative of the
Concrete Division, U.S. Army Engineer Waterways Experiment Station (WES), met
with representatives of other agencies in April 1963 and an inspection of the concrete
in question was made. As a result of this inspection and of the discussions that fol-
lowed, samples of deteriorated and undeteriorated concrete and of the aggregates used
in some of the concrete were sent to WES for laboratory investigation.

The concrete was subjected to petrographic examination and air and cement content
determinations. The purpose of the laboratory work was to determine the reason or
reasons for the deterioration, and whether additional deterioration could be expected
of the unaffected concrete already in place. In addition, recommendations to prevent
similar damage in the future were to be made if possible.
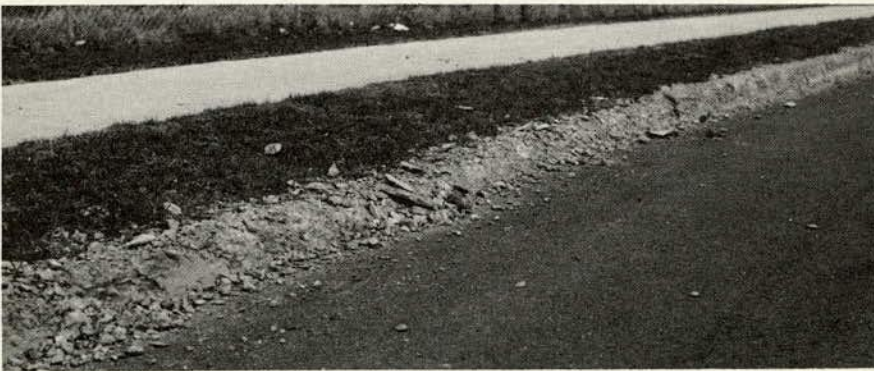
Figure 1. Close-up of section of a deteriorated concrete curb, affected area.

Identification and description of the samples (24 concrete cores, 3 concrete cylinders, and 50 lb each of coarse and fine aggregate) are given in Table 1 and are sumarized in the following tabulation:

Serial No.	Sample
CON-1 to -3 -4 to -6	4-in. diameter concrete cores from two locations in sidewalks in the affected area, Contract A
CON-13 to -15 -16 to -18 -19 to -21 -22 to -24	4-in. diameter concrete cores from four locations in curbs in affected area, Contract A
CON-7 to -9	4-in. diameter concrete cores from one location in sidewalk at another area, Contract B
CON-10 to -12	4-in. diameter concrete cores from exterior slab at a third area, Contract C
CON-25 (A, B, C)	Three 6 by 12 in. jobsite concrete cylinders from a fourth area, Contract D
G-1, S-1	50 lb each of coarse and fine aggregate representative of that used in the concrete of the cores of Contract A

MATERIALS AND MIXTURES

Information on materials and mixtures intended or required to have been used in the work represented by the samples and on placing dates and the nature of subsequent exposure was received from the resident engineer. Sidewalks constructed in the affected area (Contract A) were specified to be made using class B-A (air-entrained) concrete having a minimum allowable compressive strength of 2,500 psi at 28 days. By addendum to the specification, curbs were added to the work on which concrete of this class was to be used. It was required that trial batches of concrete be made by the contractor to establish a curve, based on at least three points, for the relation of water content to 28-day compressive strength, with each point representing the average values from at least four test specimens. The maximum allowable water content was then required to be selected as that quantity giving a compressive strength 15 percent greater than the minimum specified.

In June 1960, the contractor was advised of approval of mixtures covered by a report from a commercial testing laboratory to the contractor, based on tests of concrete made by the supplier of ready-mixed concrete. The only data contained in the referenced report were results of 28-day compressive strength tests, measured in psi, of trial mixtures 1, 2, and 3 as follows:

Figure 2. Deteriorated concrete sidewalk, affected area.

	Mixture 1	Mixture 2	Mixture 3
	2,334	3,714	5,129
	2,476	3,555	5,306
	2,299	3,484	5,483
	2,423	3,643	5,235
Average	2,383	3,599	5,288

The testing laboratory made a second recommendation for class B air-entrained concrete to be proportioned as follows:

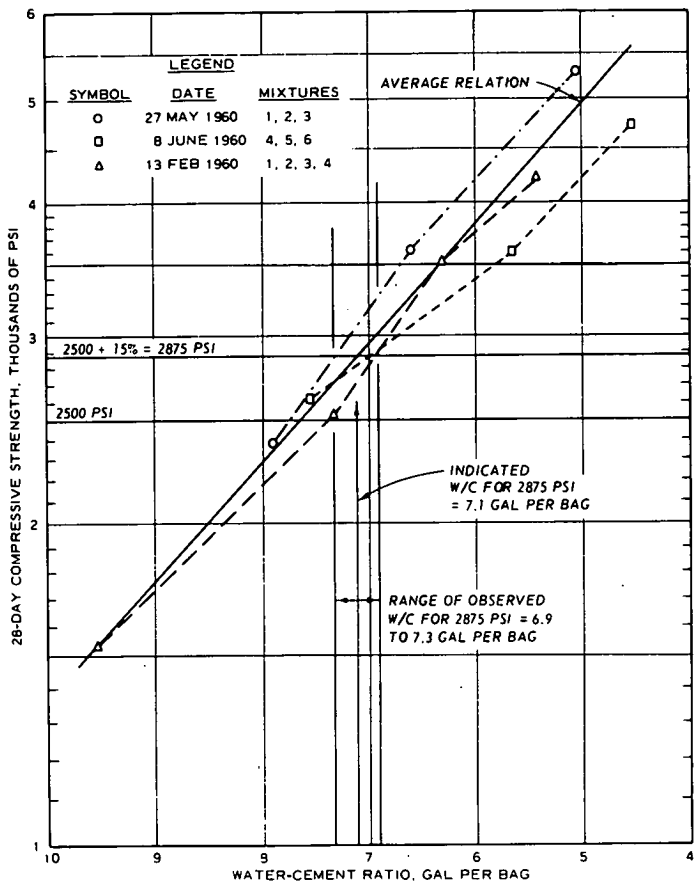| Measure	Cement	Sand	1 in. Gravel	Water
Volume proportions (1 bag)	1.00	2.30	3.86	6.70 gal
Volume proportions (1 cu yd)	5.08	11.67	19.59	34.00 gal
Absolute volume (1 cu yd)	2.43	7.61	11.45	4.53 cu ft
Dry weight (1 cu yd)	478	1,249	1,900	34.00 gal

Figure 3. Relationship of water-cement ratio to compressive strength based on data submitted for Contracts A, B, C, and D.

TABLE 1

IDENTIFICATION AND DESCRIPTION OF CONCRETE CORES AND CYLINDERS

Serial No.	Location	Diameter (in.)	Length (in.)	Concrete Condition	Date Placed	Class of Concrete	Contract
CON-1	Sidewalk, 20 ft	4	$4\frac{1}{2}$	Good	Sept. 28, 1961	B-A	A
-2	from cores 4,	4	$4\frac{3}{8}$	Good	Sept. 28, 1961	B-A	
-3	5, 6	4	$3\frac{5}{8}$	Good	Sept. 28, 1961	B-A	
-4	Sidewalk	4	5	$\frac{1}{4}$ in. spalled	Sept. 28, 1961	B-A	A
-5		4	$3\frac{3}{4}$	$\frac{3}{4}$ in. spalled	Sept. 28, 1961	B-A	
-6		4	$5\frac{1}{2}$	$\frac{3}{4}$ in. spalled	Sept. 28, 1961	B-A	
-13	Curbing, 15 ft from	4	$12\frac{1}{2}$	Good	July 12, 1961	B-A	A
-14	cores 16,	4	$9\frac{3}{4}$	Good	July 12, 1961	B-A	
-15	17, 18	4	12	Good	July 12, 1961	B-A	
-16	Curbing	4	12	$\frac{3}{4}$ in. spalled	July 12, 1961	B-A	A
-17		4	$11\frac{7}{8}$	$\frac{3}{4}$ in. spalled	July 12, 1961	B-A	
-18		4	12	$\frac{3}{4}$ in. spalled	July 12, 1961	B-A	
-19	Curbing	4	$8\frac{3}{4}$	$\frac{3}{4}$ to 1 in. spalled	Oct. 1, 1960	B-A	A
-20		4	$11\frac{1}{4}$	$\frac{3}{4}$ to 1 in. spalled	to Dec. 12,	B-A	
-21		4	$8\frac{1}{2}$	$\frac{3}{4}$ to 1 in. spalled	1960[a]	B-A	
-22	Curbing, 10 ft from	4	$10\frac{1}{2}$	Good	Oct. 1, 1960	B-A	A
-23	cores 19,	4	$5\frac{1}{4}$	Good	to Dec. 12,	B-A	
-24	20, 21	4	11	Good	1960[a]	B-A	
-10	Area 2	4	$8\frac{3}{4}$	Good	Aug. 3, 1962	A(A)	C
-11		4	$8\frac{1}{4}$	Good	Aug. 3, 1962	A(A)	
-12		4	$8\frac{1}{4}$	Good	Aug. 3, 1962	A(A)	
-7	Area 3	4	$7\frac{5}{8}$	Good	Aug. 10, 1962	A(A)	B
-8		4	$7\frac{5}{8}$	Good	Aug. 10, 1962	A(A)	
-9		4	$7\frac{5}{8}$	Good	Aug. 10, 1962	A(A)	
-25A	Area 4	6	12	Good	Oct. 16, 1962	B	D
-25B		6	12	Good	Oct. 16, 1962	B	
-25C		6	12	Good	Oct. 16, 1962	B	

[a]Concrete in certain sections was replaced May 1 and June 6, 1961.

This recommendation stated that the slump range was 2 to 3 in., the air content was 3 to 6 percent, and that $\frac{3}{4}$ oz of air-entraining agent was to be added per sack of cement.

