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REHABILITATION RESEARCH PROGRAM**

TECHNICAL REPORT REMR-CS-1

ENGINEERING CONDITION SURVEY OF CONCRETE IN SERVICE

by

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	Problem Area		Problem Area
CS	Concrete and Steel Structures	EM	Electrical and Mechanical
GT	Geotechnical	EI	Environmental Impacts
HY	Hydraulics	OM	Operations Management
CO	Coastal		

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COVER PHOTOS:

TOP — Concrete-lined ditch approximately 1 mile north of Delta-Mendota Canal, California. Concrete containing Type I cement was placed in 1957 and photographed in 1962.

BOTTOM — Typical deterioration in a concrete slab and support member due to freezing and thawing.

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Civil works structures must be continually evaluated for structural safety, stability, and operational adequacy. The overall objective of the investigation documented in the report is to develop information essential to the continued safety of concrete used in civil works structures such as flood control and multipurpose dams, navigation locks and dams, powerhouses and appurtenant structures, floodwalls, pumping stations, and similar structures. Specific objectives include (a) development and evaluation of

(Continued)

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20. ABSTRACT (Continued).

materials and techniques for repair and rehabilitation of civil works structures, (b) development of engineering guidance to evaluate and monitor safety of structures, and (c) development of design and construction methods for rehabilitating older structures to comply with current structural design criteria.

A condition survey of a civil works structure includes a comprehensive review of the design of the structure, construction techniques and materials, and operational and maintenance history. Information is obtained from available engineering data on the structure as well as on-site investigation. Data are analyzed and an evaluation report is written which includes conclusions and/or repair recommendations.

This report summarizes pertinent inspection procedures and methods of evaluation used by the Corps of Engineers in evaluating concrete civil works structures. Methods of evaluation include experience gained at the Waterways Experiment Station. Techniques are presented which have a potential for ascertaining the extent and cause of inadequacies in concrete structures.

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PREFACE

The study reported herein was authorized by Headquarters, U. S. Army Corps of Engineers (HQUSACE), under Civil Works Research Work Unit 31553, "Maintenance and Preservation of Civil Works Structures," for which Mr. James E. McDonald is Principal Investigator. Funds for the conduct of the study were provided through the Concrete Research Program, which is overseen by Mr. Fred A. Anderson, HQUSACE Technical Monitor, and through Program Related Engineering and Scientific Studies of the U. S. Bureau of Reclamation. Funds for publication of this report were provided through the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program, Mr. William F. McCleese, Program Manager.

The study was conducted at the U. S. Army Engineer Waterways Experiment Station (WES) during the period January 1982 to February 1983 under the general supervision of Mr. Bryant Mather, Chief, Structures Laboratory, and Mr. John M. Scanlon, Chief, Concrete Technology Division. This report was prepared by Messrs. Richard L. Stowe and Henry T. Thornton, Jr.

Commanders and Directors of WES during the conduct of the study and the preparation and publication of this report were COL Tilford C. Creel, CE, and COL Robert C. Lee, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
feet per second	0.3048	metres per second
foot-pounds (force)	1.355818	newton-metres
inches	0.0254	metres
miles (U. S. statute)	1.609347	kilometres
pounds (force) per square inch	6894.757	pascals
square feet	0.09290304	square metres

ENGINEERING CONDITION SURVEY OF CONCRETE IN SERVICE

PART I: INTRODUCTION

Background

1. In 1977, the U. S. Army Engineer Waterways Experiment Station (WES) initiated a major investigation of problems associated with maintenance and preservation of civil works structures. These structures include flood control and multipurpose dams, navigation locks and dams, powerhouses and appurtenant structures, floodwalls, pumping stations, and similar structures. The overall objective of this investigation is to develop information essential to the continued safety of civil works structures; specifically, (a) to develop and evaluate materials and techniques for repair and rehabilitation of civil works structures, (b) to develop engineering guidance for evaluating and monitoring the safety of structures, and (c) to develop design and construction methods for rehabilitating older structures to comply with current structural design criteria.

2. Engineer Regulation (ER) 1110-2-100 (Office, Chief of Engineers (OCE) 1983) requires that civil works structures whose failure or partial failure would endanger the lives of the public or cause substantial property damage be continually evaluated to ensure their structural safety and stability and their operational adequacy. During the periodic inspections associated with this continuing evaluation, evidence of concrete deterioration or distress is often observed which requires a critical examination for remedial action. Evaluation of older structures is also required to estimate the length of their future service life for replacement or rehabilitation planning purposes.

3. Basically, the success of any maintenance and repair measures depends upon two factors: first, the accuracy with which the cause and extent of the deterioration has been evaluated; and second, the quality of the judgment that has been used in selecting an appropriate maintenance or repair method. Once a specific conclusion as to the cause and extent of damage has been reached, then and only then can a rational selection be made among alternative maintenance and repair strategies. The necessity for accurate and efficient inspection and evaluation methods and equipment is obvious.

4. A condition survey of a concrete structure consists of a comprehensive review of the design, the construction techniques and materials, and the operational and maintenance history. It involves a meticulous examination of the performance and the condition of the structure and an analysis of any apparent weaknesses. A weakness may be some substantial inadequacy or failure of the structure. The investigators make a complete review of all available engineering data, conduct a detailed on-site investigation, analyze all the data, and write an evaluation report containing conclusions and repair recommendations as necessary; a laboratory examination may be included.

5. A condition survey is normally conducted in two phases. The Phase I objective is to ascertain the general condition of the structure and determine the need for any additional engineering studies and analysis. Problem and suspected deficiency identification is the main purpose of Phase I work. Phase II investigations are supplementary to Phase I and are performed when the results of the Phase I work indicate the necessity of additional investigations and analysis. If conducted, Phase II could include additional visual examinations, measurements, structure and foundation exploration and testing, materials testing, and stress and structural stability analysis as necessary to evaluate structural integrity.

6. It is not the intent of this report to provide detailed guidance for the evaluation of site environs, foundations, embankments, electrical and mechanical features, or hydraulics of major civil works structures. ER 1110-2-100 deals with these features of the structure. During a condition survey of the concrete, these site features are looked at for any evidence of existing or potential problems. Portions of this report contain items associated with these site features that can be checked during a condition survey of the concrete.

Objective

7. The objective of this study was to develop engineering guidance for conducting a condition survey of concrete in existing civil works structures. A condition survey is performed to detect causes of unsatisfactory performance that, if unremedied, may lead to failure or partial failure of the structure. The structural safety and stability of the structure are of prime concern.

Scope

8. This report summarizes pertinent inspection procedures and methods of evaluation presented in Corps of Engineers and other agencies' publications that are used to evaluate problems of concrete in service. Experience gained at WES in conducting condition surveys is also included. Inadequacies that are associated with concrete structures and foundations are described. Techniques are presented that have potential for use in ascertaining the extent and cause of the inadequacies in the concrete. Where techniques are presented, principles, advantages, limitations, and examples of application are discussed.

9. This report does not present rigid criteria or standards; rather it provides guidance. A structure must be evaluated in view of its peculiarities and site conditions and with full awareness of the many variables involved, some of which may not be accurately known.

Concrete Deterioration

10. Deterioration of concrete is defined (American Concrete Institute (ACI) Committee 201 1980) as "any adverse change of normal mechanical, physical, and chemical properties either on the surface or in the whole body of concrete generally through separation of its components." Deterioration of concrete can be caused by either physical or chemical factors or both.

11. Physical factors have to do with forces acting on the concrete, including those caused by temperature variations. Uplift, foundation displacement, and ice, seismic, or water forces could cause settlement or cracking of a concrete structure. Vibrations of structures caused by water surges, equipment operation, earthquakes, and barge or heavy equipment impact could cause damage to a concrete structure. Physical forces due to flowing water and ice, rocks, and various debris; cavitation; wind; and vehicular traffic, to name a few, cause erosion of concrete.

12. Absorptive aggregates may undergo freezing and thawing, causing excessive pressures which in turn cause concrete to crack and break up. Evidence of freezing and thawing in concrete is generally manifested by closely spaced parallel cracks. Infiltrating water in the cracks and subsequent freezing may cause further concrete deterioration. Thermal expansion and

contraction and wetting and drying may cause concrete disintegration.

13. Chemical factors are commonly associated with the intrusion of aggressive waters containing inorganic acids, sulfates, and certain other salts. Alkali-aggregate reaction, the reaction between the alkalies in portland cement and certain carbonate and siliceous rocks and minerals, in advanced stages, can cause physical damage to the concrete. Evidence of alkali-aggregate reaction includes cracking in random or regular patterns, expansion, gelatinous discharge, spalling, and chalky surfaces or rings around aggregates. A classic example of the damage caused by alkali-silica reaction can be seen at the William Bacon Oliver Lock and Spillway on the Warrior River near Tuscaloosa, Ala. (McDonald and Campbell 1977).

14. It often is difficult to ascertain if concrete has been or is being subjected to chemical attack. Evidence of chemical attack is generally a progressive disintegration of concrete. If the cause of the attack can be identified, it should be recorded on the sketches on which defects have been recorded. The extent of the attack area should also be described. Coring can be used to check the depth and lateral extent of damage.

15. Defective materials and inferior workmanship during construction can lead to deterioration and possible failure of a concrete structure. It is also possible that the original plans and specifications did not place sufficient emphasis on the effect which known or unknown loading conditions would have on the structure.

16. Mather (1975) warns of having tunnel vision while conducting a condition survey:

> Concrete constructions are rarely if ever brought to a state of advanced deterioration by one agent alone; thus it may be expected that foundations, structural design, loads, and imperfections of materials may all enter as important factors in the condition of one construction. Therefore if one studies the condition of a construction from only one point of view, he or she will overlook the other influences that have brought the construction to its present condition and will probably recommend inadequate remedial work because the parts played by different agents have not been appreciated.

Foundation Problems

17. Foundation defects may have been present prior to construction or

related to treatment during construction. Foundation distress may be evidenced by differential settlement, sliding, high piezometric pressures, seepage, scouring, rotation of piers, collapse of subterranean caverns, and cracks (even small ones) in concrete structures and embankments. Evidence of foundation problems is sometimes visible in alignment or grade changes of decking or roadway surfaces.

18. Foundations containing low shear strength seams such as clay or bentonite or shale may be vulnerable to sliding. Shear zones can cause problems at damsites and need to be inspected closely. Bedding plane zones in sedimentary rocks and foliation zones in metamorphics can be troublesome. Shales and schists, respectively, are prime suspects in such cases. It is extremely important to identify and evaluate the potentially hazardous interbeds on foliation; they are often deceptively thin and sometimes difficult to recover in drill core.

19. Seepage is a normal phenomenon in navigation structures and their foundations and carries its own threat to safety by causing chemical and physical alterations. Seepage water can cause severe problems where no problems exist at the time of construction. The presence of pervious seams, strata, or zones permitting seepage may result in failure by piping.

20. Jansen (1980) describes some problems that can be caused by seepage through foundations:

> Foundation seepage can cause internal erosion or solution. The removal of foundation material may leave collapsible voids and consequently precarious support for the dam. Such potential weaknesses sometimes can be identified by examining geologic conditions in the immediate vicinity of the reservoir. Actual deterioration may be evidenced by increased seepage, by sediment in seepage water, or an increase in soluble materials disclosed by chemical analysis.

Survey Team

21. It is imperative that an evaluation team with appropriate technical expertise be gathered to evaluate the problem and to ascertain the necessary corrective measures. Personnel must be practical and dedicated diagnosticians who thoroughly investigate each and every clue as it relates to the behavior of the structure. The evaluation team members must understand the modes and causes of failures in large concrete structures. The team should be comprised

of individuals with expertise in civil and mechanical engineering, engineering geology, and petrography. Preferably, they should have experience in the design, construction, and operational surveillance of large structures such as locks and dams. Depending upon the problems to be investigated, one to four people would compose a team.

Planning the Survey

22. Generally, the steps involved in a condition survey of a concrete structure follow a specific sequence. The Phase I work (paragraph 5) could involve all of the following steps with the exception of step g; the Phase II work could involve an extensive field and laboratory investigation:

- a. Meet with the client and define the problem (the office or site is appropriate).
- b. Determine the objective and scope.
- c. Select the evaluation team.
- d. Establish investigative procedures and techniques.
- e. Collect and review engineering and geological data and information.
- f. Conduct the field investigation.
- g. Conduct the laboratory investigation.
- h. Analyze findings.
- i. Make repair recommendations.
- j. Report findings.

23. The standard practice "Examination and Sampling of Hardened Concrete in Constructions," (American Society for Testing and Materials (ASTM) C823/CRD-C26)* presents some of the reasons why a condition survey is conducted:

> Investigations of the condition of concrete in service are usually undertaken for the following reasons: (a) to determine the ability of the concrete to perform satisfactorily under anticipated conditions of future service; (b) to identify the processes or materials causing distress or failure; (c) to discover conditions in the concrete that caused or contributed to satisfactory performance or to failure; (d) to establish methods for repair

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* Test methods are identified by (1) ASTM Book of Standards designations and (2) WES Concrete Handbook designations.

> or replacement without hazard of recurrence of the distress; (e) to determine conformance with construction specification requirements; (f) to develop data to aid in fixing financial and legal responsibility for cases involving failure or unsatisfactory service; and (g) to evaluate the performance of the components used in the concrete. It is assumed that the manager of the investigation will begin with one or more working hypotheses, derived from information received or gathered, that are intended to explain the reasons for the condition or conditions of the concrete, and that will be continuously revised and refined as more information is received. It is intended that at the end of the investigation, an explanation will have been produced which is the best obtainable from the investigation of the available evidence concerning the mechanisms that operated to produce the condition or conditions of the construction.
>
> The scope of an investigation of concrete in service may be limited to only isolated areas displaying deterioration. Or the investigation may be concerned with general distress, such as excessive deflection or collapse of structural members. It may involve study of the dislocation of entire structures or large portions of structures. The investigation may be confined chiefly to the study of the concrete, or it may require substantial research into other circumstances, such as foundation conditions, conditions of service, construction practices, and comparisons with other structures.

24. There are many conditions that might lead to failure in a hydraulic structure; therefore, great care must be taken not to overlook a potential weakness. Jansen (1980) proposed a checklist containing many questions investigators should ask in planning a survey. Some questions concerning the environs, foundation, embankments, electrical and mechanical features, hydraulics, and concrete are presented. The list is not considered complete. Jansen's checklist is presented below along with appropriate questions from other sources, indicated with an asterisk.

> Have changes occurred in the environs of the reservoir that may necessitate re-examination of the design or of the surveillance program (e.g., industrial activities such as deep excavation, trenching, tunneling, building construction, or storage of explosives or flammable materials)?
>
> Are the structural analyses of the dam satisfactory, or should new analyses be made using the latest design technology?
>
> Is there danger of spillway discharge undercutting the structure?
>
> Is adequate ventilation provided in shafts, tunnels, and galleries to prevent corrosion and to protect personnel from noxious gases?

Is essential machinery operable, especially such items as gates, valves, and hoists?

Is riprap, soil-cement, or other revetment intact as constructed?

Is all instrumentation in satisfactory working order?

Is there vegetation on embankments or abutments that might obscure adverse conditions from the inspector's view?

In the case of concrete dams, is there any reason to doubt the strength of the concrete? Has this been confirmed by nondestructive tests or tests of cores?

Are intake works for outlets and spillways free from silt and debris?

* Are upstream and downstream soundings current?

Have operating mechanisms that operate infrequently been checked or exercised to verify that they function properly?

* Has there been any mechanical damage to the structure such as barge damage to bridge or gated piers, or ice damage to gates, piers, or intake towers?

Are piezometer readings and water levels in wells reasonable, steady, and consistent with reservoir height?

Are reservoir linings, if any, performing as designed?

Are surveillance data receiving timely analyses?

Is leakage of water excessive? Is it increasing or decreasing? Is it clear or turbid? Are there large variations in individual drain discharges?

Are wet spots visible on the downstream face of the embankment or at abutment groins or immediately downstream?

Is there evidence of dissolution of foundation rock by seepage?

Is potentially dangerous seepage apparent in the vicinity from sources other than the reservoir, such as in the abutments at high level?

Are signs visible of any sloughing or slumping of embankments, abutments, or the reservoir environs?

At dams with concrete face slabs, is there visible warping or other distress?

Has cracking developed in structures, embankments, or foundations?

Are there any signs of erosion of the embankment or its foundation?

* Is there excessive deflection, displacement, or vibration of concrete structures; e.g., tilting or sliding of intake towers, lock walls, retaining walls, dam monoliths, or settlement of conduits?

* Are boils or piping evident in the area of any structure such as embankments, dam monoliths, lock walls, or cofferdams?

* Is seepage occurring through or under embankments or in abutments?

Has any recent seismic activity been recorded in the area? If so, are there any signs of detrimental effects on the reservoir or its environs?

Is subsidence evident at the site or in its margins? Are there any sinkholes?

Is there any progressive joint opening in the concrete?

Is there excessive erosion of concrete, such as in the spillway or stilling basin?

Is chemical deterioration of the concrete manifested, such as by leaching, crumbling, cracking, or spalling?

Do drain outlets show any adverse signs such as leaching of cement?

In frigid climates, has any damage occurred from ice thrust or freezing?

* Does water or ice surge cause vibration of the structure?

Have any structures been undermined or is scouring present?

Is there evidence of chemical alteration of foundation materials?

Are uplift pressures within the design assumptions, or is it necessary to drill more relief holes into the foundation?

Are deflection records adequate, and are deflections consistent with changes in reservoir level and temperature?

Have all adverse or questionable conditions been promptly reported?

Do operations and maintenance personnel examine the dam often enough?

Have deficiencies been remedied without delay?

* Is there an increase or decrease of flow from foundation drains, structural joints, or face drains of concrete dams?

* Is there any other evidence of distress or potential failure of the structure that could endanger life, property, or operation of the structure?

25. In reviewing pertinent engineering data and during the conduct of the survey, these and similar questions should be answered in as much detail as possible. A comprehensive picture of a structure is necessary to derive a meaningful understanding of the structure's safety.

Terms for Durability of Concrete

26. ACI Committee 201 (1980) report "Guide for Making a Condition Survey of Concrete in Service" contains standard definitions of 40 terms associated with the durability of concrete. This report also contains photographs of some of the defects that are defined. The report is reproduced as Appendix A.

Selected Literature

27. Selected literature relating to the condition of concrete in service is listed below as a ready reference. Some references (guidelines) deal with inspecting and evaluating concrete in service. Topics include causes of deterioration, periodic inspection, visual examination, sampling hardened concrete, detecting retrogression of concrete, national dam safety, etc. Selected references along with annotation are given below:

a. Liu, O'Neil, and McDonald (1978), "Maintenance and Preservation of Concrete Structures; Annotated Bibliography, 1927-1977."

   This report is a synopsis of 147 papers on methods of ascertaining the strength and condition of existing structures. The major emphasis is on nondestructive testing for determining in-place concrete quality; i.e., modulus, density, crack and void detection, and delaminated areas.

b. ACI Committee 201 (1980), "Guide for Making a Condition Survey of Concrete in Service."

   This guide provides a system for reporting on the condition of concrete in service. It includes a checklist of the many details to be considered in making a report and provides standard definitions of 40 terms associated with the durability of concrete. Its purpose is to establish a uniform system for evaluating the condition of concrete.

c. ACI Committee 201 (1977), "Guide to Durable Concrete."

   This guide discusses in some depth the more important causes of concrete deterioration, and gives recommendations on how to prevent such damage. Chapters are included on freezing and thawing, aggressive chemical exposure, abrasion, reactive aggregates, corrosion of embedded materials, repair methods, and the use of coatings to enhance durability.

d. ACI Committee 207 (1979), "Practices of Evaluation of Concrete in Existing Massive Structures for Service Conditions."

   The objectives of this standard are to present current methods available (a) for evaluating the capability of mass concrete to meet design criteria under service conditions, and (b) for detecting the retrogression in physical properties in concrete which would affect the capability of the concrete to meet design requirements in the future.

e. WES (1949), Handbook for Concrete and Cement.

   The practice referenced is ASTM C 823/CRD-C 26, "Examination and Sampling of Hardened Concrete in Constructions." This recommended practice outlines procedures for visual examination and sampling of hardened concrete in constructions. The sampling may provide materials for petrographic examination, chemical or physical analytical procedures, or any of a wide variety of destructive or nondestructive tests to determine physical, mechanical, or structural properties of the concrete.

f. OCE (1983), "Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures," ER 1110-2-100.

This regulation defines objectives, states policy, assigns responsibilities, and establishes procedures by which the Corps of Engineers carries out its responsibilities for ensuring the continuing structural adequacy of its major civil works structures in service.

g. OCE (1973), "Assembly of Data for Evaluation of Distress or Deterioration of Concrete Structures," Engineer Technical Letter 110-2-170.

This ETL provides recommended procedures to be followed for obtaining information for the evaluation of distress or deterioration of concrete structures.

h. OCE (1968), "Reporting of Evidence of Distress of Civil Works Projects," ER 1110-2-101.

This regulation prescribes responsibilities and procedures for immediate notification to higher authority of evidence of distress or potential failure of civil works projects. This applies to projects under construction or in operation.

i. Jansen (1980), *Dams and Public Safety.*

This book is a collection of lessons learned and practical methods that can be applied to the care and treatment of dams. It provides information on dam safety and evaluation, including the latest in technology and experience.

j. OCE (1979b), "National Program for Inspection of Non-Federal Dams," ER 1110-2-106.

This regulation describes a program, the major objective of which is to perform initial technical inspection and evaluation of non-Federal dams. The program is designed to identify conditions which constitute a danger to human life or property so that such hazardous conditions can be corrected by non-Federal interests.

PART II: REVIEW ENGINEERING DATA

28. Evaluation of concrete in existing structures must consider all aspects of design, construction, operation, and maintenance. For some structures, this assignment may be more challenging than the original design effort. Limited information may be available about the foundation, the construction materials, and the design. Assumptions concerning design, analysis, and factors of safety may not be recoverable; in some cases, they may not have existed. Exhaustive research, analysis, and divine inspiration may not reveal everything that should be known to make an accurate evaluation. However, new drilling, testing, and measurements could reduce the uncertainties and turn the unknowns into knowns.

29. Data sources that could yield engineering data relative to design, construction, operations, and maintenance are presented in the following paragraphs.

Foundation and Geologic Sources

30. Geologic information pertinent to evaluating the condition of a concrete structure may be obtained from:

a. Local and regional geologic data.
b. Aerial photographs of the site and vicinity.
c. Topographic maps.
d. Geologic maps, plans, and sections.
e. Geophysical data.
f. Exploration logs.
g. Drill cores.
h. Ground water levels and geohydrologic data.
i. Material test data (soil, rock, and water).
j. Foundation treatment data.
k. Mapping and details of discontinuities.
l. Seepage and drainage records.

Design

31. Engineering information and data relating to the project purpose,

the site, and the design of the structure may be obtained from:

a. Project description documents.

b. Design criteria including memoranda, normal loadings, hydrostatic loads, and static and dynamic analysis.

c. Construction contracts.

d. Plans and specifications.

e. Foundation and geologic data.

f. Climatological data.

g. Site surface data; control elevations.

h. Drainage and seepage records.

i. Environmental records of temperature, precipitation, humidity, sunny days, and exposure.

j. Seismological documents.

Construction

32. Engineering data relating to construction of a concrete structure may be obtained from:

a. Construction records, including photographs and inspection reports.

b. Instrumentation installation and observations.

c. Environmental records of temperature, precipitation, humidity, and sunny days.

d. Concrete materials data:*

(1) Strength and durability of concrete.

(2) Modulus of rupture and elasticity of concrete.

(3) Type of cement, cement sources, cement factor, admixtures, aggregate mix, and water-cement ratio.

(4) Lift height and method of placement.

(5) Treatment of contraction joints and lift surfaces.

(6) Actual time history of concrete placement and joint grouting.

(7) Heat generation characteristics of the concrete mixes.

(8) Physical, chemical, and mineralogical characteristics and sources of aggregates used; storage and processing methods.

(9) Water sources and analysis.

(10) Concrete batch plant operation.

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* Some items in this list are from Jansen (1980).

(11) Concrete quality control data.

(12) Method of transporting concrete, pumps, chutes, trucks, etc.

(13) Placement method, including vibrators.

(14) Curing methods.

(15) Hot and cold weather records.

Analytical Data

33. Documents from preconstruction and postconstruction work may contain analytical data that would be useful in the evaluation of existing concrete dams; sources could include the following taken fron Jansen (1980).

a. Records of stability and stress analyses.

b. Materials testing records.

c. Foundation studies.

d. Assumed loading conditions.

e. Assumed temperature variations.

f. Timing of grouting of construction joints in the construction sequence.

g. Extent of cooling that occurred prior to grouting.

h. Results of analysis of pressure distribution within the foundation.

i. Details of shear keys, if any, in contraction joints.

j. Results of abutment analyses.

k. Comparison of computed and measured stresses and deformations in dam and foundation.

Operations

34. Data sources for operation information that may be useful in evaluating a concrete structure may be obtained from records of:

a. High water, ice, earthquakes, or temperature extremes.

b. Boat or ship impacts.

c. Instrumentation measurements.

d. Equipment malfunction or failure.

e. Changes in operational procedures.*

f. Increased structural loads or loadings.

Maintenance

35. Maintenance records can be helpful in determining the condition and performance of a concrete structure. In addition, these records along with knowledge of the environmental condition can give an indication of the extended service life of the structure. Complete maintenance records will contain information concerning the following; items are taken from ACI 207.3R (ACI Committee 207 1979).

a. Location and extent of maintenance.

b. Type of maintenance.

c. Dates of repair.

d. Repair materials and technique.

e. Performance of repaired work.

Contacts for Data

36. Some of the engineering data required for review of the design, construction, maintenance, and operation of an old structure may be nonexistent or difficult to locate. The following sources may be helpful in locating needed engineering information; most of this list is taken from Redlinger (1974).

a. District offices.

b. Project administrative headquarters.

c. Owner or operator.

d. Former owner.

e. Design engineer.

f. State and local offices.

g. Contractor offices or personnel.

h. Retired project engineers.

i. Project operations personnel.

j. Newspapers and magazines.

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* As an example, for navigation or water storage facilities that have various means of regulating water discharge (tainter gates, sluiceways, etc.), it is important to know the specific location that is commonly used to pass water. Deterioration of the concrete and scouring of the foundation occur more often at these locations.

k. Well-drilling outfits.

l. Miners.

m. Tradesmen.

n. Suppliers.

o. Inspectors.

In-Service Reports

37. In-service reports to include routine and periodic inspection reports, condition survey reports, and instrumentation observation records are additional sources of information that can be helpful in evaluating concrete in service. The "National Program for Inspection of Non-Federal Dams" (ER 1110-2-106) has resulted in the creation of hundreds of safety inspection reports. These reports could be helpful in providing background information on a particular dam of interest. The existing or potential hazards of various dams to life or property are cited in these reports.

38. The data collected must be assembled and critically reviewed for any aspect of inadequate design, construction practices, and materials. Some of the following questions will come to mind in the review of the assembled data:

a. What was the site like initially?

b. What was done to change things during construction?

c. Was adequate information available on pre-existing foundation and geologic conditions?

d. What problems were foreseen and how did designers propose to take care of them?

e. What instrumentation was installed; was it monitored; how often?

f. What problems arose during construction; what was done about them?

g. What design modifications were made?

h. To what extent were foundations and abutments exposed?

i. Were there any significant problems during the life of the project?

j. Is maintenance cost high, low, or average?

k. What is the frequency of inspections, and of what kinds are they?

l. Have regular observations been made of the instrumentation records, and how were they analyzed?

PART III: FIELD INVESTIGATION

39. A preliminary field investigation, which is the Phase I work described in the Introduction, is conducted to verify the existence of a known or suspected deficiency or an inadequate performance. The defect or inadequate performance is examined to ascertain its nature and to estimate its extent and effect upon performance, service life, and safety of the structure.

40. The preliminary investigation may include, as appropriate, the following points:

- a. Assessment of deflection or settlement, or both.
- b. Visual examination of the exterior and interior of the concrete.
- c. Underwater inspection of the concrete and foundation.
- d. Core drilling.
- e. Assessment or installation of instrumentation.
- f. Petrographic examination and testing of selected samples of concrete or foundation material, or both.
- g. Examination of secondary chemical deposits in or on the concrete, or both.

The quality of the in-place concrete can be estimated using nondestructive testing procedures, such as impact devices, ultrasonic methods, and in some instances borehole devices such as cameras and televiewer loggers. Coring and sampling and laboratory testing are generally necessary to determine the relative quality of the concrete and foundation material.

41. During the conduct of the preliminary investigation, the site environs, foundation, embankment, electrical and mechanical and hydraulic features are scrutinized for evidence of existing or potential problems. The checklist present in Part I under "Planning the Survey" is helpful for evaluating these site features.

42. A detailed investigation, which is the Phase II work described in the Introduction, of concrete in service is performed when it is necessary to supplement the results of the preliminary investigation. The detailed investigation could include measurements of relative movement of structural elements or of the foundation, additional visual examinations of the concrete, structure and foundation exploration, materials testing, and stress and structural stability analysis.

43. During the field investigation there should be a systematic

approach to the visual examination. A walk-through of the entire project should be made; the environs, appurtenant structures, and the main structure are examined. The visual inspection in the atmosphere can be done with sketches, still photographic techniques, or video recordings. Inaccessible areas can be inspected with zoom lenses or powerful low-light high definition binoculars backed up by detailed photographs taken using flash guns (Browne, Doyle, and Papworth 1981). It is essential that any deficiency noted be located precisely and rapidly; this includes all categories of concrete as well as other affected elements at the project. This can be achieved by zoning and subzoning the particular element into inspection areas and locating the defects in the areas on a scale drawing or photograph. The methods for recording and presenting the mass of survey data will change from structure to structure depending on the complexity of the problem being examined.

Alignment of Structure

Embankments

44. An embankment should be examined for signs of cracking, sliding, sinkholes, erosion, seepage, animal burrows, or undesirable vegetative growth (Jansen 1980). Note any signs of settlement or horizontal movement, wet areas, springs, and boils. Displacement of embankments may be detected by misalignment of walls, guardrails, walkways, roadways, or appurtenant structures. During the examination of an embankment, special attention must be given to areas where concrete structures are within or abutting against the fill. These interfaces may be conducive to internal vibration.

Concrete

45. Misalignment of concrete elements of a structure can be detected by evidences of volume change, deflection, or dislocation. The closing or opening of joints, tilting, shearing, or shifting of hardware or machinery are evidences of misalignment. Deflection problems can often be caused by inadequate consideration of deflection during design (Allen 1977). This statement points up the fact that deflections may not be due to some excessive force developed at the site, but rather to an unanticipated force.

46. Movements of embankment structures or appurtenance structures can be evaluated using various instrumentation and monumentation. Common systems for checking alignment include direct alignment, trilateration, and

triangulation which involve a series of embedded plugs and stationary reference monuments. Precision surveying techniques and equipment are used to ascertain the movement of the plugs.

Exterior Condition of Concrete

Classification of surface deterioration

47. As the exterior concrete is being evaluated, it seems worthwhile to use a classification system for surface deterioration. A classification system would be useful in categorizing the concrete. When and if samples of concrete are taken, the different conditions of concrete could easily be considered using such a system. A number of systems are available that incorporate descriptive terms like "good, fair, poor concrete," "light, moderate, and severe deterioration," etc. Lehtinen (1979) uses a chart that appears reasonable and useful. It is based upon the degree of deterioration that is observed and the degrees are defined with descriptive terms and dimensions:

Classification of Surface Deterioration, after Lehtinen (1979)

Degree	Water Leakage	Leaching: Area (m^2)	Leaching: Thickness (mm)	Rusting Degree	Scaling (mm)
0	--				0-0.5
1	Moist	<0.5	<0.5	Minor	0.5-1.0
2					1-3
3	Water dripping	<1.0	<5.0	Moderate	3-10
4					10-50
5	Measurable	>1.0	>5.0	Severe	>50

This classification system could be used to delineate categories of concrete as they are zoned and subzoned on sketches, diagrams, or photographs of structural elements. After completing a survey, average values of defects can be expressed for the concrete structure. The classification system can serve as a data base for a particular structure. During subsequent condition surveys, the growth of deterioration can be elevated. If unchecked, deterioration of concrete will continue with time. When this or a similar classification system has been used in a region, comparisons between structures are possible.

Cracking survey

48. A crack is defined as "an incomplete separation into one or more parts with or without space between" (Jansen 1980). For purposes of this report, a structural crack is defined as an incomplete separation into one or more parts with or without space between and brings into question the structural integrity of an element. A structural crack should be examined in detail; ultrasonic pulse velocity, mapping, and coring should be used to ascertain the extent of a structural crack. Excessive stress or inadequate concrete strength may cause structural cracking. ACI Committee 201 (1980) describes a cracking survey as follows:

> A cracking survey is an examination of a concrete structure for the purpose of locating, marking, and identifying cracks, and of the relationship of the cracks with the other destructive phenomena. In most cases, cracking is the first symptom of concrete distress. Hence, a cracking survey is significant in evaluating the future serviceability of the structure. Some cracks may occur at an early age and may not be progressive; others may occur at later ages and increase in extent with time; and some may occur following some unusual event.