It will be noted that this mixture requires 6.7 gal of water per bag of cement and 5.1 bags of cement per cubic yard of concrete. The water-cement ratio of 6.7 gal per bag is lower than that indicated to be required by the strength test results, which would appear to justify a water-cement ratio of at least 6.9 gal and possibly as high as 7.3 gal per bag. The first recommendation suggested a water-cement ratio of 7.2 gal per bag and a cement content of 5.35 bags per cu yd. Presumably the approval of the first recommendation was rescinded by the second recommendation.

All 28-day cylinder test data included in the laboratory mixture proportioning reports furnished WES are shown in Figure 3. Because the relation of water-cement ratio to concrete strength is a straight line on a semilogarithmic plot, the data have been so plotted rather than in arithmetic plots such as were included in the data submitted.

From the information reviewed in the preceding paragraphs, it is assumed that the maximum allowable water content of the concrete in the sidewalks and curbs of the affected area was 6.7 gal per bag with a corresponding cement content of 5.1 bags per cu yd. Test reports 60, 61, 65, and 67 cover 28-day tests of field-made cylinders of class B air-entrained concrete used in porches and curbs in the affected area in October 1960. The reported strengths for nine cylinders ranged from 2,550 to 4,420 psi and averaged 3,335 psi. The average curve in Figure 3 predicts a strength of 3,175 psi for a water-cement ratio of 6.7 gal per bag.

TESTS

Air Content

A slice about 1 in. thick was cut parallel to the axis from the central portion of each of the 24 concrete cores and 3 concrete cylinders. Such slices representing each set of cores or cylinders (slices from cores 1 through 7, 10, 13, 16, 20, and 24, and from cylinder 25C) were chosen to provide sufficient surface area to represent the mortar, allowing for the maximum size of the coarse aggregate used in the concrete. These surfaces were ground with abrasive and water to develop a smooth plane surface. The air content of the hardened concrete was determined micrometrically by the point-count method at a magnification of 40 diameters, in accordance with ASTM Designation C 457-60 T. The following criteria were used in classifying the types of air voids encountered:

1. Large voids of irregular shape attributable to incomplete consolidation were ignored on the premise that such air-filled space would not have been present in the concrete contained in a sample that had been properly prepared for test in the freshly mixed condition in accordance with ASTM Designation C 231-62.

2. Entrained air voids were differentiated as those voids having essentially a spherical shape and represented in the polished surfaces as circles having a diameter of 1.25 mm or less.

3. Entrapped air voids were considered to be those having an irregular (nonspherical) shape and represented by sections having a diameter greater than 1.25 mm.

Cement Content

Portions of the 18 cores taken to represent the concrete from six locations in the affected area (Contract A) that were not used in the examination for air content were used to determine cement contents by chemical analysis in accordance with ASTM Designation C 85-54. Representative samples of aggregate similar to those used in this concrete were analyzed chemically for soluble silica and soluble calcium oxide. The results of the aggregate analyses and the SiO_2 value obtained from National Bureau of Standards analyses of portland cements used in the construction, or the CaO value determined by means of ASTM C 85-54, were used to calculate the cement contents of the cores analyzed. The specific gravities of the cores were determined using a modification of Method CRD-C 23-58 (U. S. Corps of Engineers Handbook for Concrete and Cement). The cement contents of the cores in terms of bags of cement per cubic yard of concrete were calculated from the cement content and specific gravity data.

Petrographic Examination

The 18 cores from the affected area and the slices taken from them were examined visually and with a stereomicroscope for signs of deleterious chemical reactions.

TEST RESULTS

Air Content

Table 2 gives a compilation of the air content data on the concrete from the affected area (Contract A). It shows location and condition of the cores and amount of entrained,

TABLE 2

AIR CONTENT OF CORES FROM AFFECTED AREA, CONTRACT A

Concrete[a]		Core No.[b]	Air Content, percent[c]		
Location	Condition		Entrained	Entrapped	Total
Sidewalk	Good	CON-1	3.0	2.6	2.9
		-2	0.4	1.0	1.4
		-3	0.2	1.3	1.5
		Average	0.3	1.6	1.9
Sidewalk	Poor	-4	0.5	1.9	2.4
		-5	0.5	1.4	1.9
		-6	0.6	3.0	3.6
		Average	0.5	2.1	2.6
Curb (road No. 2)	Good	-13A	2.3	2.1	4.4
		-13B	1.3	2.3	3.6
		Average	1.8	2.2	4.0
Curb (road No. 2)	Poor	-16A	0.7	0.5	1.2
		-16B	0.8	1.5	2.3
		Average	0.8	1.0	1.8
Curb (road No. 1)	Good	-24A	5.4	0.7	6.1
		-24B	4.9	0.6	5.5
		Average	5.2	0.6	5.8
Curb (road No. 1)	Poor	-20A	0.4	0.4	0.8
		-20B	0.5	1.0	1.5
		Average	0.5	0.7	1.2

[a]Air-entrained concrete is represented by cores 13 and 24; other cores are of non-air-entrained concrete.
[b]"A" and "B" designate top and bottom portions, respectively, of cores 11 to 12 in. long.
[c]Averages are computed as percentage of total number of points observed in all specimens examined that were found to lie in voids of each type.

entrapped, and total air in typical cores. Table 3 gives similar data for concrete from the sidewalk at another area (Contract B), the exterior slab at a third area (Contract C), and a fourth area (Contract D).

Portions of the prepared surfaces of two of the cores used for air content determinations were photographed. Figures 4 and 5 show the appearance of non-air-entrained concrete (core CON-20) from which about 1 in. of the top surface had been spalled, and Figures 6 and 7 illustrate air-entrained concrete (core CON-24) in good condition. Comparison of Figures 5 and 7 demonstrates that the concrete in the former is not air-entrained.

TABLE 3

AIR CONTENT OF CORES AND CYLINDERS FROM OTHER AREAS

Concrete		Core or Cylinder No.	Air Content, percent		
Location	Condition		Entrained	Entrapped	Total
Sidewalk, Area 2	Good	CON-7[a]	3.8	2.0	5.8
Exterior slab, Area 3	Good	-10[b]	2.8	1.1	3.9
Area 4	Good	-25C[c]			
		Top	0.3	0.3	0.6
		Bottom	0.5	0.5	1.0
		Average[d]	0.4	0.4	0.8

[a]Air-entrained concrete from Contract B.
[b]Air-entrained concrete from Contract C.
[c]Non-air-entrained concrete from Contract D.
[d]Averages computed as percentage of total number of points observed in all specimens examined that were found to lie in voids of each type.
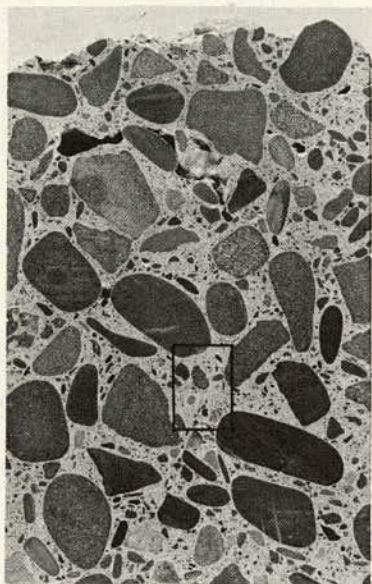
Figure 4. Sawed surface of top half of core CON-20, ½ natural size. From ¾ to 1 in. of top surface has scaled off. The section outlined is magnified in Figure 5.
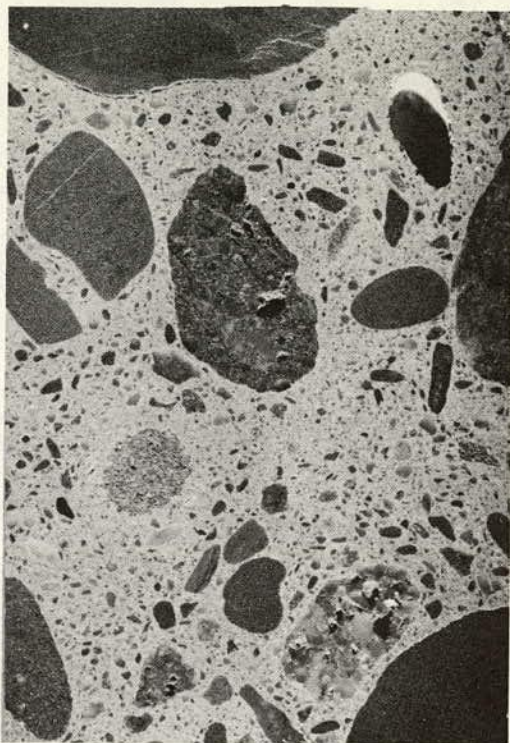
Figure 5. Sawed surface of core CON-20. This is the area outlined in Figure 4. The actual magnification is 4.4 times natural size.
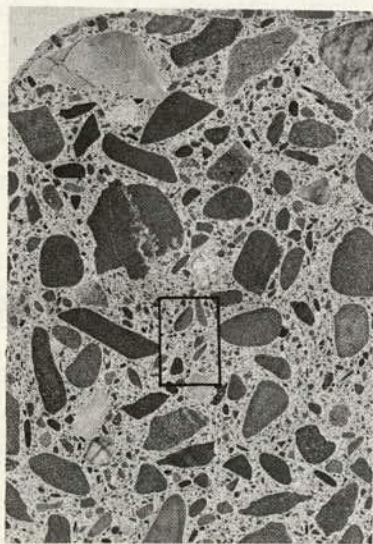
Figure 6. Sawed surface of top half of core CON-24, ½ natural size. Curbing was in good condition with formed and finished surfaces intact. The section outlined is magnified in Figure 7.
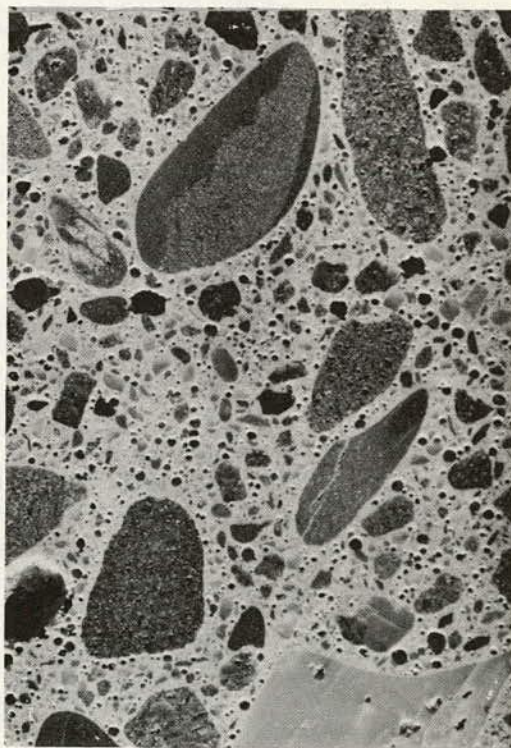
Figure 7. Sawed surface of core CON-24. This is the area outlined in Figure 6. The actual magnification is 4.4 times natural size.