49. Cracks are classified by direction, width, and depth; adjectives used to denote direction of cracks are longitudinal, transverse, vertical, diagonal, and random. In addition to these terms, visual cracking terminology to include pattern cracking, surface checking, hairline cracking, and D-cracking is also used; see term definitions and photographs in Appendix A. A cracking survey will include descriptions of procedures and devices to be used in measuring the width and depths of cracks (ACI Committee 201 1980). Suggested width ranges are: fine - generally less than 1 mm; medium - between 1 and 2 mm; and wide - over 2 mm.

50. Conditions associated with cracking should be described. These conditions could include differential movement, spalling, seepage, deposits from leaching, etc. Sometimes cracks can be located by using a hammer, i.e., a hollow sound can indicate the presence of a crack. It is often helpful to note crack frequency in assessing the quality of in-place concrete.

51. Whenever feasible, the external expression of a crack should be correlated with internal cracks. External expressions of cracks may be traced by inspection of emptying and filling culverts, pipe, or electrical galleries or other openings in the structure. Coring, ultrasonic pulse velocity, and trace injection (dye water in an adjacent borehole, for example) can be used

to trace the extent of cracking. Stowe et al. (1980) reported the use of such techniques to trace cracks in a gate monolith of a high lift lock on the Illinois Waterway; Figure 1 illustrated the extent of cracking in the downstream river gate monolith. The extent of cracking will be discussed later under the section of this report dealing with the ultrasonic pulse velocity technique as a means of tracing internal cracking.

52. Where patches or overlays exist, crack surveys of structures are difficult to perform and are likely to be unreliable. Cracks beneath these repairs may represent an obvious or partial failure at greater depth.

Surface mapping

53. Surface mapping of defects can be accomplished using detailed drawings, photographs, or movie film mapping. These drawings should be permanent records and can be used in subsequent surveys as base data. Items most often identified and mapped are included in the following side heading, "Common surface defects." A list of items recommended for use in surface mapping has been published (ACI Committee 207 1979).

54. Surface mapping is begun at one end of the structure and continued in a systematic manner throughout the structure. A grid is sometimes used to overlay a section and then cracks and other defects are easily referenced. The importance of photographs of significant distress areas, with a scale or familiar object included, cannot be overemphasized for establishing trends and rates of deterioration. Profiles are recommended when they are feasible to obtain; they show the depth of missing concrete. Figure 2 presents examples of surface mapping using photographs and sketches (McDonald and Campbell 1977), and Figures 3 and 4 present a map of surface deterioration and a profile of lock wall erosion, respectively.

Common surface defects

55. The following surface features are common on concrete that has deteriorated. Defects can range from a single crack caused by concrete contraction to disintegration due to any cause, e.g., erosion or freezing and thawing. Defects include distortion, cracking, efflorescence, exudation, incrustation, pitting, popout, scaling, peeling, spalls, dummy areas, stalactites, stalagmites, dusting, and corrosion. Additional features include excessive wear due to erosion or cavitation, discoloration, evidences of cement-aggregate reactions, and chemical attack. See Appendix A for definition of most of these terms.

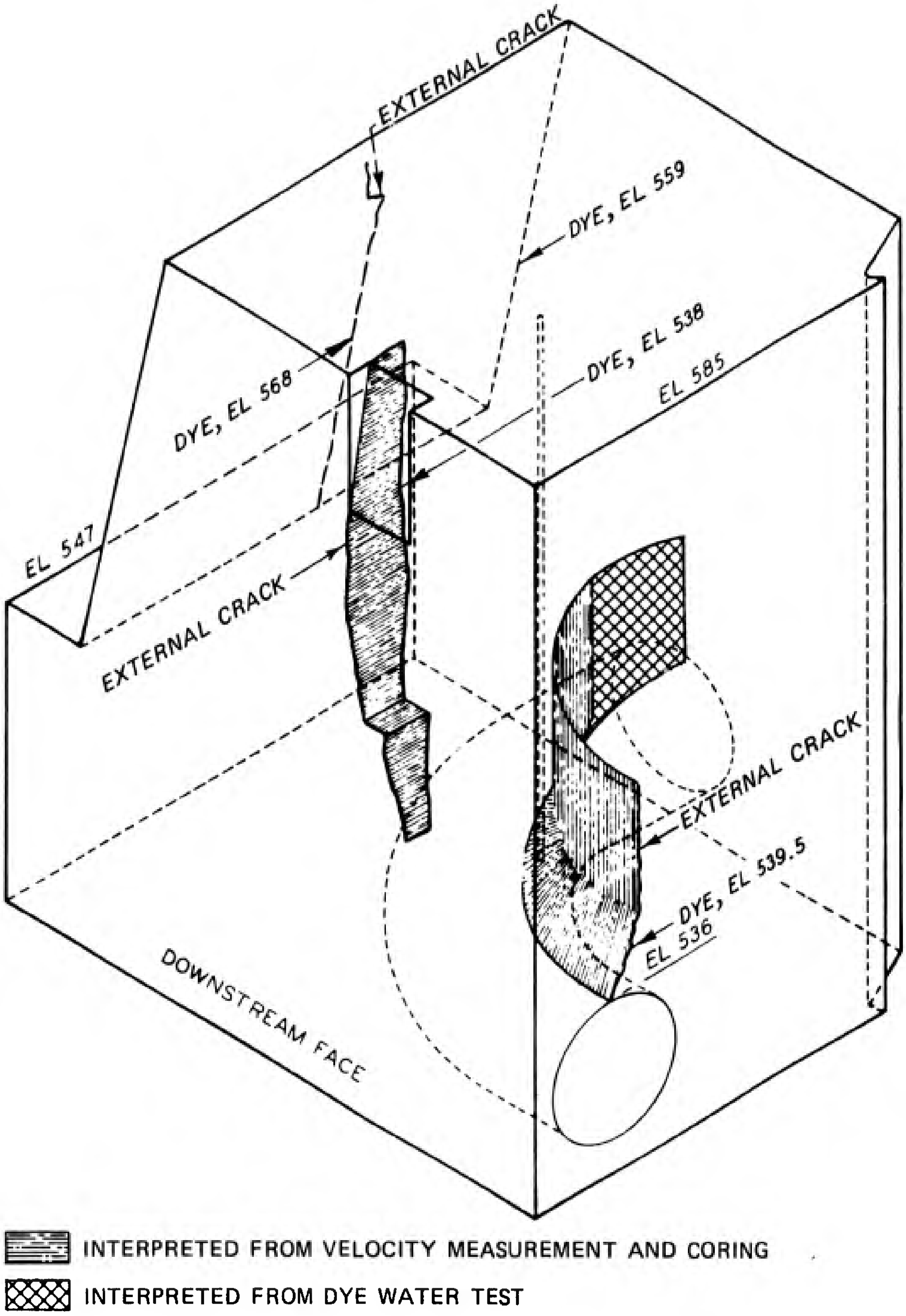
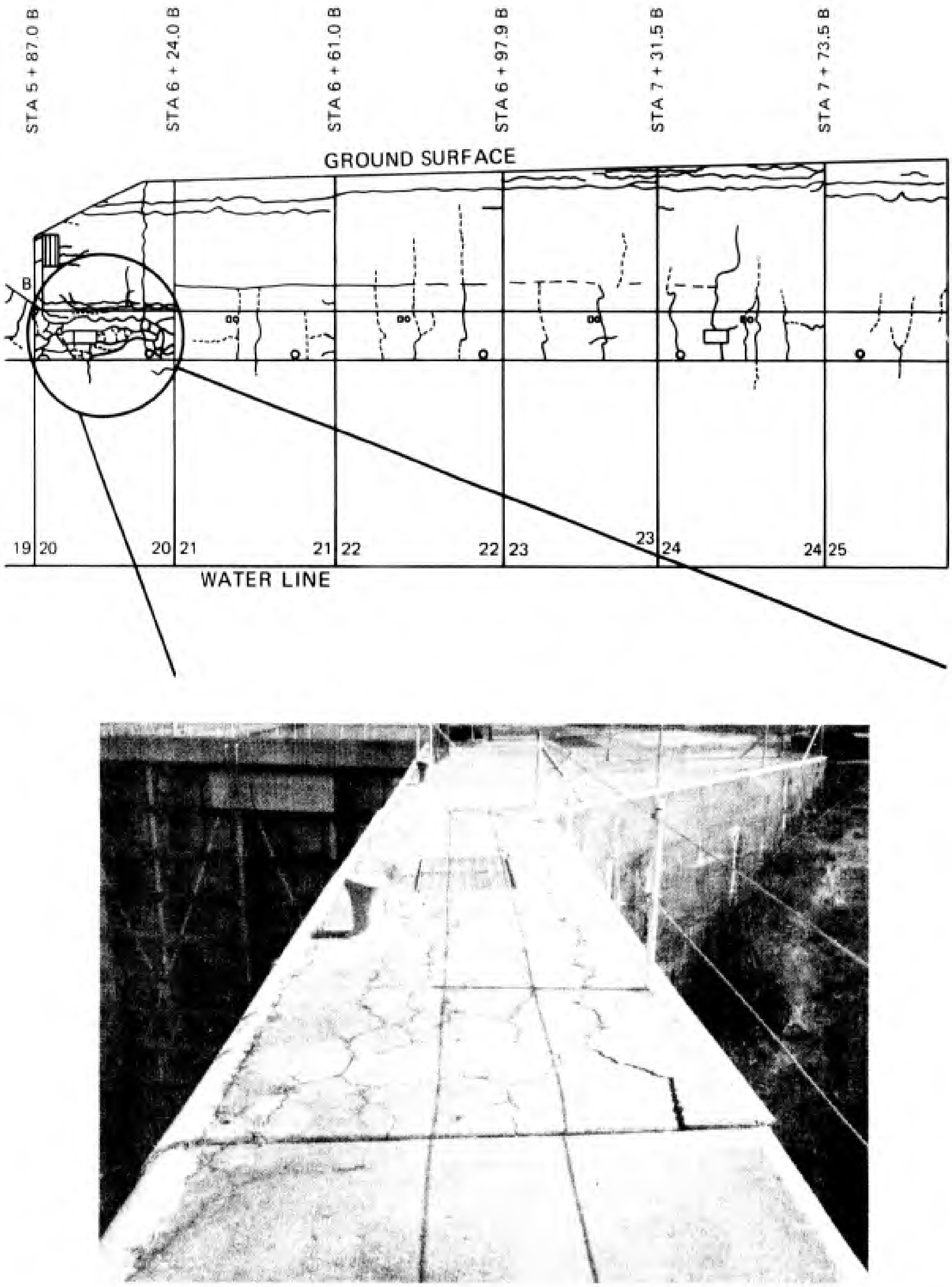


Figure 1. Extent of cracking, downstream gate monolith, river wall, Lockport Lock, Illinois Waterway



PHOTOGRAPH TOP OF GUIDE WALL
LOOKING UPSTREAM
MONOLITH NO. 20

Figure 2. Surface crack mapping, land wall, and lower end (after McDonald and Campbell 1977)

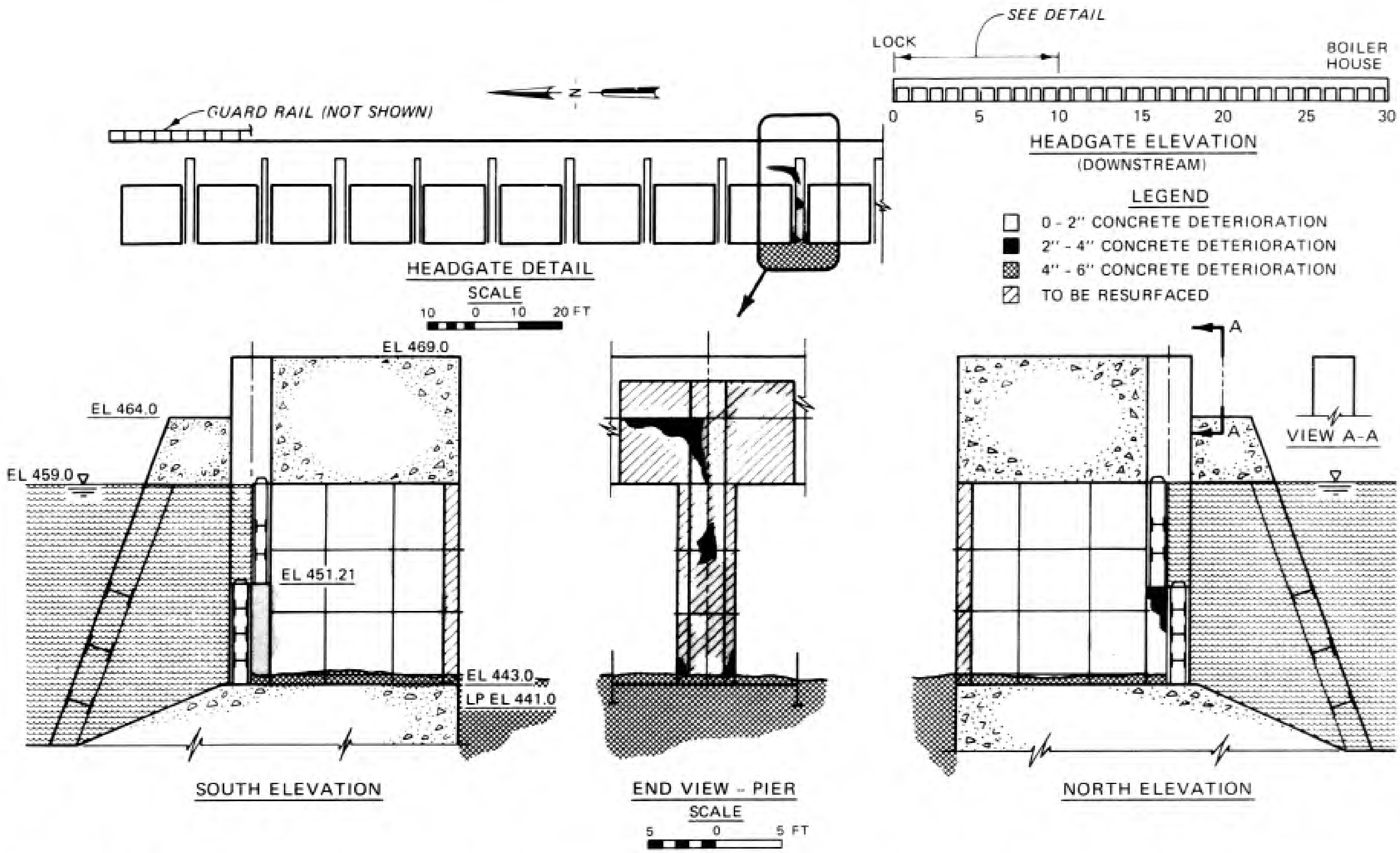


Figure 3. Mapping concrete surface deterioration (after U. S. Army Engineer District, Chicago 1977)

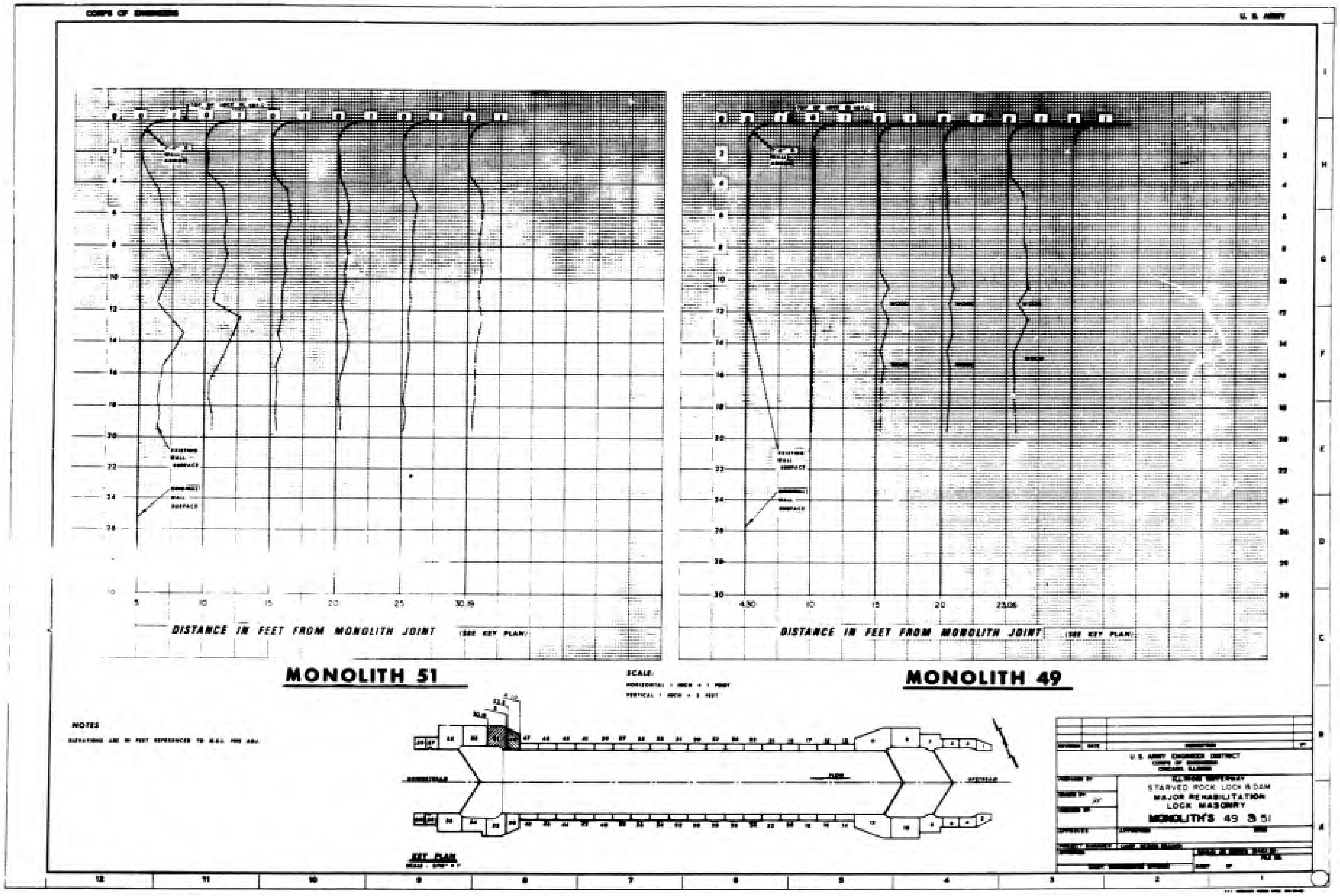


Figure 4. Lock wall profile indicating loss of concrete on lock wall (after U. S. Army Engineer District, Chicago 1977)

56. Common textural defects of concrete that are the result of construction practices include bleeding channels, sand streak, honeycombing, water pockets, stratification, sand pockets, evidence of segregation and bleeding, indications of high, low, or normal water content, and exposed reinforcement.