TABLE 4

SPECIFIC GRAVITY OF CORES AND CEMENT CONTENT OF CONCRETE

| Core No. | Specific Gravity | Cement Content Determined Chemically | | | Cement Content from Reported Mixture Proportions and Air Content Micrometrically Determined | |
		Based on SiO_2, percent by weight	Based on CaO, percent by weight	Actual Cement Content, bags per cu yd[b]	Calculated Cement Content, bags per cu yd[c]	Actual Cement Content, bags per cu yd[d]
CON-1 to -3	2.26	11.2	12.6	4.53	5.08	5.17
-4 to -6	2.26	10.1	11.3	4.09	5.08	5.13
-13 to -15	2.26	11.5	12.3	4.64	5.08	5.06
-16 to -18	2.27	10.8	11.0	4.39	5.08	5.18
-19 to -21	2.26	12.1	16.0	4.89	5.08	5.21
-22 to -24	2.18	10.9	11.4	4.27	5.08	4.96

[a] $\frac{A}{B \cdot C}$, where A = weight of samples dried to constant weight at 105 C; B = weight of samples soaked 16 hours and surface-dried; and C = weight in water of samples after 16-hour soaking.
[b] Based on SiO_2 determination and specific gravity.
[c] Cement content of mixture described in commercial testing laboratory "second" recommendation presumably approved for use, including 3.6 percent air.
[d] Calculated from theoretical cement content using air contents in Table 2.

Cement Content

Table 4 gives cement contents of concrete from the affected area (Contract A) obtained (a) by chemical analyses and (b) from the mixture proportions by adjusting the theoretical cement content to an actual cement content using the micrometric air contents. It also gives the specific gravities determined.

The 18 cores obtained from six locations in the affected area were selected to provide samples of concrete in relatively good and relatively poor condition from areas separated by expansion joints. This selection provided three sets of concrete cores for comparison. Table 5 gives this comparison and includes the data developed in this examination.

Petrographic Examination

Petrographic examination of the concrete revealed no indications of deleterious chemical reactions or premature freezing.

DISCUSSION OF TEST RESULTS

Affected Area Concrete

It was the opinion of those who inspected the deteriorated concrete in April 1963 that the damage was caused by freezing and thawing and hastened by applications of de-icing salts. The concrete was believed to have been susceptible to such damage because of inadequate air-entrainment or low cement content, or both. The informa-

TABLE 5

COMPARISON OF CEMENT CONTENT, AIR CONTENT, AND PHYSICAL CONDITION OF CONCRETE, CONTRACT A

| Location | Core No. | Concrete Condition | Cement Content, bags per cu yd | | | Total Air Content, percent | With Air Entrainment | Without Air Entrainment |
			Chemical Determination[a]	Mixture Data and Micrometric Air Content[b]	Difference			
Sidewalk	CON-1 to -3	Good	4.53	5.17	0.64	1.9		X
	-4 to -6	Poor	4.09	5.13	1.04	2.6		X
Curbing	-13 to -15	Good	4.64	5.06	0.42	4.0	X	
	-16 to -18	Poor	4.39	5.18	0.79	1.8		X
Curbing	-19 to -21	Poor	4.89	5.21	0.32	1.2		X
	-22 to -24	Good	4.27	4.96	0.69	5.8	X	

[a] ASTM Designation C 85-54. [b] Mixture assumed to have been used, adjusted by micrometric air contents.

tion developed during the laboratory examination (Tables 2 through 5 and Figures 4 through 7) appears to indicate that the primary reason for the rapid deterioration of some of the concrete was freezing-and-thawing damage that occurred because the concrete was not sufficiently air-entrained. The cement contents determined by chemical analysis were, in general, close enough to the cement contents shown in the mixture proportions believed to have been used, adjusted by the air contents determined, to suggest that low cement was not a major factor responsible for the deterioration. Petrographic examination of the concrete revealed no indications that deleterious chemical reactions or premature freezing might have contributed to the deterioration.

The sidewalk concrete represented by cores CON-1 to -6 was non-air-entrained; the concrete in cores 1 to 3 was in good condition and that in cores 4 to 6 in poor condition. The explanation for this difference is probably twofold. First, cores 4 to 6 were taken nearer the curb and had more exposure to de-icing salt solutions produced by the melting of salts applied to the streets (i.e., the salt solution accumulated near the curb and passing traffic splashed it onto the sidewalk). Second, the concrete of cores 4 to 6 does have a cement content that is significantly lower than it should have been and is considerably lower than the cement content of cores 1 to 3. The air-entrained sidewalk concrete of low air content can be expected to suffer ultimately the same type of deterioration that the air-entrained concrete curbing of low air content has already shown. Any other air-entrained concrete of low air content present can also be expected to deteriorate much more rapidly than that protected from frost damage by an adequate air-void system.

The specifications called for all of the concrete used to be air-entrained and to contain 3 to 6 percent air. However, it has been shown that some of the concrete was not sufficiently air-entrained and contained only from 1.2 to 2.6 percent air (Table 2). The air contents (Table 3) of the concrete from the other three nearby areas were within the range required by applicable specifications.

Aggregates

The aggregates used in all of the concrete represented by the samples examined are natural sand and gravel. Test data on this material indicate relatively high absorption (2.6 percent for sand, 1.7 percent for gravel), but also high resistance to freezing and thawing in concrete (durability factor DFE = 87). More recent data show relatively similar results; an absorption of 1.8 percent for sand and 2.4 percent for No. 4 to ¾-in. gravel, but a DFE of 91. It is therefore indicated that, while by some traditional measures of physical quality these aggregates would be regarded as of only fair physical quality, the aggregates are entirely capable of being employed to make concrete having a high degree of frost resistance if the concrete has an adequate air-void system and is otherwise manufactured according to proper practices.

CONCLUSION

It was concluded that deterioration of the sort that developed on some of the curbs and sidewalks in the affected area could have been prevented, and that similar deterioration can be prevented in the future, even in cases where heavy applications of de-icing chemicals may have been used, by the use of air-entrained concrete containing an adequate air-void system.

ACKNOWLEDGMENTS

Scaling of Air-Entrained and Non-Air-Entrained Concrete

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RICHARD C. MIELENZ, Master Builders Division of Martin Marietta Corporation, Cleveland

•IN THE Great Lakes area, a residential alley paving 8 in. thick was laid directly on a clay subgrade. The paving was placed in two seasons, late June and early July, in each case. Surface scaling randomly distributed was observed at the time of the examination, which was made after the concrete placed the first season had gone through two winters and the concrete placed the second season had gone through one winter. Cars and delivery vehicles had been dripping chloride solution as they came in off the salted streets. At the time of examination, the appearance of the scaling was somewhat different for the two concretes. Deterioration of the concrete placed in 1957 and exposed for one winter was at a more advanced stage than that of the concrete placed in 1956 and exposed for two winters. Some pitting was evident in the concrete placed earlier, but surprisingly little cracking.

CASE HISTORIES

The records showed that the same concrete mixture possessing the same properties was requested from the ready-mixed concrete supplier for both seasons. The brand of cement was unknown. The same aggregate source was used, and it had had a fairly good record locally. Supposedly, the mix contained six 94-lb bags of portland cement, 6 fl oz of Darex air-entraining admixture, and 34 gal total water content per cu yd. Some water was added at the site to the 7-cu yd loads in the transit mixer, but the amount varied and there was no available record of it.

The foreman, who was talked to after the petrographic findings, claimed that for the most part he could not obtain water to sprinkle the surface in the finishing operation. The concrete had looked dry to him, it could be walked on in a couple of hours after finishing, and no attempt was made to cure it.

WEATHER DATA

Records of local weather during and immediately following the placing of the concrete during the summers of 1956 and 1957 and for the winters of 1956-57 and 1957-58 were obtained from the nearby station of the U.S. Weather Bureau.

The weather during the latter part of June and early July 1956, when the pavement of area 80 was placed, was hot and humid with day and night temperatures in the range of 100 to 59 F. The first freezing occurred on November 8 and during that month the minimum temperature was below freezing on 17 days, the minimum temperature being 15 F. During December, 21 days of freezing occurred, and January 1957 is reported to have been the coldest January in 17 years to that date; a minimum temperature of -3 F was recorded. The cold weather was accompanied by 2.10 in. of precipitation. February 1957 was relatively mild, with subfreezing temperatures occurring on 17 days. On only 3 days did the temperature not rise above the freezing point; thus, numerous cycles of freezing-thawing were experienced. Occasional freezing continued throughout March, and the last freeze of the winter was on April 14, 1957.

The period from June 25 to July 10, 1957, was warm to hot. Rainfall totaled 1.01 in. during this period. More than 7.5 in. of rain fell during July, and an additional 7.0 in. of rain were recorded during August. From July 2 to December 31, 1957, precipitation totaled 24.93 in. More than 9 in. of precipitation occurred during the last 3 months of
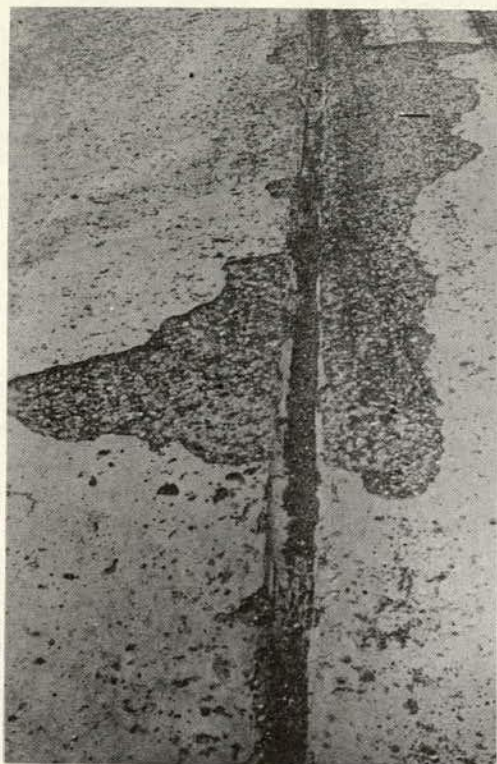
Figure 1. Typical scaling of concrete as it had occurred in areas 92, 93, and 98. Note localized pitting and spalling of the finished surface in advance of general scaling.

the year. The first freezing occurred on November 8, and there were 12 days during that month on which freezing temperatures were experienced; the minimum temperature was 15 F. Except for November 30, the maximum temperature exceeded 32 F. December 1957 was unusually warm, with 17 days in which freezing temperatures were encountered; the minimum temperature was 4 F. January and February 1958 included 52 days in which freezing temperatures occurred (minimum -5 F). In March 1958 there were 9 days in which the temperature fell below 32 F (minimum 28 F).

INVESTIGATION

Photographs of areas of the pavements at different stages of scaling were taken; Figure 1 shows several stages of deterioration, ranging from severe to lesser stages of symptoms. Four pairs of 4-in. diameter cores were taken at random in various areas. One core of each pair was tested for compressive strength (ASTM Designation C 42); the other was examined by petrographic methods and the air content was determined microscopically. The compressive strengths and visual impressions were as follows:

Area 80—Compressive strength of 5,150 psi; broken around aggregate particles; spongy-looking mortar; first season concrete.

Area 92—Compressive strength of 6,000 psi; good break through aggregate particles; good-appearing mortar except top 3 in. appeared less dense than did the concrete at greater depth; second season concrete.

Area 93—Compressive strength of 6,000 psi. Good break through aggregate; good mortar except top $\frac{1}{2}$ to $\frac{3}{4}$ in.; second season concrete.

Area 98—Compressive strength of 6,600 psi. Good break through aggregate particles; good mortar except top $\frac{1}{2}$ to $\frac{3}{4}$ in.; second season concrete.

PETROGRAPHIC EXAMINATION

The four cores extended through the thickness of the pavement. The top of the core from area 80 was partially scaled to a depth of $\frac{1}{16}$ to $\frac{1}{8}$ in., whereas the tops of the cores from areas 92, 93, and 98 were almost completely scaled to a depth of $\frac{1}{8}$ to $\frac{5}{16}$ in. In each instance, active progress of the scaling was indicated by roughly horizontal microfractures through the mortar matrix at depths of $\frac{3}{16}$ to $\frac{3}{8}$ in.

The top 4 in. of each core were sawed parallel to the axis of drilling, and plane surfaces were prepared by lapping. Air content of the concrete was determined by the microscopical point-count procedure in accordance with ASTM Designation C 457, with the following results:

Pavement Area	Air Content, Percent by Volume of Concrete	Pavement Area	Air Content, Percent by Volume of Concrete
80	6.30	93	0.90
92	0.26	98	0.45

The air-void system in the core from area 80 was typical of air-entrained concrete prepared in accordance with American Concrete Institute Standard 613, except that the air content was slightly higher than the usual recommendation for pavement concrete. The air voids were small and well distributed throughout the upper 4 in. of the core. The air-void system in the cores from areas 92, 93, and 98 was typical of non-air-entrained concrete; the air voids were characteristically large and sparsely distributed.

The concrete was generally hard, compact, and apparently properly proportioned. Except for the topmost portion of the core from area 80 (see the following paragraph) and the fractured near-surface portion of the cores from areas 92, 93, and 98, the cement paste matrix was dense and firm. Portland cement had been used; no mineral admixture, such as fly ash, was present. The examination revealed no evidence of excessive bleeding, segregation, or poor finishing practices. No appreciable concentration of mortar or laitance materials was present at or near the top surface.

The cement paste matrix in the topmost $\frac{1}{4}$ to $\frac{3}{8}$ in. of the core from area 80 was bleached almost white and was substantially weaker and more absorptive than the cement paste matrix in the concrete at greater depth. In the bleached zone, the cement paste matrix was very highly carbonated as a result of carbon dioxide penetration into the concrete at an early age. Such a bleached and highly absorptive zone was not present in the cores from areas 92, 93, and 98.

The aggregate was a combination of natural gravel, crushed gravel, and natural sand. The aggregate in the four cores was similar. The coarse aggregate and the coarse fractions of the sand included moderate proportions of chalcedonic and quartzose cherts and cherty dolomites. No contaminating substances or natural coating materials were detected. Secondary rims resulting from cement-aggregate reactions were observed on occasional particles of dolomite, on cherts, and on cherty zones in dolomite particles. Trace amounts of alkalic silica gel were detected in the core from area 92. In this core, occasional particles of chert in the coarse sand fractions were softened as a result of the alkali-silica reaction. No cracking or other evidences of distress were observed in association with the affected particles.

DISCUSSION OF RESULTS

The concrete of area 80 was subjected to daytime temperatures exceeding 90 F shortly after placing was completed; it was not cured. Explicit correlation of the concreting operations with the local weather conditions is not possible because of lack of information on exact dates during which the work was accomplished. However, it is evident that drying and carbonation of the near-surface region of the pavement relate to drying conditions immediately following the placement. The winter of 1956-57 was unusually severe (especially during January 1957), and severe freezing and thawing conditions were experienced a second time in the winter of 1957-58, especially in December, January, and February, when numerous cycles of freezing and thawing occurred.

The concrete of areas 92, 93, and 98 was subjected to high temperatures in the period during and immediately following placement. However, the placing operations were accompanied and followed by heavy precipitation that persisted through August and resumed in October. With the onset of freezing in early November, the concrete probably was in a nearly saturated condition. A high degree of saturation of the pavement would be promoted also by direct contact of the concrete with a clay subgrade. Hence, the non-air-entrained concrete would be especially susceptible to breakdown in freezing and thawing in spite of a relatively low water-cement ratio. The observed compressive strength of the three cores from these areas suggests a net water-cement ratio substantially less than 6 gal per 94-lb sack of cement (0.53 by weight).

CONCLUSIONS

1. Area 80 constituted well-air-entrained concrete, but the topmost $\frac{1}{4}$ to $\frac{3}{8}$ in. of the pavement was susceptible to disruption by freezing and thawing because of high absorptivity, resulting from lack of adequate curing, and early drying and carbonation of the cement paste matrix. The action of freezing probably was aggravated by the presence of de-icing salts.

2. Areas 92, 93, and 98 comprised non-air-entrained concrete that was not resistant to freezing and thawing while wet, especially in the presence of de-icing salts. Examination of the local weather records indicates that, although they were not purposefully cured, these pavements could not experience any substantial drying following placing and were well saturated with water at the time of freezing and thawing because of heavy precipitation during the late summer and early fall of 1957. Thus, this concrete of low water-cement ratio was susceptible to rapid disintegration during repeated freezing.

3. Incipient cement-aggregate reactions involving dolomites, chalcedonic cherts, and cherty phases of dolomites had occurred, but no deleterious effects were indicated by the petrographic examination.

4. The experience illustrates the fact that air-entrainment will not ensure adequate durability of exposed concrete in the absence of good concreting practices. In this instance, the defect leading to failure of concrete in area 80 was the result of inattention to curing and protection of the concrete from early drying prior to development of a sound cement paste matrix of low absorptivity. The defect leading to failure of the pavement surface in the other areas was the absence of a proper entrained air void system in the concrete.

Random Cracking of Bridge Decks Caused by Plastic Shrinkage

HOWARD H. NEWLON, JR., Virginia Department of Highways

●IN THIS STUDY the problem was to determine the causes of severe random cracking that had affected two of four reinforced concrete spans on an Interstate System bridge. This cracking, shown in Figure 1, was severe enough to allow water and de-icing salts to pass through the slab and attack the stringers as shown in Figure 2. The cracking covered 100 percent of the surface of span 1 and approximately 25 percent of the surface of span 4. The remaining two slabs were unaffected. The reasons for determining the causes of the cracking were to provide bases (a) for preventing recurrence on subsequent projects and (b) for recommending repair procedures.

INVESTIGATION OF THE PROBLEM

The overall characteristics of the cracking based on visual survey were similar to those attributed by Lerch (3) to "plastic shrinkage" and suggested that the cracking had occurred early in the life of the deck. Because the cracking had not been noticed until after two winters of traffic, the first step was to establish the nature of the cracking in terms of time of occurrence.

To accomplish this, two 4-in. diameter cores were removed from the cracked area for petrographic examination. Examination of these cores indicated that the cracks traveled around rather than through aggregate particles. Considering this finding and the fact that the strength of the concrete was high based on information from construction records and the petrographic examination, it was possible to conclude that the cracking occurred when the concrete was relatively green and perhaps fresh. This conclusion was supported by an earlier investigation (4) of a similar though less severe occurrence in which it was shown, from a petrographic examination of cores, that the curing compound actually penetrated into the crack as shown in Figure 3, thus proving that formation of the crack preceded or very closely followed the application of curing compound. Results from an earlier, unpublished laboratory study of similar quality concrete suggested that the absence of broken aggregate associated with the crack indicates that the crack occurred during the initial 24 hours.