57. Any damaged concrete should be located on sketches along with the affected area and depth; sections and profiles are valuable in evaluating the extent and depth of damaged concrete. Possible causation or contributory factors of distressed concrete shall be noted. ACI Committee 207 (1979) describes some evidences associated with distress that need to be noted.

> Notation of evidence in the areas of damage commonly provide keys to diagnosing the cause. Such evidence may be loose, semi-detached fragments, D-cracking, rock and debris piles, offsets or protrusions, coloration, and overall condition of the damage area and of the surrounding concrete. These observations should be recorded and photographed.

58. The surface defects described thus far may occur on other than exposed surfaces. ACI Committee 207 (1979) report "Practice for Evaluation of Concrete in Existing Massive Structures for Service Conditions" includes the following procedure for covered surfaces.

> During routine inspections only exposed surfaces are generally surveyed. However, for periodic inspections or for special observations deemed necessary during routine inspections, surfaces flooded, under water, or backfilled and underground should be checked for surface damage by various methods. The method selected may depend on the size and depth of the area to be surveyed, conditions in the area, including water depth, and whether maintenance work will be done at the time of the inspection. Usual methods used include excavation, dewatering the structure, observation by submerged closed circuit television camera mini-submarine inspection, diver inspection, and sounding. Dewatering or excavation are usually the most expensive and, therefore, are generally done only when there is concern about safety of the structure.

59. Operational difficulties need to be evaluated as they may indicate distress. A jammed gate or valve may be caused by movement of a structural element or deteriorated concrete that has expanded and made the gate or valve inoperative.

Stains

60. Most stains on concrete surfaces cause an unpleasant appearance as

opposed to causing damage to the concrete. Natural causes such as runoff water that deposits soot or metallic salts account for some staining of concrete. Construction or maintenance accidents cause staining, e.g., paint, creosote, asphalt, lubricating oil, grease, and iron stains are common. The type and extent of stain should be ascertained in the field; if the stain cannot be identified and it is desirable to remove it, take a sample of the concrete for laboratory examination. It is important that the stain be identified so that appropriate methods of removing it can be recommended.

Seepage

61. "Practices for Evaluation of Concrete in Existing Massive Structures for Service Conditions" (ACI Committee 207 1979) includes the following information on seepage.

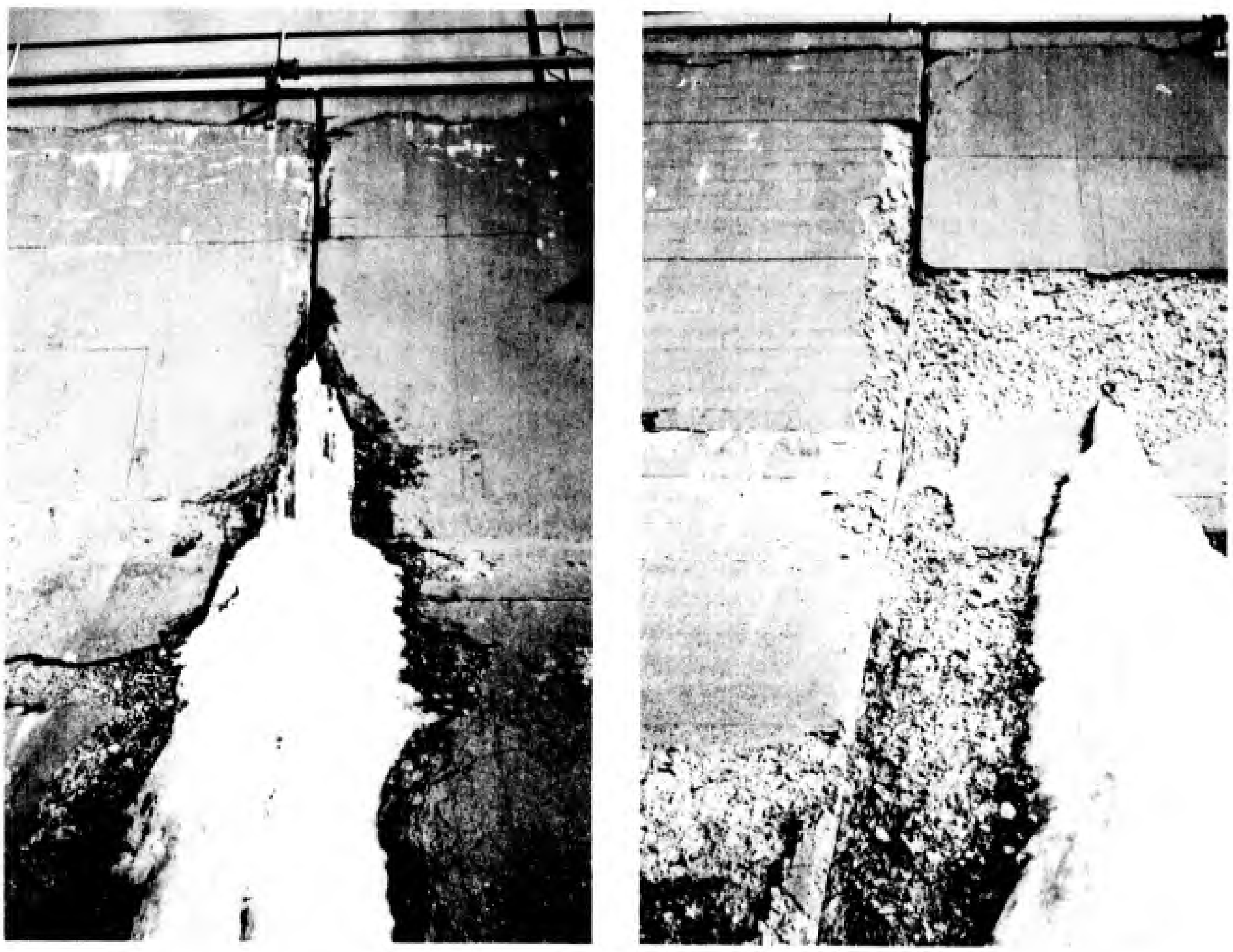
> Seepage is the movement of water or other fluids through pores or interstices. Some structures may include design features to safely control seepage such as waterstops, sealed joints, cutoff walls, grout curtains, granular drains and drainage galleries. These features should be checked to assure they are functioning as designed. Seepage can be important with respect to durability, can indicate failure of the structure to function monolithically and may also indicate operating problems in water retention structures. Seepage occasionally occurs through unbonded horizontal or vertical construction joints; around waterstops or sealants in expansion, contraction or control joints; along cracks; along the interface between concrete and some other material such as foundation contacts, form bolt or tie holes, or other embedded items; or through areas of porous low quality concrete.
>
> Water from seepage may result in the development of excessive hydrostatic heads on portions of the structure, may attack the concrete chemically, provide excess moisture to produce mechanical failure during freeze-thaw cycles, or may transport undesirable particles from the concrete or foundations. Analysis of seepage water can be used to evaluate chemical activity. The appearance of seepage water, whether clear or cloudy, will indicate the presence of transported sediments or dissolved minerals. Determination should also be made of the extent and the quantity of seepage water if measurable.
>
> Frequently, it is important to know the source and velocity of seepage. The source can sometimes be obtained by simple measurements comparing the temperature of seepage with groundwater or reservoir temperatures. Dye tests can be made utilizing commercial dyes such as Rhodamine B (red) or Fluoroscein (green) both of which are acceptable

by the DEQ (Department of Environmental Quality). The dye is introduced into water at some location near the upstream face, in drill holes, or other appropriate accessible points. The location and time of reappearance will indicate the source of various seeps and will provide the velocity of dye movement.

62. Seepage occurred through vertical construction joints in the river lock wall at Lockport Lock on the Illinois Waterway; see Figure 5 (Stowe et al. 1980). The photograph was taken during the wintertime; thus, the ice formation over the vertical construction joint (Figure 5a). Figure 5b shows ice formed as water exited from a form tie hole.

Joint condition

63. Joints in concrete structures should be checked to assure they are



a. Ice over vertical construction joint

b. Ice formed from water coming out of form tie hole

Figure 5. Seepage of water, Lockport Lock, Illinois Waterway

in good condition and functioning as intended (ACI Committee 207 1979). Expansion, contraction, and construction joints should be located, described, and their existing condition noted. Opened or displaced joints (surface offsets) should be checked for movement if appropriate; various loading conditions should be considered when measurements of joints are taken. All joints should be checked for defects such as spalling or D-cracking, chemical attack, evidence of seepage, or emission of solids. Condition of joint filler, if present, should also be examined. If joint construction drawings are not available, they should be drawn up and the joint survey information recorded on them kept as base data.

Additional causes and
evidences of deterioration

64. Some of the common causes of concrete deterioration and evidence of the same are briefly described; i.e., drying shrinkage, thermal stresses, freezing and thawing, sulfate attack, acid attack, alkali-silica reaction, alkali-carbonate reaction, and abrasion and cavitation.

65. Drying shrinkage. When concrete is subjected to wetting and drying cycles, it expands and contracts or shrinks, respectively. Volume change results from the wetting and drying and develops tensile stresses within the concrete; when the tensile strength of the concrete is exceeded, the concrete will crack. Drying shrinkage cracks are generally fine and show no evidence of movement; they are usually shallow but the linear distance can be several feet.

66. Thermal stresses. One problem in large concrete sections is the probability of high tensile stresses or strains resulting from heat generated by the hydration of cement with subsequent differential cooling. If the thermally induced tensile strain is greater than the concrete tensile strain capacity, the concrete will crack (Liu 1981). The evidence of cracking due to thermal stresses is typically orthogonal or blocky, similar to that for drying shrinkage; however, thermal cracks are generally much deeper. Figure 6 illustrates thermal cracking in the replacement concrete for a lock wall resurfacing project.

67. Freezing and thawing. "As the temperature of saturated concrete is lowered, the freezable water held in the capillary pores in the cement paste and aggregates freezes, and expansion of the concrete takes place. If subsequent thawing is followed by refreezing, further expansion takes place, so

Figure 6. Thermal cracking in lock wall resurfacing concrete

that repeated cycles of freezing and thawing have a cumulative effect." (Liu 1981).

68. "One common indication of freezing and thawing deterioration is the appearance of cracks which run approximately parallel to joints or edges of concrete surfaces. As deterioration progresses, these parallel cracks occur farther away from the joint. This type of cracking has been designated as D-cracking. As cracking spreads from a joint, disintegration starts near the joint and eventually the affected concrete spalls." (McDonald 1981). Pattern cracking is also associated with freezing and thawing.

69. "Scaling of the near-surface portion of hardened concrete or mortar is another indication of freezing and thawing deterioration. Concrete mortar of poor quality will crumble away with repeated freezing and thawing cycles, gradually exposing the coarse aggregate particles." (McDonald 1981). Figure 7 illustrates evidence of freezing and thawing (spalling and cracking).

70. Sulfate attack. "Chemical or physical reaction or both between sulfates usually in soil or ground water and concrete or mortar, primarily with calcium aluminate hydrates in the cement-paste matrix, often causes deterioration." (Bellport 1968). The reaction between the sulfates and the concrete results in an increase in solid volume. This expansion is blamed for the resulting disintegration of the concrete. Concrete attacked by sulfate generally is light in color and falls apart easily when struck with a hammer. The affected concrete can undergo color changes, and have stains, crazing, cracking, spalling, and scaling associated with it. Figure 8 illustrates

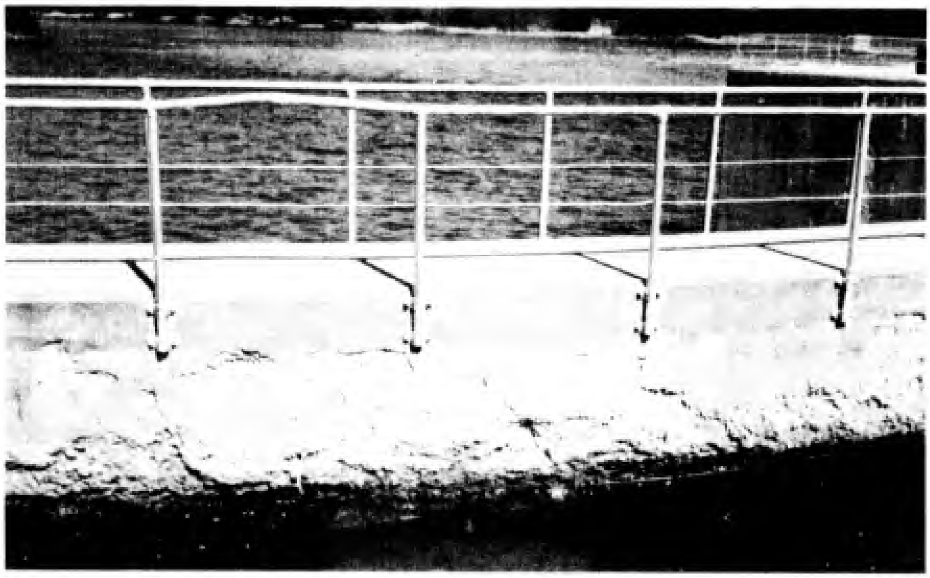


Figure 7. Evidence of freezing and thawing

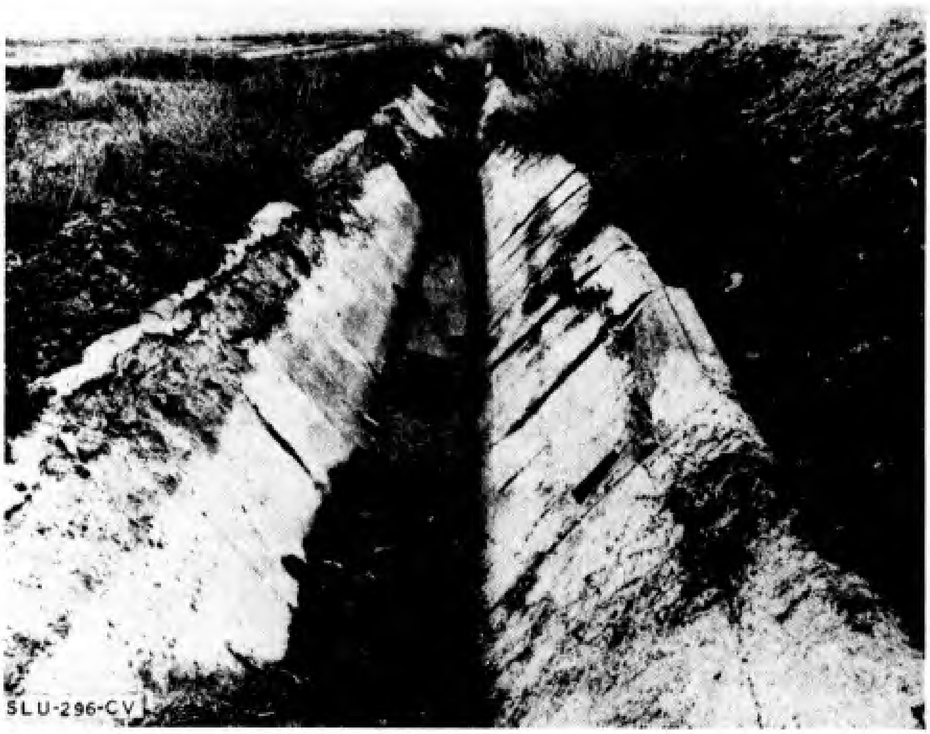


Figure 8. Concrete-lined ditch approximately 1 mile north of Delta-Mendota Canal, California. Concrete containing Type I cement was placed in 1957 and photographed in 1962 (after Bellport 1968)

damage to a concrete-lined irrigation ditch exposed to sulfate conditions (Bellport 1968).

71. Acid attack. "The deterioration of concrete by acids is primarily the result of a reaction between these chemicals and the calcium hydroxide of the hydrated portland cement. Where limestone and dolomitic aggregates are used, they are also subject to attack by acids. In most cases the chemical reaction results in the formation of water-soluble compounds which are then leached away by the aqueous solutions." (Liu 1981). The results of acid attack on concrete can include color changes, efflorescence, cracking, spalling, and disintegration; see Figure 9. Expansion of the structure can sometimes be seen as a swelling of concrete elements. Acid water will affect concrete and may cause corrosion of reinforcement exposed by open cracks. Deposits on the concrete can be associated with acid attack. "Strongly acid water will destroy any portland-cement concrete, grout or mortar" (McDonald 1981).

72. Thornton (1978) states that concrete constituents are susceptible to attack and decomposition by acid. Certain genera of anerobic bacteria produce acids which attack the concrete. In some cases, structural concrete can turn to "mush" due to this process. Thornton (1978) describes such a problem at the Piedmont and Clendening Lakes in southeastern Ohio. He concludes "...that the deterioration is due to acid attack and is the final stage of a corrosive process caused by sulfur bacteria action."

73. Alkali-aggregate reaction. Alkali-carbonate rock reaction and alkali-silica reaction are the two common types of reaction that result in damage to concrete. The definition of these two reactions is taken from ACI Committee 116R (1978).

> Alkali-carbonate rock reaction - The reaction between the alkalies (sodium and potassium) in portland cement and certain carbonate rocks, particularly calcitic dolomite and dolomitic limestones, present in some aggregates; the products of the reaction may cause abnormal expansion and cracking of concrete in service.
>
> Alkali-silica reaction - The reaction between the alkalies (sodium and potassium) in portland cement and certain siliceous rocks or minerals, such as opaline chert and acidic volcanic glass, present in some aggregates; the products of the reaction may cause abnormal expansion and cracking of concrete in service.

74. In surface and near surface concrete that is affected by alkali reaction, there will often be evidence of efflorescence, exudation, or

Figure 9. Concrete deterioration caused by acid attack (after Thornton 1978)

incrustation. "The affected concrete is characterized by a network of pattern or map cracks usually most strongly developed in areas of the structure where the concrete has a constantly renewable supply of moisture, such as close to the waterline in piers, from the ground behind retaining walls, beneath road or sidewalk slabs, or by wick action in posts of columns. Additional signs of the severity of the reaction are close expansion joints with possible crushing of the adjacent concrete." (Liu 1981).

75. A distinguishing feature of alkali-carbonate rock reactions is the general absence of silica gel exudations at cracks; see Figure 10. Evidence

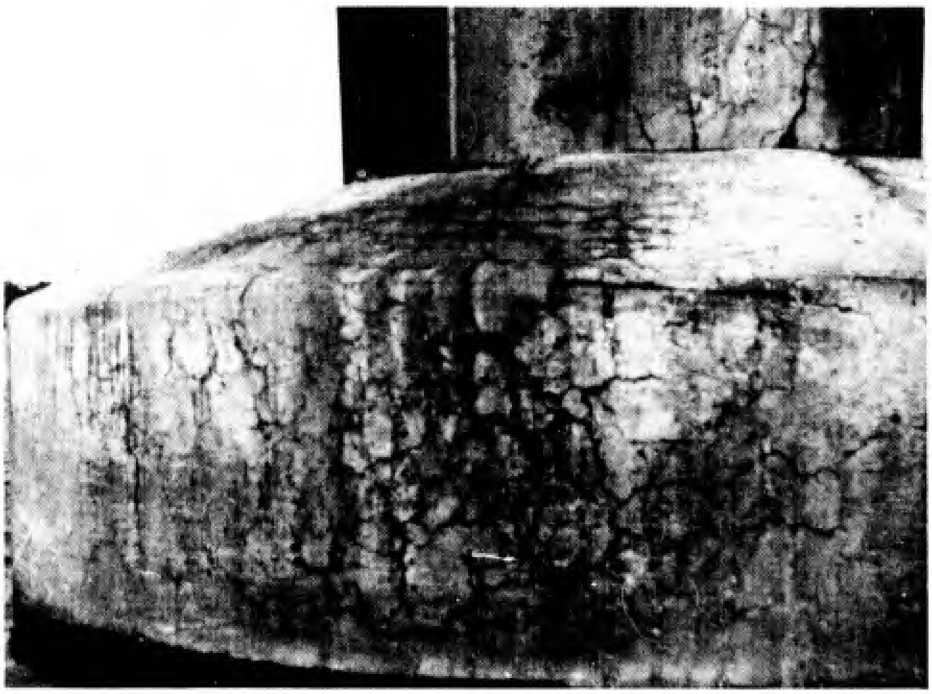


Figure 10. Expansive cracking in pier pedestal due to alkali-carbonate rock reaction

of alkali-silica reaction is the presence of a liquid or viscous gel-like material at a pore, crack, or opening in the concrete; see Figure 11. Sometimes white reaction product rings are present around aggregate particles or the reaction product replaces the entire aggregate; a classic example of alkali-silica reaction is presented in Figure 12.

76. Jansen (1980) describes some problems caused by alkali-silica reaction:

> Expansion in the decomposing concrete can be substantial. Total upstream deflection of the arch crown at one 61-meter (200-foot) high dam in California was about 127 millimeters (5 inches) in the first 10 years after completion

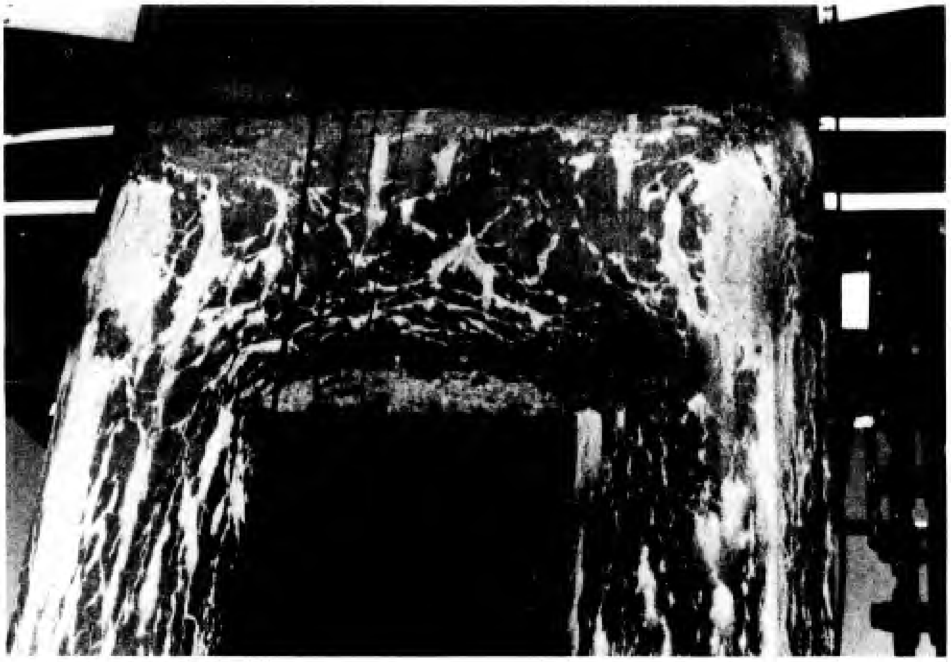


Figure 11. Service bridge support columns, Lock and Dam No. 24, Mississippi River; alkali-silica reaction product present at cracks

> of construction. Rates of movement usually appear to decrease as the dam increases in age.
>
> Alkali-aggregate reaction sometimes causes the disbonding of blocks at lift surfaces. Loss of strength by disbonding, and the accompanying increase in hydrostatic pressure along the lift surfaces, will reduce resistance to sliding and overturning. Alkali-aggregate reaction can cause expansion of a concrete dam with consequent cracking and deterioration, and possible binding of gates, valves, and metalwork. Once alkali-aggregate reactivity has developed in a relatively thin concrete dam, it cannot be stopped practically by any means now known. Where deterioration has progressed to a dangerously advanced stage, the effective remedies are to remove and replace the defective concrete or to build a new dam to replace the old one.

77. "An opening up of the interior structure of the concrete by crack formations caused by alkali-aggregate reaction could prepare the way for frost attack whereby the rate of disintegration would be increased. Dynamic loads, such as vibration and impact, create tensile stresses in the concrete and contribute to the formation and extension of cracks." (Idorn and Nepper-Christensen 1970). Vibrations are thought by some to cause initial microfracturing in sound concrete. This type of cracking could occur at any structure

Figure 12. Photograph shows evidence of alkali-silica reaction, i. e. gel pockets and cracks traced in reaction products. Thin section blank, core 5-1, 7.5- to 9.0-ft section, Tuscaloosa Lock (after McDonald and Campbell 1977)

subjected to dynamic loading. Once the concrete is cracked, any chemical attack can occur, allowing frost attack to occur in some cases.

78. Erosion. In the following excerpt, Liu (1981) describes concrete erosion caused by abrasion and cavitation. See Figures 13 and 14, respectively, for examples of concrete erosion due to abrasion and cavitation.

> Abrasion-erosion damage may result from the abrasive effects of waterborne gravel, rocks, and other debris being circulated over a concrete surface, which is distinguished from the holes and pits formed by cavitation-erosion. Spillway aprons and stilling basins are particularly susceptible to abrasion.
>
> Cavitation is the formation and subsequent collapse of vapor bubbles in a liquid stream. High velocity flow into areas of low pressure due to sudden directional changes produce such vapor cavities. When the cavities reach a high pressure zone, they condense suddenly with an almost instantaneous reduction in volume. This collapse, or implosion, originates a shock wave, similar to a water hammer, which upon reaching an adjacent surface induces very high stresses concentrated in a small area. Repeated collapse of vapor bubbles on or near the surface of the concrete will cause pitting. In general, damage from cavitation is not common in open conduits at water velocities below 40 ft per sec.* At higher velocities the forces of cavitation are sufficient to erode away large quantities of high-quality concrete and to penetrate through thick steel plates in a comparatively short time. Concrete spillways and outlet works of many high dams have been severely damaged by forces of cavitation.

Corrosion of metal

79. With adequate construction design and practice, embedded metal in concrete is sufficiently protected by the concrete. However, when the concrete deteriorates, the ingress of moisture can cause corrosion of the reinforcement steel. Corrosion of the steel results in a volume increase, due to the oxide produced, and the concrete cover will be cracked and spalled. Cracks due to corrosion of reinforcement steel generally run in straight, parallel lines at uniform intervals corresponding to the reinforcement spacing, and usually show evidence of rust staining. Some evidence of corrosion is given in the following paragraphs.

80. In any efforts to detect corrosion of metals, examination of the

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* A table of factors for converting non-SI to SI (metric) units of measurement is presented on page 4.

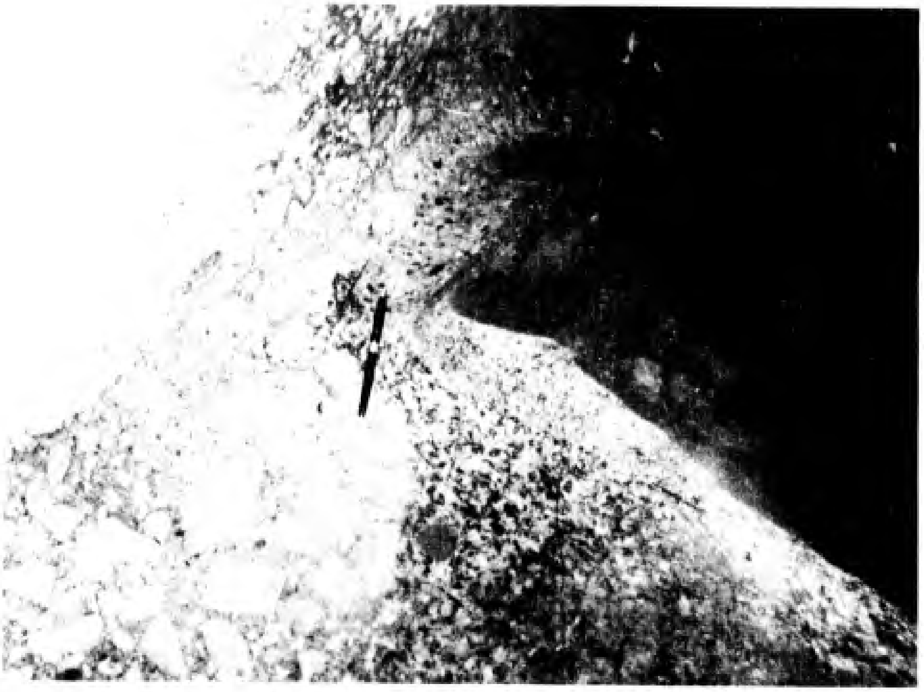

Figure 13. Photograph illustrating abrasion of concrete, characteristic of smooth worn surface

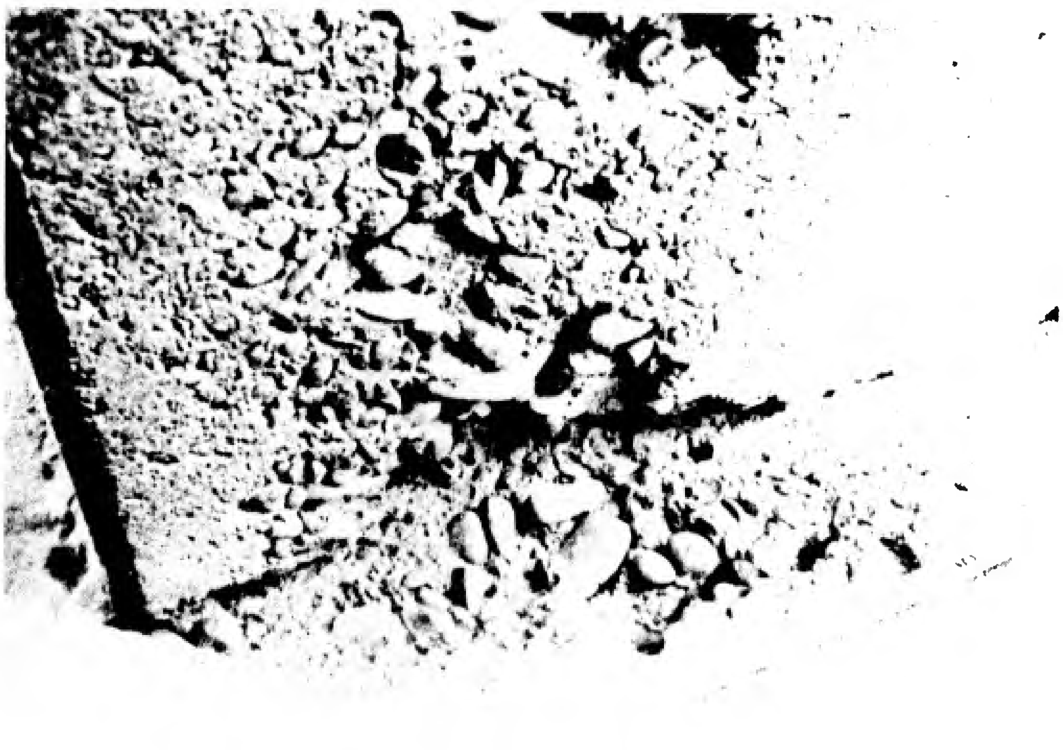

Figure 14. Photograph illustrating cavitation of concrete; note characteristic pitted surface

metal surface for corrosion products is of primary importance. The greatest majority of metal used in construction today is steel, and in this case, the obvious corrosion product to look for is rust. Rust on steel can range anywhere from a black-colored coating which is tightly adhered to the base metal to a large, flaky, reddish-orange substance that can be removed from the base metal with very little effort. These corrosion conditions are obvious and their severity can be evaluated by sampling the material that can be removed from the surface. In looking for evidence of corrosion to metal structures or structural elements, it is a good practice to examine those points where the metal comes in contact with any liquid or any other metal. Water is the most obvious liquid, but even some mild alkalies and acids can cause corrosion. Where water runs down the surface of a metal, there is likely to be some form of corrosion present. It may not always be readily observable and sometimes it will be remotely hidden. In any place where water or other liquids can be trapped against a metal surface, such as in the soil surrounding buried metal, or in dirt that has become caked on the surface of the metal, corrosion is likely to be hidden and not discovered. Where one metal comes in contact with another metal of different composition, as at connections, galvanic corrosion cells can form which cause sacrificial corrosion to one metal to protect the other. If undetected, this condition could lead to the failure of one of the metals.