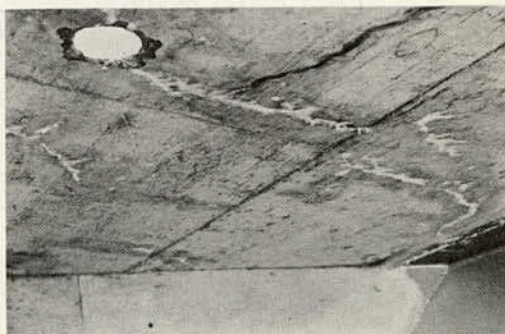Figure 1. Random cracking in span 4 (surface damp).

Figure 2. Underside of span 4 showing leaching through cracks and corrosion of longitudinal stringer. Core was removed from hole at upper left.
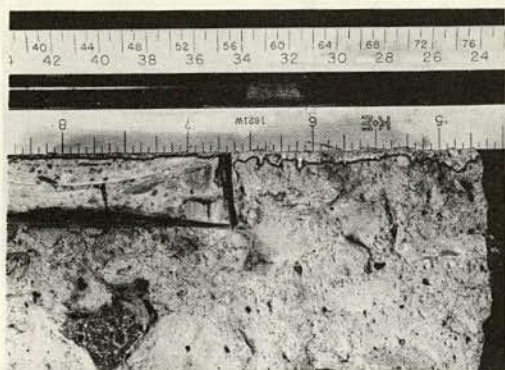
Figure 3. Top of core removed from a bridge showing random cracking. The right side shows penetration of curing compound into crack to a depth indicated by the dark line. The normal depth of curing compound is indicated on the sawed left section of core.

The petrographic examination of the cores revealed no characteristics indicative of any adverse reactions that would suggest delayed or continuing formation of cracks.

Although the cracking was mostly random, in certain areas it appeared to be influenced by the top reinforcing steel. This may be seen in Figure 1. This is also consistent with the view of plastic shrinkage expressed by Blakey (1) that, as evaporation takes place and the surface dries, the resistance to sedimentation is greater over the reinforcement, which then controls the location of the crack that would otherwise form randomly.

After it was established beyond reasonable doubt that the problem was early and probably plastic cracking, the next step was to determine the factors that influenced the crack formation. Operating on the assumption that the cracking was caused primarily by plastic shrinkage, data concerning the mixtures and weather conditions were accumulated for study. Lerch (3) has identified the following factors as important contributors to the development of excessive plastic shrinkage: (a) high concrete temperatures (more precisely, large differential temperature between concrete and surrounding air); (b) low relative humidity; and (c) high wind velocity.

The construction, materials, and testing records indicated no differences among the concretes in the four spans that would be expected to result in high shrinkage. The information of primary interest in these records was the time during which concreting took place. Routine construction records did not contain sufficient weather data to
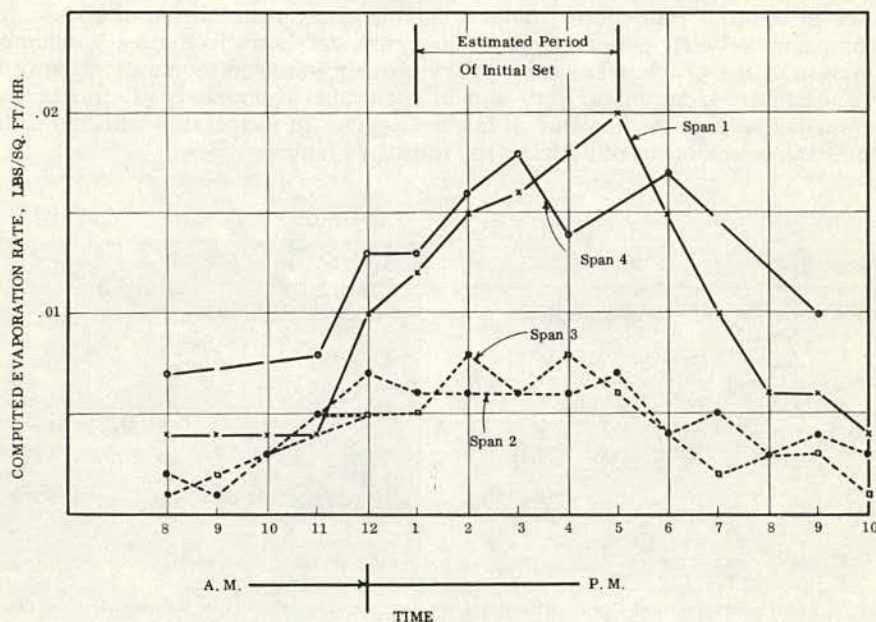

Figure 4. Variation with time of computed evaporation rates for spans 1 and 4 (cracked) and 2 and 3 (uncracked).

TABLE 1

IMPORTANT ATMOSPHERIC CONDITIONS FOR SPANS
4 (CRACKED) AND 2 (UNCRACKED)

Hour	Air Temperature (deg F)		Wind Velocity (mph)		Relative Humidity (percent)	
	Span 4	Span 2	Span 4	Span 2	Span 4	Span 2
8 a.m.	37	32	16	0	57	88
9 a.m.	—	37	—	2	—	82
10 a.m.	—	42	—	3	—	76
11 a.m.	44	49	16	4	43	45
12 noon	48	52	14	3	43	41
1 p.m.	49	53	13	3	38	28
2 p.m.	50	57	17	4	36	27
3 p.m.	53	59	18	3	33	30
4 p.m.	54	59	15	4	32	43
5 p.m.	55	57	16	4	30	47
6 p.m.	—	52	—	2	—	50
7 p.m.	—	49	—	6	—	60
8 p.m.	49	47	22	2	50	65

indicate conditions at critical times. Fortunately, however, the project was within 10 miles of an airport at which the Federal Aviation Agency hourly monitors important weather conditions. From this station, hourly measurements of air temperature, wind velocity, and relative humidity were obtained for use in estimating the rate of evaporation as recommended by the Portland Cement Association based on the work of Lerch (5). Unfortunately, concrete temperatures were not available so evaporation rates were computed based on the assumption that the concrete temperature was 10 F higher than that of the air.

It was assumed, on the basis of data from a previous study (2), that the time between placement and setting, during which plastic cracking could occur, was 5 hours. Computed evaporation rates, based on a differential temperature between concrete and air of +10 F, are plotted in Figure 4. The evaporation rates for placement days on which slabs cracked are similar and significantly higher than those for the days on which unaffected slabs resulted. The computed values are above 0.10 lb/sq ft/hr and approach 0.20 lb/sq ft/hr. Guides furnished by the PCA (5) state that there is no way to predict with certainty when plastic shrinkage cracking will occur. When the rate of evaporation is as high as 0.2 to 0.3 lb/sq ft/hr, precautionary measures are almost mandatory. However, if the rate of evaporation exceeds 0.1 lb/sq ft/hr, cracking may occur.

Although plastic shrinkage cracking is normally associated with hot weather concreting, experience in Virginia has shown that spring and fall are more critical times because of the occurrence of higher winds and lower humidities than are common in summer. For comparative purposes, the important atmospheric data for span 4 (cracked) and span 2 (uncracked) are given in Table 1. In these cases the controlling factor was the wind velocity. In other cases where the wind has been moderate and the temperature higher, the relative humidity has been the controlling factor.

Because the purpose of this paper is to indicate very briefly the approach used to document the cause of the observed distress, an extended discussion of plastic cracking is not warranted. This may be found in the references cited. Many factors exert an influence to reduce the severity of the cracking and some of these may not yet be recognized. The results of the investigation outlined in this case study have, however, been substantiated in other cases studied by the author as well as in an extensive case study in progress.

The results obtained in this and other case studies indicate that when early random cracking occurs, the severe drying conditions as reflected in the computed evaporation rate exist. The existence of the severe conditions does not always result in this early cracking. This would suggest that the cracking can be prevented, or at least can be reduced in severity, by construction factors that modify the influence of the four major variables of air temperature, concrete temperature, wind velocity, and relative humidity. This suggestion confirms the views expressed in the references cited earlier, as well as the study reported by Shalon and Ravina (6), that there is still need for study of the phenomenon of early cracking.

ACKNOWLEDGMENTS

REFERENCES

1. Blakey, F. A. Concrete in Motion. Div. of Building Research, CSIRO of Australia, Rept. L 12, July 1962.
2. Hilton, M. H., Newlon, H. H., Jr., and Shelburne, T. E. Research Relating to Bridge Decks in Virginia. AASHO Bridge Committee Session, New York, Oct. 1965.
3. Lerch, William. Plastic Shrinkage. Jour. American Concrete Institute, Title 53-44, Feb. 1957.
4. Newlon, H. H., Jr. Concrete Investigation No. 5—Determination of the Causes of Cracking in a Bridge Deck. Virginia Highway Research Council, Feb. 1963.
5. Curing of Concrete. Portland Cement Association, Concrete Information Sheet ST-95, 1963.
6. Shalon, R., and Ravina, D. Studies in Concreting in Hot Countries. Symposium on Concrete and Reinforced Concrete in Hot Countries, RILEM, Haifa, July 1960.

Discussion

BRYANT MATHER and H. G. GEYMAYER, U.S. Army Engineer Waterways Experiment Station—The author has presented in a most interesting and instructive way a case history of plastic shrinkage cracking. This condition has been considered to be a reaction of unset concrete to the consequences of certain combinations of ambient conditions that result in an excessive rate of evaporation of moisture from the concrete surface. Little or no attention appears to have been given to other factors that may influence the degree to which a given concrete surface develops cracks in this period.

At the instant the consolidation of the concrete has been accomplished, it might be assumed that the surface of that concrete could be considered to have no tensile stresses. If, over a period of time, such stresses develop so that at some later moment the stresses have a magnitude in excess of the tensile strength that develops concurrently in the surface, cracking will occur. However, the magnitude of the stresses is not predictable entirely from the evaporation rate nor is the development of tensile strength a constant for all concretes.