81. Corrosion does not always manifest itself as a coating deposited on a base metal, such as in the case of rusting. Pitting corrosion causes small cavities or craters to form in the surface of the metal. If this type of corrosion occurs in moving water, for example, the corrosion products formed in conjunction with the pit can be eroded away leaving a clean bare metal surface that has the appearance of being pockmarked. Changes in surface texture of a metal can be indicative of a type of corrosion that removes one metal from an alloy without disturbing the others, leaving a weak residue at the surface. This can be detected through observation and scraping of the surface of the metal.

82. One form of corrosion to metals cannot be detected by observation alone. Stress corrosion is a condition where otherwise ductile metals can become brittle under the influences of a corrosive environment and the application of stress to the metal. This condition is not identified by surface changes such as corrosion products, and the end result can be rapid, brittle

failure of the metal without warning. Different corrosive environments affect different types of metals. If stress and corrosive atmospheres are present, a check of the type of metals affected by the particular environment present will tell if the metal is susceptible to this form of corrosion, and further tests can be made where necessary.

Previous repairs

83. Repairs to older concrete and masonry structures are not uncommon. The survey team needs to describe any repair in terms of its ability to perform the function for which it was placed. Repairs can include a patch over a form-tie hole or the overlay of the vertical surface of a lock wall. The quality of the patch material (concrete, mortar, epoxy, etc.) should be ascertained; note if the repair is cracked, spalled, etc. If cracked, could the cracking be caused by reflective cracking from the material beneath the repair, or in the repair material? The quality of the bond between the old and repair material should be described. Terms should be used to describe the repair: good (serving intended function); fair (marginal benefit, will last so many more years); poor (has deteriorated, no longer serves intended function); see Figures 15 and 16 for examples of poor repairs.

Mechanical nondestructive techniques

84. Information on the quality of in-place concrete can be obtained in a number of ways. Nondestructive techniques can be used without destroying or removing concrete. In this section, the two mechanical techniques that are commonly used will be discussed, i.e., the Schmidt hammer and the Windsor probe. Techniques incorporating electrical signals will be discussed later. Malhotra (1976) presents words of interest about these two test techniques.

> Weil... and the RILEM* Working Group on Nondestructive Testing of Concrete... have pointed out the need for extreme care in the use of these tests. Frequent calibration and checking of the test hammers are most desirable. The type of cement appears to affect the test results; for example, concrete made with high-alumina cement has given different results from concrete made with portland cement... It has also been reported by the RILEM Working Group... that in one instance fire-damaged concrete has given a higher estimated strength than comparable undamaged concrete. To correctly interpret the test data, it is desirable to know the mix

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* Réunion International des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions.

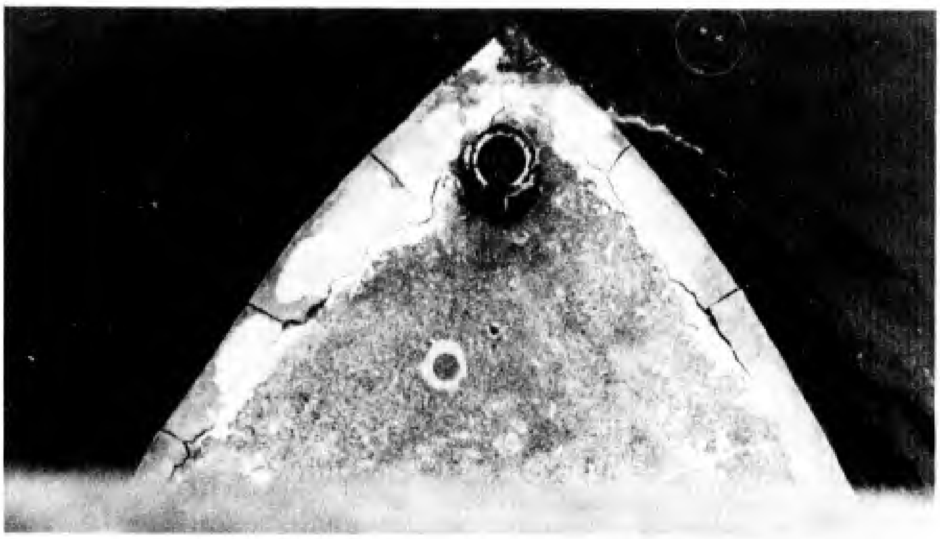


Figure 15. Poor repair; epoxy mortar repair to concrete pier which has failed, Lock and Dam No. 24, Mississippi River

Figure 16. Poor repair; cement mortar repair with an overlay of epoxy. Overlay has failed and exudation coming from beneath the cement mortar repair. Lock and Dam No. 24, Mississippi River

proportions, type of coarse aggregate used, age, and moisture conditions of concrete under test. Weil[...] has recommended the removal of soft mortar layers from the surface of concrete before using the impact hammers.

Studies carried out by Weil[...] and the RILEM Working Group[...] indicate that the strength of concrete under investigation can be predicted with an accuracy of 20-30 percent by the use of test hammers.

85. Rebound hammer. The rebound hammer, also called the Swiss or Schmidt hammer (see Figure 17), is used to assess the uniformity of concrete in place and to delineate zones or areas of poor quality concrete.

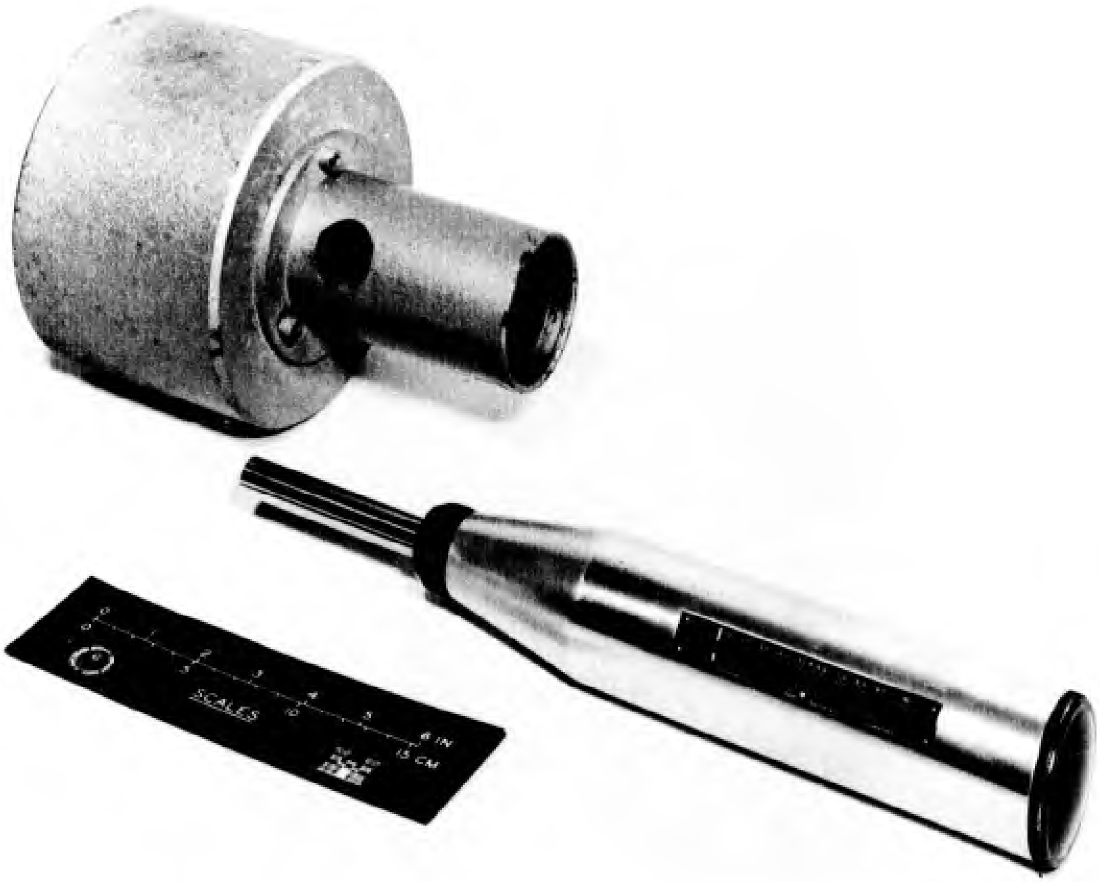


Figure 17. Schmidt hammer and calibration anvil

86. The rebound hammer contains a spring-loaded steel hammer which when released strikes a steel plunger in contact with the concrete (ASTM C805-79/ CRD-C 22-80). The amount of rebound of the plunger is measured on a linear scale attached to the instrument; in effect, the plunger rebounds depending upon the resiliency of the material struck.

87. Some advantages of the rebound hammer are that it is easily portable, easy to use, low in cost per test, and can be used quickly to cover a

large surface area. The hammer is valuable as a purely qualitative tool.

88. Malhotra (1976) summarizes the limitations of the Schmidt hammer.

> The limitations of the Schmidt hammer are many; these should be recognized and allowances be made when using the hammer. It cannot be overstressed that this instrument must not be regarded as a substitute for standard compression tests but as a method for determining the uniformity of concrete in the structures and comparing one concrete against another. Estimation of strength of concrete by the Schmidt hammer within an accuracy of ±15 to ±20 percent may be possible only for specimens cast, cured, and tested under identical conditions as those from which the calibration curves are established. The prediction of strength of structural concrete by using calibration charts based on the laboratory is not recommended.

The user of the Schmidt hammer should make up his own calibration curves for the particular concrete under study.

89. The rebound hammer can be used to good advantage during construction. Stowe (1974) used the hammer to investigate low quality concrete in the New Walter Reed Hospital, Washington, D. C. Rebound readings and compressive strength results of cores were sufficient to demonstrate to the contractor that inadequate concrete existed in 22 columns of the first floor. The contractor removed and replaced the 22 columns at an estimated cost of $1.5 million (1974 cost figure). Additional examples of application of the rebound hammer are cited in Grieb (1958), Willetts (1958), Moore (1973), and Victor (1963).

90. Windsor probe. "The Windsor probe equipment consists of a powder-actuated gun or driver, hardened alloy probes, loaded cartridges, depth gage for measuring penetration of probes, and other related equipment. The probe is driven into the concrete by the firing of a precision powder charge that develops an energy of 575 ft-lb (779.6 N·m)" (Malhotra 1976). Figure 18 illustrates the Windsor probe.

91. "The Windsor probe test is basically a hardness tester and, like other hardness testers, should not be expected to yield absolute values of strength of concrete in a structure. However, like the Schmidt rebound hammer, the probe test provides an excellent means for determining the relative strength of concrete in the same structure or relative strengths in different structures without extensive calibration with specific concretes" (Malhotra 1976).

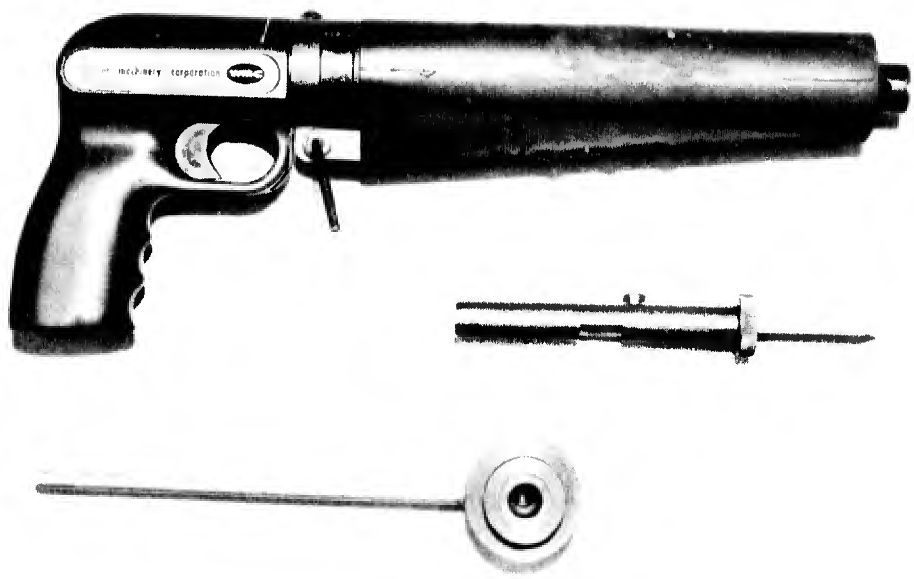


Figure 18. Windsor probe with accessory equipment

92. "The calibration charts provided by the manufacturer do not appear to be satisfactory. It is, therefore, desirable for each user of the Windsor probe to prepare his own calibration charts for the type of concrete under investigation. With change in source of aggregates, new calibration charts become mandatory" (Malhotra 1976). The use of the Windsor probe results in damage to the concrete, i.e., the probes must be withdrawn from the concrete and the hole patched.

93. There are a number of laboratory investigations, cited in the literature, that deal with the evaluation of the Windsor probe for estimating compressive strength of concrete (Gaynor 1969, Malhotra 1970, 1971). Klotz (1972) states that extensive applications of the Windsor probe test system have been made for the determination of in-place concrete strength and in-place quality. The Windsor probe has been used to test reinforced concrete pipe, highway bridge piers, abutments, pavements, and concrete damaged by fire. Klotz concludes that the probe affords a quick and relatively accurate means of ascertaining strength of concrete; he does not define the term "relatively accurate."

94. New York City's Board of Standards and Appeals approves the Windsor probe as an alternative to concrete core testing for compressive strength

(Concrete Industry Board, Inc. 1971). This author agrees with a number of other researchers in that the Windsor probe, or any other hardness tester of concrete, does not supply the accuracy required to replace conventional core tests.

Interior Condition of Concrete

95. The interior concrete condition, as well as the exterior condition, must be evaluated to fully determine the quality of the concrete in a structure or in an element. Various techniques are available for estimating the quality of concrete with depth. Techniques commonly used are described in the following paragraphs. Core sampling is the obvious method of obtaining information on concrete with depth. A discussion of coring (size, quantity, etc.) is presented later in this report.

96. The chemical, physical, and mechanical properties of concrete within a structure can be used to evaluate concrete soundness. Tests that are conducted for determining these three types of properties are discussed in Part IV of this report.

Dynamic or vibration nondestructive testing techniques

97. Ultrasonic pulse velocity. The ultrasonic pulse velocity method (see Figure 19), ASTM C597-71/CRD-C 51-72, involves the measurement of the time of travel of electronically pulsed compressional waves through a known distance in concrete. From known time and distance, the pulse velocity through the concrete can be calculated. This method is used extensively in the field for determining the general quality of concrete, locating cracked and inferior concrete, and providing input to condition surveys of concrete structures. The equipment is portable, has sufficient power to penetrate 50 to 70 ft (15 to 21 m) of good continuous concrete, and has a high data acquisition-to-cost ratio. Standard transducers and those used in boreholes are available and serve to eliminate most problems of access to surfaces, including those underwater. Empirical correlations between pulse velocities and compressive strengths have proved very useful for specific structures and concretes and can be established with limited coring.