In his last paragraph, the author suggests that the existence of severe evaporation conditions does not always produce cracking. We believe it might also be suggested that, when cracking does occur, the stresses that cause cracking are not always caused solely by the consequences of the evaporation induced by the severe ambient drying

conditions. Some of the tensile stress that can, and certainly must in some cases, contribute to early cracking must be produced by autogenous volume changes in the concrete itself as hydration of the cement proceeds. Similarly, the failure of cracking to develop under severe ambient conditions must, in some cases, be due to the prolonged retention of plasticity, the accelerated development of tensile strength, or the possession by the concrete of a pore structure that causes the drying influence to act with reduced efficiency in extracting moisture from the mass. These appear to be some of the factors that merit further study.

HOWARD H. NEWLON, JR., Closure—The author agrees that the factors outlined by Mather and Geymayer can be influential in the occurrence of cracking or lack thereof. It did not seem necessary to consider them in the specific case described, which is most typical of those brought to the author's attention. Such factors are always considered when studying cases of cracking where records show that severe environmental factors did not exist. In the vast majority of cases of slabs exhibiting the type of cracking described in the paper, detailed study has shown that high wind and low humidity did exist. Seldom, if ever, have we studied cracking of this type where the severe environment was absent.

Deterioration of Concrete Chimneys

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Samples from two damaged concrete factory chimneys were
examined. The chimneys had been exposed to severely aggres-
sive conditions. During the field examinations, cracks in the
concrete were observed in both cases. At some places the
cracking was so serious that pieces of concrete were loose.
Concrete samples from the chimneys were examined by X-ray
diffraction analyses, chemical analyses, and by thin-section ob-
servations. The concrete from both chimneys was carbonated
and in both samples sulfate reactions had occurred. Further-
more, alkali-aggregate reactions had occurred in the concrete
from one of the chimneys.

•SAMPLES from two deteriorated chimneys, one located by the Mediterranean Sea and
the other in Northern Europe, were investigated. The external climate of the location
seemed to be of minor importance, however, compared with the chimney milieu. This
environment was characterized by high temperatures, great and rapid variations of
temperature, and the presence of sulfuric acid and carbonic acid.

CHIMNEY BY THE MEDITERRANEAN

The 61-meter high reinforced concrete chimney, built in 1949, had suffered damage
in various sections of the shaft. The section from the base to somewhere above the be-
ginning of the opening of the smoke inlet was practically undamaged, whereas the fol-
lowing section up to 16 meters above the base was seriously cracked, with larger and
smaller loose concrete slabs or shells on the exterior surface, reaching in depth to the
reinforcement. Similar damage was found in various places up to near the top. The
reinforcement was located too far inside the outer face of the shaft, with up to 7 to 8
cm of concrete cover, and the loose concrete slabs were generally of this order of
thickness. The reinforcing steel behind the slabs that cracked off the steel was only
slightly corroded.

A thin section was prepared from a piece of the deteriorated concrete. From ob-
servations of the thin section, it could be concluded that the concrete was strongly car-
bonated along with secondary precipitations of gypsum in the cement paste. Flint peb-
bles with distinct symptoms of alkali-aggregate reactions were observed (Fig. 1).

CHIMNEY IN NORTHERN EUROPE

The chimney in Northern Europe was built in 1957 of reinforced concrete. In the
autumn of 1965 it was inspected and it was noted that the concrete had scaled off the
steel in parts of the upper 60 to 70 cm of the shaft (Fig. 2). The reinforcement was
rusting and a 2-mm thick layer of rust could easily be removed. In the construction
joint 13 meters from the top, loose flakes of concrete were observed 15 cm above the
joint.

A concrete specimen from the top section was examined by chemical analysis, X-ray
diffraction analysis, and a thin-section investigation. The concrete proved to have un-
dergone leaching and carbonation along with the corrosion of the reinforcement. On
the outer surface of the specimen, a coating of the mineral syngenite $[K_2Ca(SO_4) \cdot H_2O]$
was observed (Fig. 3), while a coating consisting of the hemihydrate $CaSO_4 \cdot \frac{1}{2}H_2O$ was
found on the inside of the chimney and situated in a crack in the structure.

Figure 1. Thin section of concrete from factory chimney near the Mediterranean Sea. Flint pebbles to the right have darkened rim caused by partial solution resulting from alkaliaggregate reaction. Air bubble in center of picture is partly filled by alkali-silica gel, fractured during drying.

THE MILIEU OF THE CHIMNEYS

In spite of the difference in the observations from the two chimneys, it seems useful to consider them together because the milieu has a special character. Among the different factors that may contribute to the deterioration of the concrete in a chimney, the following can be mentioned:

1. The content of carbon dioxide in the smoke accelerates the carbonation of the concrete. The ability of the concrete to protect the reinforcement against corrosion is thereby reduced and formation of rust may result in spalling of the covering layer.

2. The temperature variations, presumably most pronounced at the top of the chimney, cause unequal thermal deformation of the concrete, possibly accompanied by crack formations.

3. The content of sulfur gases in the smoke condenses on the sides of the chimney, thereby developing sulfuric acid that deteriorates the concrete. The reaction products formed are gypsum and other calcium-sulfates.
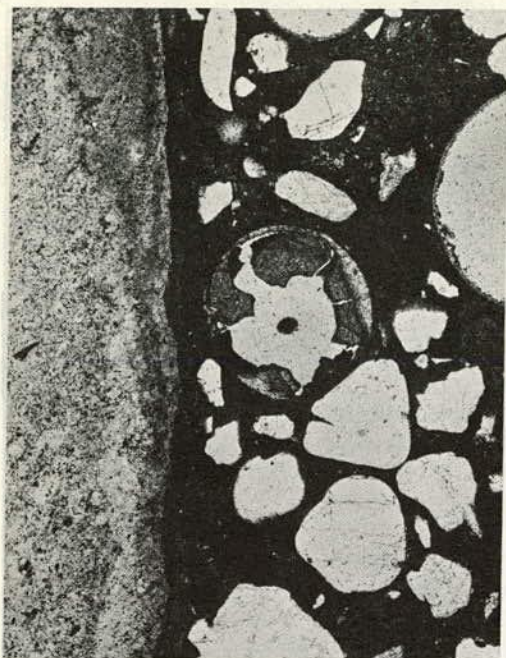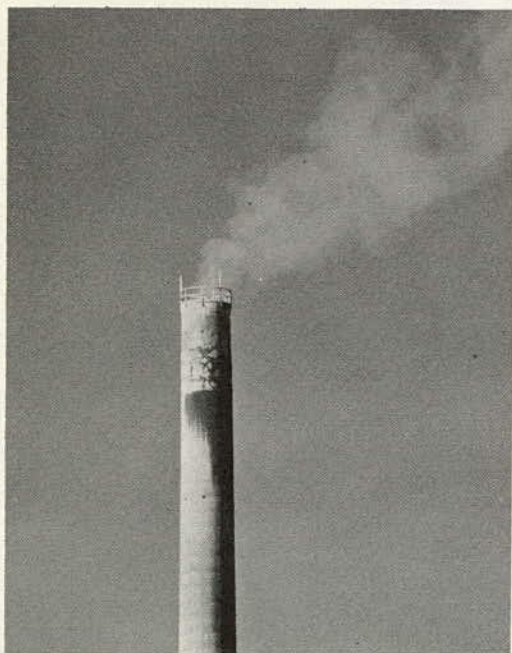

Figure 2. Top section of concrete chimney showing disintegration.

Figure 3. Total photo of a thin section prepared from a deteriorated piece of concrete taken from the outside upper part of a chimney (crossed nicols, 1.26x magnification).

DISCUSSION OF INVESTIGATION

It is supposed that a combination of the three factors mentioned previously caused the damage to the chimneys investigated. The presence of gypsum and of reacted flint pebbles in the concrete of the chimney near the Mediterranean indicates that sulfate attack and alkali-aggregate reactions in a combined action have caused a particularly rapid disintegration. The serious damage to the chimney is also caused by a too-thick concrete cover of reinforcement. The ring reinforcement, being too far away from the outer shaft face, has been unable to prevent the cracking of the outer part of the concrete as a result of tensile stress caused by the heating of the interior of the shaft.

The formation of cracks in the Northern European chimney probably resulted from the great and rapid temperature variations at the top of the shaft. But phenomena such as leaching, carbonation, and precipitation of corrosion products all contribute to a reduction of the strength of the concrete and thereby to the risk of crack formation. The coating of syngenite on the outer surface of the chimney probably came from potassium and sulfate from the smoke; the calcium may originate from either the smoke or the concrete. The presence of a hemihydrate instead of gypsum in the coating in the fracture could be explained from the temperature of the chimney.

CONCLUSION

The conclusion from the investigations is that concretes used for factory chimneys are exposed to a particularly aggressive milieu. This should be taken into consideration in construction of chimneys and reasonable precautions specified, such as (a) use of sulfate-resistant cement, (b) use of aggregate resistant to alkali-aggregate reactions, (c) possible treatment with epoxy resin, and (d) careful construction.

Deterioration of Some Concrete Structures of a Sulfuric Acid Plant

G. M. IDORN and H. KROGH, Concrete Research Laboratory, Karlstrup, Denmark

The paper describes an investigation of deteriorated concrete structures of a sulfuric acid plant, including some isolated concrete structures, the foundation of an acid silo, and a harbor quay for storage of pyrites. Thin-section examinations of samples of the concrete showed that the cement paste had been transformed into a microcystalline porous substance resembling gypsum. Chemical analyses of the groundwater, of the water in the harbor, and of the water in a ditch of the quay revealed a high content of sulfate ions and, in the water in the ditch, a high content of iron (+2) ions. The atmosphere smelled of sulfur dioxide. It is concluded that the concrete is probably deteriorated by the attack caused by (a) sulfur trioxide in the atmosphere combining with water, (b) sulfuric acid waste, and (c) the lowering of pH in the disintegration of pyrites. Low-C_3A cement was recommended for the future construction of concrete structures at the plant.

•THIS PAPER deals with the problem of deterioration of concrete structures near a sulfuric acid plant as a result of the aggressive environment. The investigation involved isolated concrete structures, the foundation of an acid silo, and a quay for storage of pyrites.

An investigation of the isolated structures revealed disintegration to such an extent that the aggregates became visible. The reinforcement bars in the structures were corroded. The concrete of the existing quay was not attacked, but the inspection was difficult because the foundation was coated with a layer of white mud.

ENVIRONMENT AND EXAMINATION OF THIN SECTIONS

The atmosphere smelled of sulfur dioxide. Sulfur dioxide and probably sulfur trioxide existed in the atmosphere and reacted with water, giving acid. Samples of the water in the harbor, of the groundwater, and of the water in a ditch of the quay were taken. Chemical analyses of these water samples showed a high content of sulfate ions and, in the water in the ditch of the quay, a high content of iron (+2) ions.

Specimens were taken of concrete from the isolated structures, from the foundation of the acid silo, and from the quay, and thin sections of the specimens were made. Examinations of the thin sections (thickness 20 μ) showed that most of the cement paste was replaced by a microcrystalline substance. The substance had a gray color in crossed nicols and was colorless in parallel nicols. Some of the small crystals had a distinct cleavage. Some cracks were filled by a substance that had a banded structure in crossed nicols; this substance resembled gypsum.

DISCUSSION OF RESULTS

The indications from the observations were of an acid attack on the concrete structures and on the foundation of the acid silo, and an attack on the quay caused by weathering of pyrites. Sulfur trioxide combines with water to yield sulfuric acid. The acid attacks the concrete by dissolving calcium hydroxide and thereafter the cement gel, precipitating gypsum. As sulfuric acid is formed on the surface, the deterioration

mechanism will not be serious until the concrete has been exposed to acid attack for a period of time. The attack on the structures was most likely due to acid made from sulfuric dioxide and sulfur trioxide. The foundation from the acid silo seemed to have been exposed to acid attack, probably because some sulfuric acid had been spilled.

The disintegration of the pyrites was as follows:

$$\begin{matrix} FeS \\ FeS_2 \end{matrix} \rightarrow FeSO_4 \rightarrow Fe_2(SO_4)_3 \rightarrow Fe_2O_3 \cdot aq$$
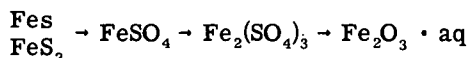
A small amount of oxygen leads to oxidation of pyrites to iron (+2) sulfate and sulfuric acid. Iron (+2) sulfate is dissolved in the solution (pH < 10) and comes into the cement paste. Hydroxide ions in the cement paste precipitate iron (+2) ions as iron (+2) hydroxide, which is oxidized by the oxygen dissolved in the water to iron (+3) hydroxide or iron (+3) oxide. The solution in contact with the solid iron (+3) oxide has pH < 3 (1). The attack inside the pores of the concrete is caused by the low pH solution.

Oxidation of pyrites to iron (+3) sulfate gives iron (+3) oxide on the surface and sulfuric acid; iron (+3) sulfate is only dissolved in solutions with a pH < 6. The attack is consequently concentrated at the surfaces of the concrete and is therefore not as dangerous as the attack of iron (+2) sulfate.

The high concentration of sulfate ions offers a risk of sulfate reaction leading to precipitation of gypsum and expansion of the cement paste.

· CONCLUSION

The deterioration of the concrete was found mainly to be due to an attack of acid and an attack caused by lowering of the pH in the disintegration of pyrites. The following procedures were recommended:

1. Use limestone as subgrade under the concrete pavement of the quay. Maintain a high pH in the concrete.
2. Use cement with a small content of C_3A. The high concentration of sulfate ions offers a risk of sulfate reactions.
3. Specify the manufacture of a high-quality, impermeable concrete.
4. Inspect the concrete structures periodically and check the chemical composition of the groundwater.

REFERENCE

1. Bastiansen, R., Moum, J., and Rosenquist, I. Th. Bidrag til belysning av visse bygningstekniske problemer ved Oslo-områdets alunskifere. Norges Geotekniske Institut, Serie: NGI Publisjon No. 22, Oslo, 1957, 70 pp.

Disintegration of Concrete in Foundation and Anchorage Blocks for an Aerial Mast

G. M. IDORN and P. NEPPER-CHRISTENSEN, Concrete Research Laboratory, Karlstrup, Denmark

Alkali-aggregate reactions were found to be one of the causes of deterioration of the concrete in foundation and stay anchorage blocks for an aerial mast. Freezing-thawing and vibrations transmitted to the foundation and blocks from the stays were probably contributing factors in the disintegration process; the question is still open as to which of the three factors was primary and which secondary. The risk of deleterious alkali-aggregate reactions in concrete containing reactive aggregates is increased when the structure of which the concrete forms a part is carrying relatively heavy dynamic loadings or is subject to the action of other disintegrative forces such as frost action.

•THE PAPER describes an investigation aimed at clarifying the causes of heavy disintegration of concrete as observed in foundations for an aerial mast.

The structure consists of a steel mast, 230 m high, supported on a concrete foundation and fixed in a vertical position by 15 steel cables. The stays lie in three vertical planes forming angles of 120 deg between each other with five stays in each plane. At one end the stays are attached to the mast, at the other end they are attached to large bent steel bars anchored in reinforced concrete foundations (Fig. 1). The arrangement of the foundation and anchorage blocks is shown in Figure 2.

Each anchorage block is about 3 m long, 1.5 m wide, and 2.5 m high with about 1 m visible above ground level.

Figure 1. Anchorage block 8 (Fig. 2) with coarse cracks and map-cracking.
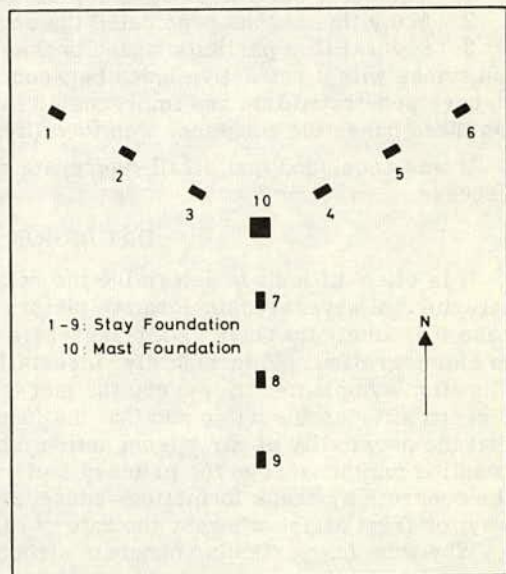

1-9: Stay Foundation
10: Mast Foundation

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Figure 2. Arrangement of anchorage blocks and foundation (not in scale).

Figure 3. Close-up of mast foundation (10 in Fig. 2) showing extensive map-cracking and pop-outs.

Few and incomplete data were available on the materials and mix proportions used for the concrete in the foundations. However, it could be ascertained that ordinary portland cement, ordinary Danish pit aggregates, and "foundation concrete", with a relatively low cement content (usually about 150 kg/cu m) and a relatively high water content, had been used. Danish pit aggregates usually contain 10 to 30 percent weak, porous flint and limestone. The average content of alkalies in ordinary Danish portland cement is about 0.7 percent.

The foundation and anchorage blocks are situated in agricultural fields. They are subject to the action of the weather, which in Denmark is characterized by a rather high relative humidity during the whole year and a great number of freezing-thawing cycles during the winter.

INVESTIGATION OF STRUCTURE

After 3 years of service (the blocks and foundation were cast in the summer of 1957), cracks began to appear in the foundations. After 5 years of service, the foundations were extensively cracked, with map-cracking, single coarse cracks, and pop-outs (Figs. 1 and 3). Disintegration was most pronounced in foundations 7, 8, and 9 (Fig. 2).

During the field inspection, samples of the concrete were taken for laboratory investigation. The samples were examined by microscope and the findings were as follows:

1. The outer surface of the samples was map-cracked;
2. Many fine cracks penetrated the cement paste; and
3. Several flint particles at the broken faces were coated with a glassy, isotropic substance with a refractive index between 1.45 and 1.48. The same substance seemed to have penetrated into and impregnated the mortar surrounding the flint aggregates. On these bases the substance was identified as alkali-silica gel.

It was concluded that alkali-aggregate reactions had taken part in the disintegration process.

DISCUSSION OF RESULTS

It is often difficult to determine the relative influence on the durability of a concrete structure of several disintegrative factors acting at the same time. In the present case it is likely that both alkali-aggregate reactions and frost action have contributed to disintegration. Unfortunately, no detailed description exists of the very first deterioration symptoms. However, the fact that visible deterioration did not occur until 3 years after construction and that the foundations were cast during summertime, so that the possibility of early frost action may be ruled out, indicates that alkali-aggregate reaction may have been the primary factor. An opening up of the interior structure of the concrete by crack formations caused by alkali-aggregate reaction would prepare the way for frost attack whereby the rate of disintegration would be increased.

Why was this particular concrete structure not able to withstand the disintegrative forces longer than 3 years? The initial quality of the concrete for the exposure conditions is believed to have been inadequate, as is often the case. However, one other

factor should be noted: Dynamic loadings caused by wind forces acting on the mast are transmitted through the cables to the anchorage blocks and through the base of the mast to its foundation. These dynamic loads, such as vibration and impact, create tensile stresses in the concrete and contribute to the formation and extension of cracks. This point of view is supported by the fact that southern winds prevail most of the year in Denmark. Thus anchorage blocks 7, 8, and 9 in the north-south oriented row would be expected to be most damaged, which was actually the case. A similar case of anchorage block deterioration caused by alkali-aggregate reaction has been described by Plum, Poulsen, and Idorn (1).