98. The ultrasonic pulse velocity is probably the most widely used method for the nondestructive evaluation of in-place concrete and for

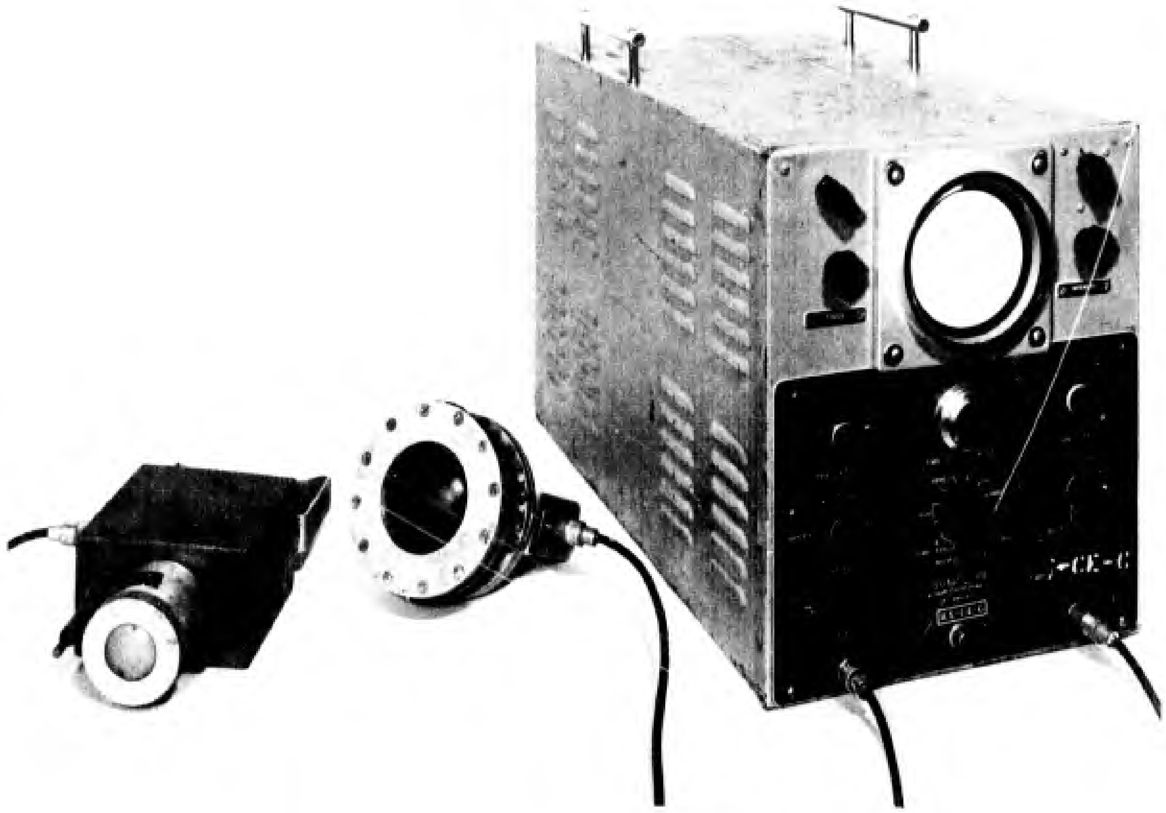


Figure 19. Ultrasonic pulse velocity apparatus

providing input for condition surveys of CE structures. Velocity measurements made through good quality, continuous concrete will normally produce high velocities accompanied by good signal strengths. Poor quality or deteriorated concrete will usually decrease velocity and signal strength. Concrete of otherwise good quality, but containing cracks, may produce high or low velocities, depending upon the nature and number of cracks, but will almost always diminish signal strength. These principles have been used over the years to determine the general condition and quality of concrete, to assess the extent and severity of cracks in concrete, and to delineate areas of deteriorated and/or poor quality concrete. The investigation into the extent of cracking in the downstream gate monolith at Lockport Lock, Illinois Waterway (Stowe et al. 1980), mentioned in paragraph 51 and shown in Figure 1, involved the combination of ultrasonic pulse velocity, dye injection, and coring methods. Vertical cracks were visible on each of the three exposed vertical faces of monolith 57. Velocity measurements were made through sections of this monolith in an attempt to determine the depth of these cracks and whether they joined within the monolith. To facilitate these measurements, a 7-3/4-in.-diam vertical hole was drilled in the monolith. By lowering an omnidirectional borehole transducer into the water-filled hole, it was possible to transmit signals to various points on the vertical faces of the monolith. Results of velocity measurements suggested that the visible crack on the downstream face of monolith 57 extended deep enough into the concrete to affect velocities 28 to 33 ft from the top of the monolith (see Figure 1). Horizontal borings into the downstream face also yielded information which was germane to the interpretation of crack depth.

99. The ultrasonic velocity data clearly indicated that the crack above the emptying culvert and the crack in the downstream face of monolith 57 extended well into the monolith. A dyed-water test was performed using the vertical boring in an attempt to further trace the cracks. Simply, the test involved filling the borehole with dyed water and observing where it exited on the three free surfaces of monolith 57. In summary, the combination of techniques resulted in what was felt to be a reliable and accurate assessment of the location and extent of cracks within the monolith.

100. A similar investigation (Thornton and Glass 1980) was performed at Lock and Dam No. 24 on the Mississippi River where ultrasonic velocity measurements were made through 14 concrete piers (see Figure 20) which constitute

Figure 20. Piers at Lock and Dam No. 24, Mississippi River, with wooden platform in place to facilitate pulse velocity measurement

part of the dam structure. Velocity measurements were also made through selected concrete columns that support the service bridge. Areas characterized by low velocities were delineated. The velocity data indicated that a condition of cracking and deterioration existed in the concrete around and near the trunnion, head plates, and connections of the embedded anchorage beams in the piers. Velocity measurements also indicated that the columns had been damaged by cracking. Follow-up work in the form of a condition survey (Stowe and Thornton 1981) which included coring and laboratory testing of concrete confirmed that 80 percent of the bridge support columns were in various states of deterioration and that zones of concrete downstream of the trunnion shafts were severely deteriorated. Cores recovered from areas representative of the different degrees of damaged concrete were examined and tested in the laboratory. Some of the cores were extensively damaged by cycles of freezing and thawing to depths of 2 ft. Compressive strengths of the cores ranged from 2010 to 9770 psi. Techniques for removal and repair of damaged concrete were recommended.

101. Another investigation consisted of a detailed testing program designed to determine the condition of the Lake Superior Regulatory Structure, Sault Ste. Marie, Michigan. Ultrasonic velocity measurements provided data on the condition of the tainter gates, operating machinery, and concrete piers of the Regulatory Structure and its foundation. Applications of other nondestructive testing methods are described in the report of this investigation (Thornton et al. 1981).

102. Acoustic pulse-echo. Situations arise frequently that call for the capability to develop data on a concrete structure which has only one accessible surface. These situations do not lend themselves to inspection and determination of problem parameters by use of "through-transmission" as described in the previous section on ultrasonic pulse velocity. Therefore, considerable expense and destruction of the structure are often incurred in defining the problem. These undesirable features can be drastically reduced or eliminated by using the pulse-echo technique.

103. As explained by Alexander (1980), the measurement of the time required for a pulse of sonic or ultrasonic energy to pass from one boundary to another and back to the original boundary is used to determine the thickness of concrete with only one accessible surface. The reflection, or echo, from the opposite boundary (back surface, crack, or deteriorated area) will occur

because of the difference in impedance of the concrete compared with air, water, or soil at the reflecting surface. This reflection technique is referred to as pulse echo.

104. In May 1977, an inquiry was received from the Savannah District concerning the availability of a method or technique for determining the depth of driven concrete piles which serve as foundation and support piers for a concrete loading wharf in Kings Bay, Georgia. After limited field testing, a short-term sonic pulse-echo investigation was performed in early June 1977. The performance of the system and the overall results of the investigation were considered to be acceptable, but difficulty with signal analysis because of extraneous reflections left some questions unanswered. At this point in development, successful pulse-echo measurements can be made on driven piles (see Figure 21), drilled piers in low-damping environments (water, sandy soil, etc.), and where extraneous reflections caused by adjoining elements are minimal.

Figure 21. Making pulse-echo measurements on driven piles in high damping environment

105. Resonant frequency. As stated by Thornton (1977), the resonant frequency method (see Figure 22), CRD-C 18-59, involves determination of natural frequencies of vibration in concrete specimens. Frequencies of vibration are then utilized to determine various physical properties of the specimens.

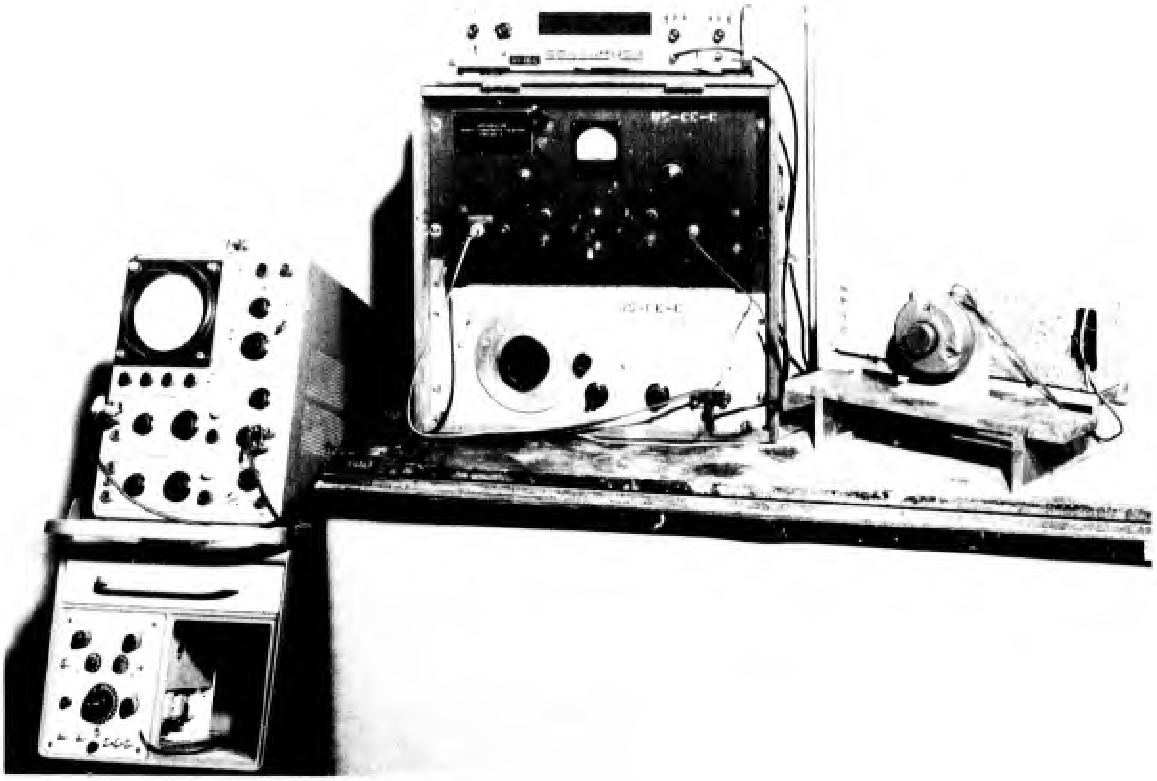


Figure 22. Laboratory resonant frequency vibration apparatus

These properties include dynamic Young's moduli of elasticity and rigidity (shear modulus) and Poisson's ratio. The resonant frequency method is used almost exclusively in the laboratory, and at WES it is used extensively for detecting changes in the dynamic moduli of test specimens undergoing accelerated freezing-and-thawing tests. It is also used as a monitor for the progress of deterioration of bars in sulfate resistance tests. In 1981, Alexander reported the results of work on the development of the resonant frequency technique as a nondestructive method for evaluation of concrete structures in place and in real time. The continuation of this work is in progress at WES.

106. Dynamic deflection. The dynamic deflection method of nondestructive testing (NDT) applies a sine wave repetitive force or oscillatory load to a slablike or continuous system such as floor slabs or pavement and measures the deflection produced in the system. By evaluating the deflection measurements, the shape of the deflection basin can be determined. The dynamic deflection method is a very practical and useful tool for the evaluation of highway and airport runway pavements, as well as other types of flat slab concrete construction. The evaluation is not limited to the quality or

condition of the structural member but can be extended into such areas as assessing support parameters, assessing joint efficiency, and determining extent or degree of crack damage.

107. WES directed an investigation in which this method was used to locate void areas beneath a concrete-lined river channel after the channel had been dewatered. The concrete lining consisted of a 6-in.-thick (150-mm-thick), V-shaped reinforced concrete slab with one vertical on six horizontal side slopes. Certain portions of the concrete lining were undermined in the late stages of construction when water in the diversion channel overtopped its banks and flowed into the channel. The purpose for using the dynamic deflection method was to delineate the undermined slabs so that replacement could be accomplished at minimum cost. By correlating the results of deflection measurements with the results of limited coring, a procedure was developed whereby voids with depths as small as 1/2 in. (10 mm) could be detected. The results of this investigation enabled the sponsor to realize substantial cost savings.

108. Acoustic emission method. When materials are subjected to stresses which cause them to be strained beyond their elastic limit, localized deformations will occur. The occurrence of these deformations in the forms of dislocation movement or microcrack growth result in the release of stored strain energy. The release of this energy causes the propagation of rapid elastic waves throughout the material. These elastic waves can be detected as small displacements by sensors placed on the surface of the material. It is therefore conceivable that this method could be used as an early warning signal to indicate the start of mechanical failure within a structure.

109. Probably the most extensive use of this method has been to monitor the in-service behavior of pressure vessels to indicate the presence and growth of fatigue cracks, and to monitor the response of systems to preservice load tests. Although some work was done as early as the 1950's by L'Hermite (1962) and some later data have been published by Green (1970), Malhotra (1972), and Mlakar, Walker, and Sullivan (1981), the application of acoustic emission techniques to the evaluation of concrete structures is very new. Malhotra (1976) says,

> The acoustic emission methods are still in their infancy. The equipment, though available commercially, is very expensive, and proper test methods have yet to be developed. Furthermore, observations can be made only during

a period of increasing deformation and stress, and this method cannot be used for individual or comparative measurements of concrete in a static condition of loading. However, this technique has potential for nondestructive evaluation of loading levels in structures. It may be possible to monitor large structural members to locate the origin of cracking and the zones of maximum deterioration. However, at the present state of development, the cost of equipment prohibits the use of this technique for the above type of investigation and limits its use to the testing of laboratory specimens only.

110. Other methods. There are other nondestructive methods available which have not been as widely used to gather data on in situ concrete for input to condition surveys. These methods include pullout tests, radioactive methods, nuclear methods, magnetic methods, electrical methods, and microwave techniques. These methods are described in detail by Malhotra (1976). Clifton et al. (1982) provides another source which contains descriptions, applications, and limitations of some of these nondestructive testing methods.

Embedded steel

111. Often there is concern for the condition of embedded items in the concrete, e.g., reinforcement steel, plates, steel insulating materials, anchorage steel, etc. This concern could be expressed where cracking or evidence of corrosion (e.g., rust stains) is present on a concrete surface, or where exposed steel due to concrete removal, shows evidence of corrosion.

112. Three methods of checking on the condition of embedded steel are chipping, core drilling, and excavation. Steel items can be located using design drawings and drilled using diamond core bits. Embedded steel covered with 3 to 4-1/2 in. of concrete can be located using one of two commercially available instruments, an R-meter (formally called a Teba Pachometer) and a V-meter (formally called a Covermeter).* Both instruments have been used successfully.

113. The principle of operation of both instruments is nearly the same. Both are based on a method of comparing the fixed electromagnetic characteristics of a reference transformer with the variable electromagnetic characteristics of a measured transformer. The variability is indicated on a specially calibrated galvanometer. The instruments operate on 1.5-volt batteries.

114. Advantages of these instruments include their portability, being

* Both meters are commercially available.

lightweight, and suitability for use in small spaces. The R-meter is capable of identifying different size rods up to 1 in. with a light cover of concrete up to 2 in.

115. The instruments appear to be limited in that they are only effective at relatively shallow depths, up to about 4-1/2 in. for the R-meter and about 3 in. for the V-meter. Identification of different size rods is possible with the R-meter but requires careful, particular work.

116. An example of application is given in the report by Saucier (1965). Saucier's work involved a laboratory investigation of the two instruments. Concrete beams and slabs were cast containing single and multiple embedded reinforcing steel of different sizes. Saucier's conclusions are basically reflected in the following: "Alignment and lateral positioning of reinforcing rods can be determined with the R-meter if rods are embedded not more than 4-1/2 in. and spaced not less than 2 in. apart." Knowing the diameter of an embedded rod, the thickness of concrete covering can be approximately determined to an accuracy of 1/4 in. for coverings up to 4 in.; the V-meter has the same accuracy for coverings up to 3 in. Saucier concludes that "when information on rod size and amount of cover are desired," the R-meter is more practical.

117. The principal advantages and limitations of excavating concrete to examine embedded steel are apparent to the reader. Stowe and Thornton (1981) report on a case involving Lock and Dam No. 24 on the Mississippi River. In this example, concrete was excavated to ascertain the condition of anchorage steel in a portion of a tainter gate. Figure 23 illustrates the exposed downstream portion of tainter gate pier No. 2 that contains cracking and exudation adjacent to the trunnion shaft. Concern was expressed that water infiltrating through these cracks and cracks abutting into the anchorage head plates (see Figure 24) could cause corrosion on a portion of the anchorage steel. Figures 25 and 26 illustrate, respectively, the excavation (about 1.1 cu yd) and the typical corrosion of the exposed head plates. About 1/64-in. thickness reduction of the 2-in. steel head plate was measured; the area affected covered about 12 sq ft on the two exposed head plates. The amount of corrosion was considered insignificant. A bituminous impregnated plastic cork covering the anchorage steel has apparently worked well in limiting the amount of infiltrating water getting to the steel through the concrete; the cork was placed during construction.