Observations on other structures—e.g., a crane bridge, particularly the parts where braking of the crane takes place, and road curbstones frequently subjected to collision with cars—also indicate that dynamic loadings may sometimes be a contributing factor of deterioration.

CONCLUSION

Combined action of alkali-aggregate reactions, freezing and thawing, and dynamic loading were found to be the cause of rather heavy and rapid deterioration of anchorage blocks and foundations for an aerial mast. The initial quality of this concrete was not adequate for the exposure conditions present.

Recommendations for repair constituted (a) removal of all disintegrated concrete to a depth of at least 15 cm from the surface; (b) careful cleaning of the new surface; (c) casting of a covering of high-quality reinforced concrete on top of the old concrete, the high-quality of the concrete being attained by using nonreactive aggregates, a water-cement ratio not higher than 0.55, and a cement content not less than 400 kg/cu m; (d) careful compaction; and (e) wet curing for at least one week after casting.

No new damage has been reported thus far.

REFERENCE

1. Plum, N. M., Poulsen, E., and Idorn, G. M. Foreløbig oversigt over alkalireaktioner i beton i Danmark. Ingeniøren, No. 27 and 28, Oct. 1957.
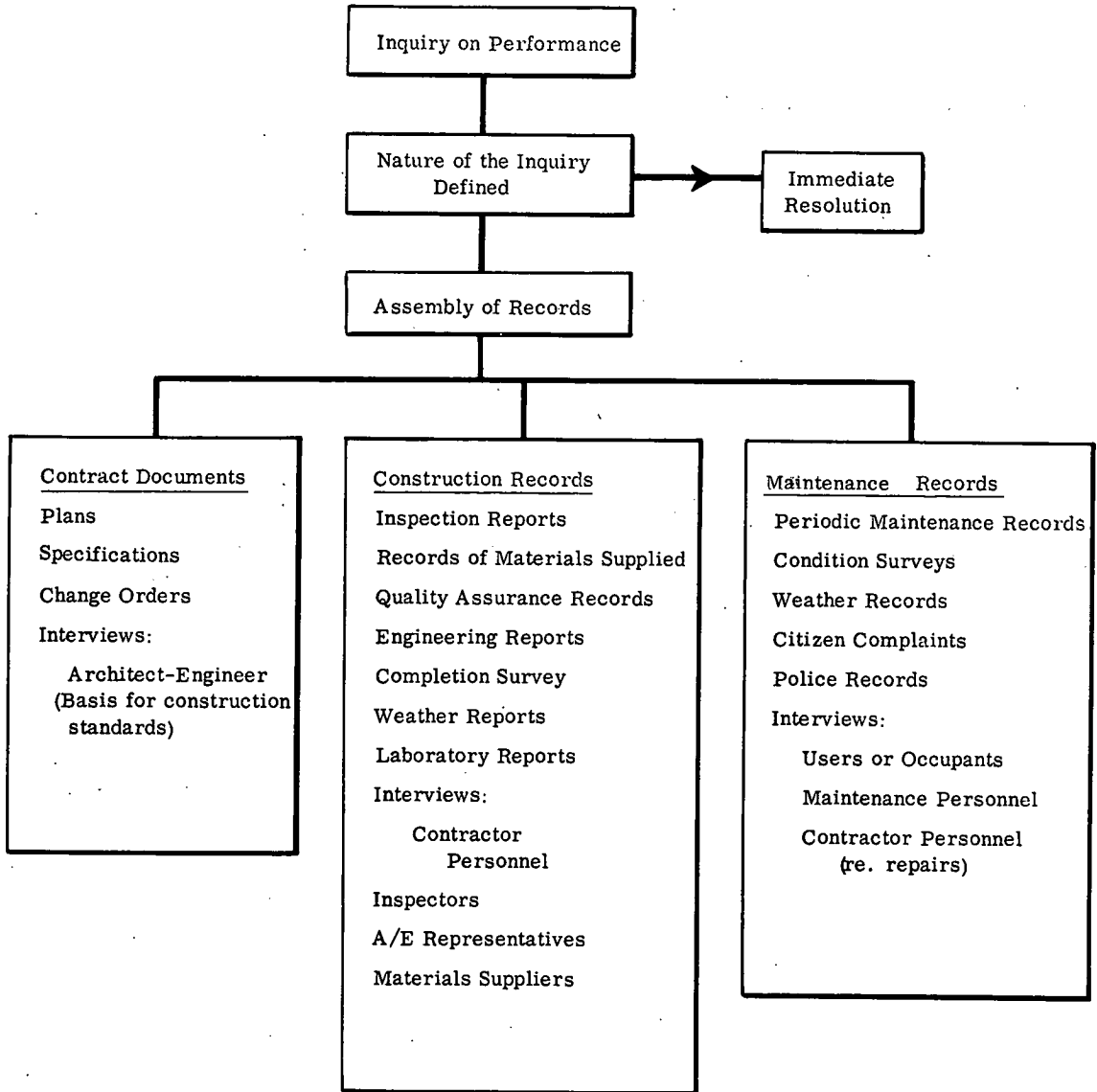
Part III

Appendixes

Appendix A
Bibliography

1. Guide for Making a Condition Survey of Concrete in Service. Jour. of American Concrete Institute, ACI Committee 201, Nov. 1968.

2. Cordon, W. A. Freezing and Thawing of Concrete—Mechanisms and Control. American Concrete Institute, Monograph No. 3, 1966.

3. Feld, Jacob. Lessons From Failures of Concrete Structures. American Concrete Institute, Monograph No. 1.

4. Idorn, G. M. Durability of Concrete Structures in Denmark. Technical University of Denmark, Copenhagen, 1967.

5. Mather, Bryant. The Behavior of Concrete. Third Annual Shaw Lecture, Dept. of Civil Engineering, North Carolina State Univ., Raleigh, 1967.

6. Durability of Concrete Bridge Decks: Report No. 1—Kansas; Report No. 2—Michigan; Report No. 3—California; Report No. 4—Missouri; and Report No. 5—California, Illinois, Michigan, Minnesota, New Jersey, Ohio, Texas, and Virginia. Portland Cement Association, Bureau of Public Roads, and Various State Highway Departments.

7. RILEM Symposium on Durability of Concrete 1969, Parts I and II. Academia, Prague, 1969.

8. Swenson, E. G., ed. Performance of Concrete: Resistance of Concrete and Other Environmental Conditions. Univ. of Toronto Press, 1968.

9. Tamburger, K. G., and Smith, Peter. A System for the Strength Evaluation of Concrete Bridges. Ontario Department of Highways, Report No. 1R25, Oct. 1968.

10. Waddell, J. J., ed. Concrete Construction Handbook (particularly chapters 7, 44, and 45). McGraw-Hill, New York, 1968.

11. Whitehurst, E. A. Evaluation of Concrete Properties From Sonic Tests. American Concrete Institute, Monograph No. 2, 1966.

12. Woods, Hubert. Durability of Concrete Construction. American Concrete Institute, Monograph No. 4, 1968.

Appendix B
Assembly of Records for Investigation of Performance of Concrete in Service

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                    │   Inquiry on Performance    │
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                    ┌─────────────────────────────┐        ┌───────────────┐
                    │    Nature of the Inquiry     │───────▶│   Immediate   │
                    │         Defined              │        │  Resolution   │
                    └─────────────────────────────┘        └───────────────┘
                                  │
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                    │     Assembly of Records     │
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Contract Documents	Construction Records	Maintenance Records
Plans	Inspection Reports	Periodic Maintenance Records
Specifications	Records of Materials Supplied	Condition Surveys
Change Orders	Quality Assurance Records	Weather Records
Interviews:	Engineering Reports	Citizen Complaints
Architect-Engineer	Completion Survey	Police Records
(Basis for construction	Weather Reports	Interviews:
standards)	Laboratory Reports	Users or Occupants
	Interviews:	Maintenance Personnel
	Contractor	Contractor Personnel
	Personnel	(re. repairs)
	Inspectors	
	A/E Representatives	
	Materials Suppliers	

Prepared by R. C. Mielenz
August 12, 1969

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Under the terms of its Congressional charter, the Academy is also called upon to act as an official—yet independent—adviser to the Federal Government in any matter of science and technology. This provision accounts for the close ties that have always existed between the Academy and the Government, although the Academy is not a governmental agency and its activities are not limited to those on behalf of the Government.

The NATIONAL ACADEMY OF ENGINEERING was established on December 5, 1964. On that date the Council of the National Academy of Sciences, under the authority of its Act of Incorporation, adopted Articles of Organization bringing the National Academy of Engineering into being, independent and autonomous in its organization and the election of its members, and closely coordinated with the National Academy of Sciences in its advisory activities. The two Academies join in the furtherance of science and engineering and share the responsibility of advising the Federal Government, upon request, on any subject of science or technology.

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The DIVISION OF ENGINEERING is one of the eight major Divisions into which the National Research Council is organized for the conduct of its work. Its membership includes representatives of the nation's leading technical societies as well as a number of members-at-large. Its Chairman is appointed by the Council of the Academy of Sciences upon nomination by the Council of the Academy of Engineering.

The HIGHWAY RESEARCH BOARD, an agency of the Division of Engineering, was established November 11, 1920, as a cooperative organization of the highway technologists of America operating under the auspices of the National Research Council and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of transportation. The purpose of the Board is to advance knowledge concerning the nature and performance of transportation systems, through the stimulation of research and dissemination of information derived therefrom.

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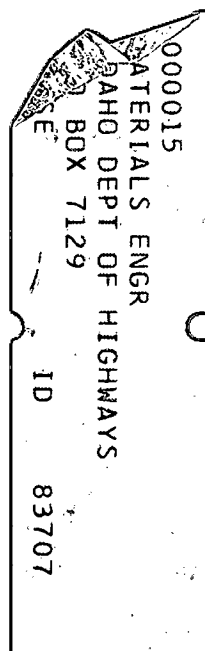
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