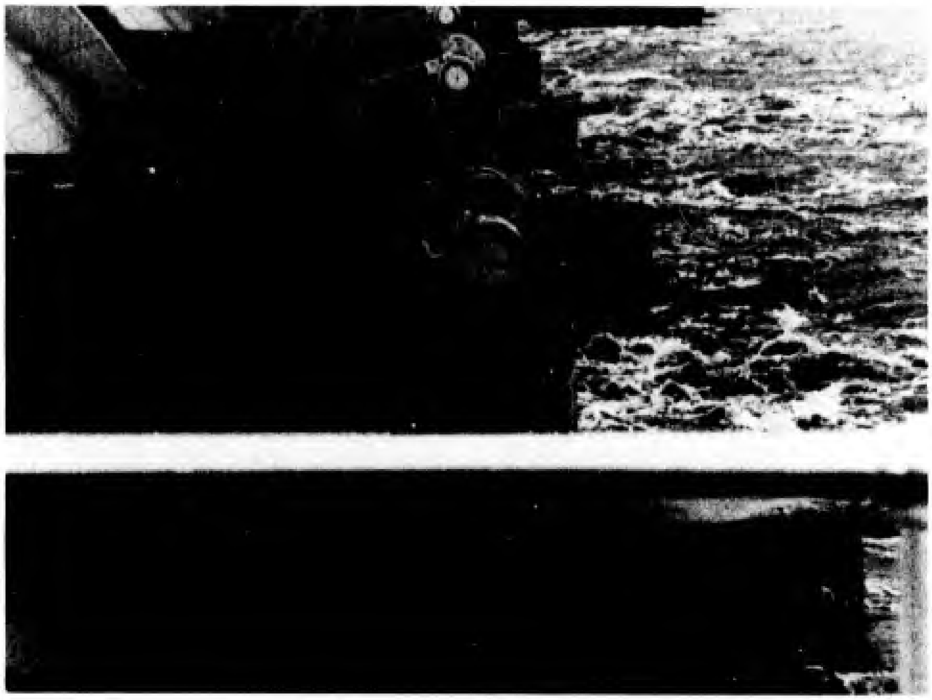


Figure 23. Looking across dam from pier No. 1. Typical cracking pattern with exudation downstream of trunnion shaft, pier No. 2, Lock and Dam No. 24, Mississippi River

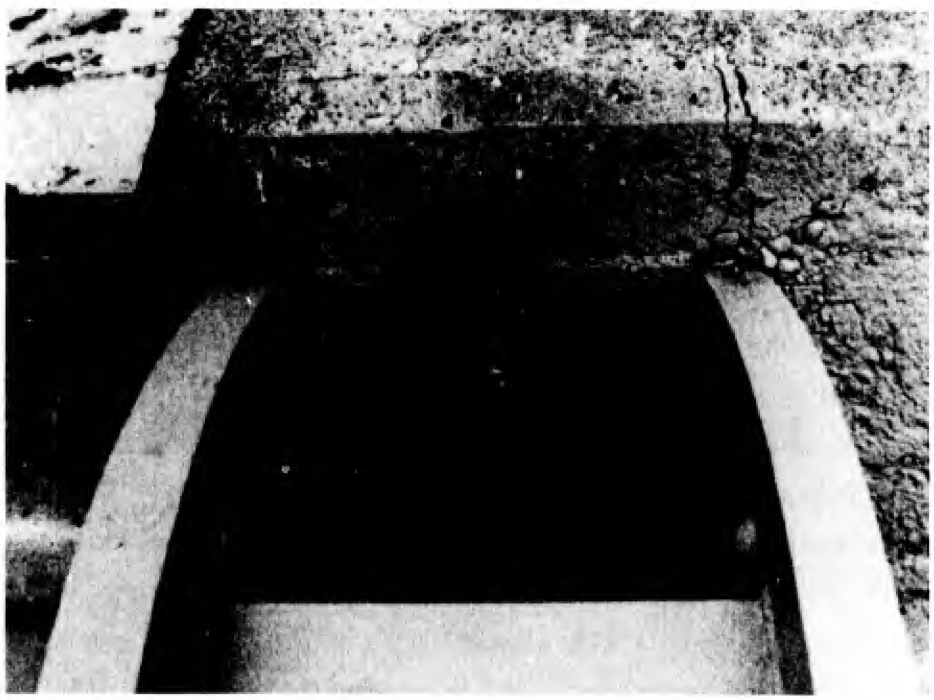


Figure 24. Looking upstream at trunnion shaft area. Typical crack pattern where anchorage head plates extend from concrete, Lock and Dam No. 24, Mississippi River

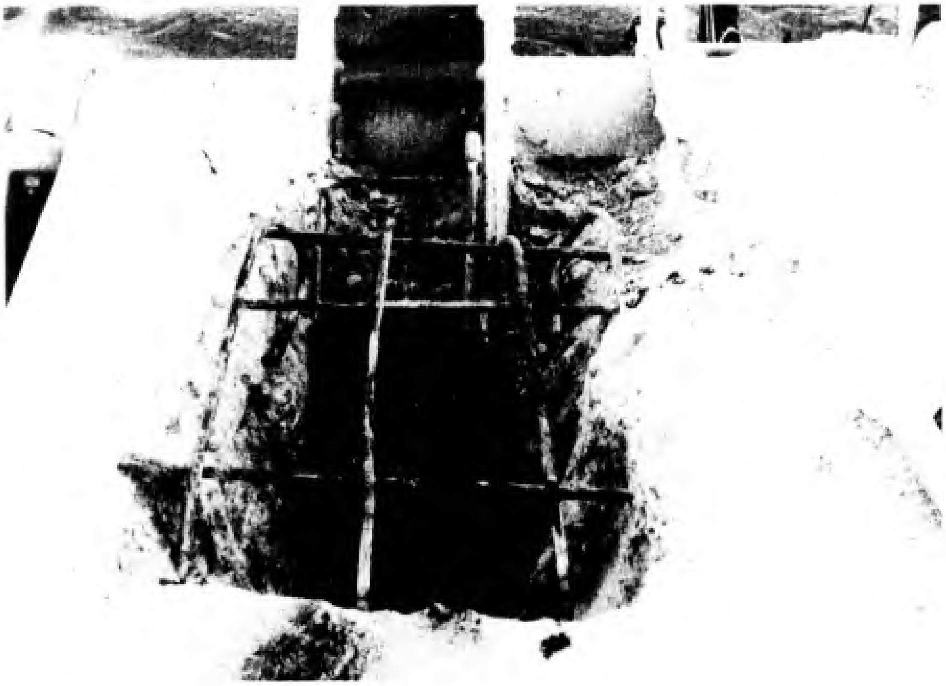


Figure 25. Looking downstream at trunnion shaft area. Portions of anchorage steel (head plates) and reinforcing steel exposed by excavating the surrounding concrete, Lock and Dam No. 24, Mississippi River

Figure 26. Looking upstream at cross member steel plate in foreground and anchorage head plates in the background partially uncovered, Lock and Dam No. 24, Mississippi River

118. Figure 27 illustrates a similar situation at the Cedars Lock and Dam on the Lower Fox River in Wisconsin; i.e., cracks and exudation. Recommendations were made for concrete removal and examination of the anchorage steel (Stowe and Alvin 1982).

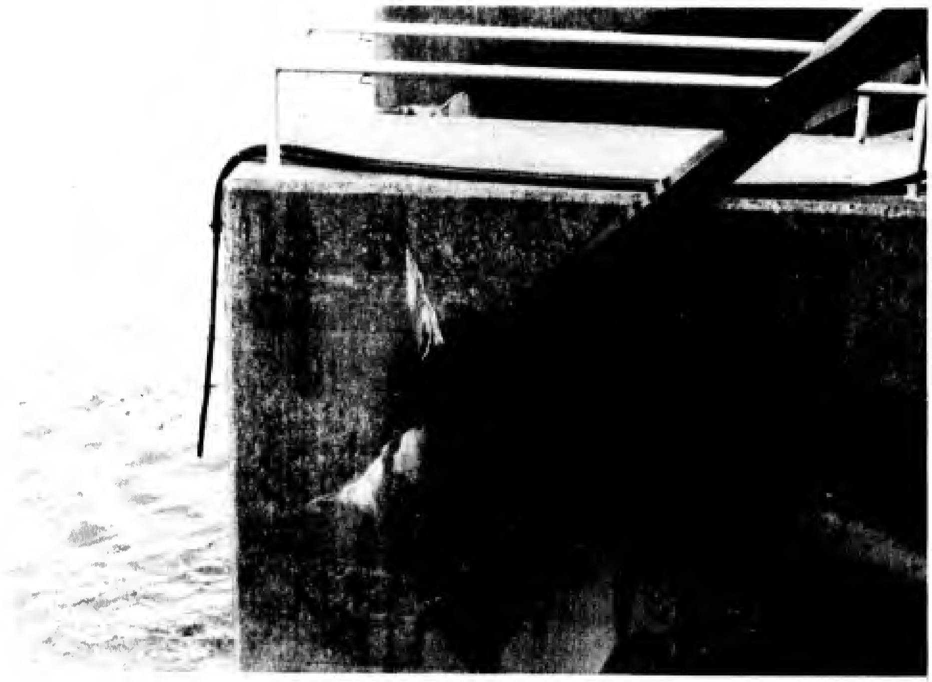


Figure 27. Taken from tainter gate pier No. 1, looking at north side of pier No. 2. Typical cracking through concrete pier at gate hinge pin, Cedars Lock and Dam, Lower Fox River, Wisconsin

Borehole inspection

119. Downhole logging is a useful adjunct to drilling and sampling of concrete. The logging is used for two general purposes: (a) interpretation of features and (b) ascertaining material properties. Three logging tools commonly used to directly examine geologic features, i.e., borehole camera (photographic), borehole television camera, and borehole televiewer, may also be used to evaluate features in concrete structures. When drill hole coring is not practical or core recovery is poor, these three logging tools can provide a method of locating cracks, voids, contacts (concrete and foundation), and other discontinuities of significance, as well as flowing water or other anomalous features. Engineering Pamphlet (EP) 1110-1-10 (OCE 1982) entitled "Borehole Viewing Systems" is in publication at this writing. The pamphlet describes and cites Corps availability of the borehole camera, televiewer, and television probes plus other downhole probes that may be used to evaluate

foundations. The pamphlet should be an excellent reference for those interested in using these downhole probes.

120. Several geophysical logging techniques may be used to provide supplemental data on the physical properties and condition of in-place concrete. In general, density, porosity, and velocity are the most common properties obtained from geophysical logs. In-place density can be directly obtained from the Density Log. "Porosity may be determined from several logs including Sonic, Density, and Neutron Logs" (ACI Committee 207 1979). These logs, in combination with Resistivity and Caliper Logs, provide a record of the uniformity of concrete with depth. Logging of drill holes and interpretation of logs should be done by firms which specialize in this exploration technique (ACI Committee 207 1979). Dohr (1974) and the Corps of Engineers EM 1110-1-1802 (OCE 1979a) are good references for those interested in the possible use of downhole geophysical logging to determine the properties of concrete. The use of the borehole camera and televiewer is described in the following paragraphs.

121. Borehole camera. "The NX borehole camera was designed to photograph subsurface conditions on the walls of small diameter holes in rock" (Trantina and Cluff 1963). NX coring results in a hole having a diameter about 3 in. in diameter.

122. The camera is housed in a 30-in.-long steel cylinder with a 2-3/4-in. diameter. "Light reflected from the wall of the hole is projected through a cylindrical quartz window into a conical mirror and upward into the camera lens" (Trantina and Cluff 1963). The unit uses a floating magnetic compass and tilt indicator to record information on each photograph. Thus, orientation and dip of features within the bore wall can be determined. The camera is lowered and raised in the borehole by operation of a hand winch or power winch. Continuous, 360-deg photographs are taken as the camera is raised from the bottom of the hole; black and white or color film may be used in the camera.

123. One advantage of the camera is that it allows for a direct examination of the condition of the bore wall. It can be run in small diameter boreholes which reduces the cost of drilling larger borings. The camera will operate in wet or dry holes; in a dry hole, the bore wall becomes more photogenic when sprayed with water or kerosene. The film allows for permanent records, easy storage, portability, and reproducibility. Interpretation of the photographs is not difficult.

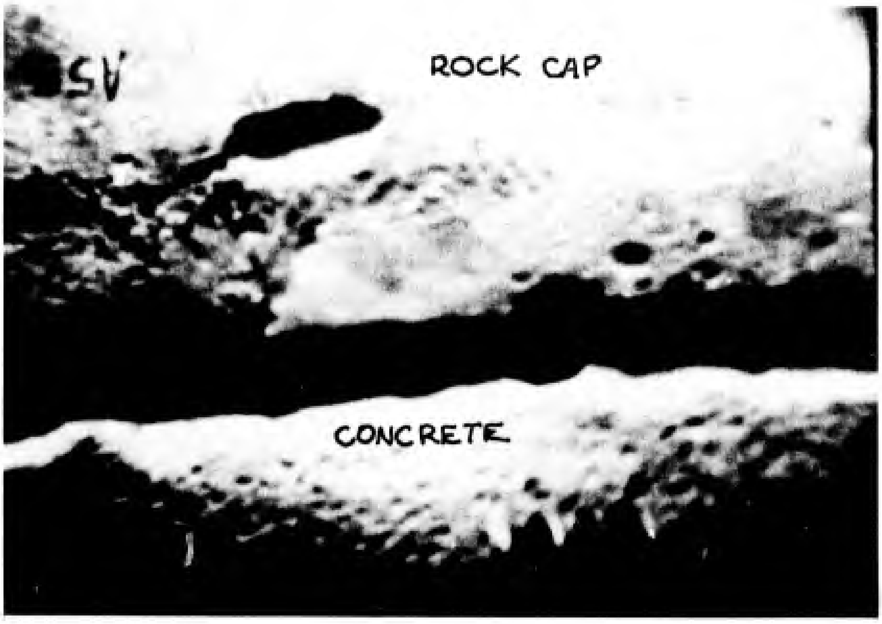
124. One limitation of the camera, when used in water-filled holes, is that it requires relatively clear water in which to function properly. Mud-caking of boreholes is troublesome; however, the hole can sometimes be cleaned by flushing. Holes greater than about 12 in. in diameter are difficult to photograph.

125. To the authors' knowledge, the NX borehole camera has not been extensively used to examine the condition of concrete in structures. Two applications, one by Thornton et al. (1981) and one by the Montreal Engineering Company, Limited, for Great Lakes Power, Limited (1981), are available. The main purpose of using the NX borehole camera on these projects was to evaluate the orientation and dip of natural foundation discontinuities. At the same time, supplemental information about quality of the concrete and masonry in the dam piers was obtained. An anomalous feature was noted in one of the piers reported on by the Canadians.

126. In Pier 8 on the Canadian side of the compensating works, a large gap at the concrete-rock interface was observed and water flow through the gap was reported. The borehole was located in the upstream section of the pier about 2 ft from the sides and about 8 ft from the upstream nose of the pier. An explanation of the significance of water flow at the concrete-rock interface under the pier was not presented in the report; it is possible that there is no significance in this particular case. However, this example points out the usefulness of the borehole camera in that an anomalous feature was observed. The core log described the contact of the concrete-rock interface as being crushed by drilling pressure, no drilling fluid lost, and 100 percent core recovery at the contact. There appears to be some inconsistency between the photograph interpretation and the core log. Figure 28 illustrates the quality of photographs obtainable with the borehole camera.

127. Borehole television. The television probe is configured in much the same way as the "NX" borehole camera. The TV camera is hooked up to a console and monitoring unit on the surface. The probe can be remote-controlled for focusing and aiming axially and in a radial direction. A real time continuous display can be viewed as the camera is run in the borehole. The display can be video-taped, if desired, for a permanent record.

128. One advantage of the TV camera is its focusing capability, with which large void sizes can be estimated. The camera can be used in "NX" size holes and in larger holes with a centering device.



a. Horizontal cracks, average width 4.5 mm, open contact between capping block and concrete (Montreal Engineering Company, Limited 1981). Rock cap overlays the concrete portion of a pier

b. Void 3 to 4 cm wide indicating honeycomb concrete, (Montreal Engineering Company, Limited 1981)

Figure 28. Borehole camera photographs, concrete borehole wall

129. For high quality resolution, the turbidity of the water should be nearly zero. Generally, the TV camera and accessory equipment are housed in a van or similar vehicle. Access to the borings in lock and dam sections may require barge and crane support.

130. Logan (1965) cites a dozen applications with the TV camera to evaluate foundations; e.g., a suspected shear zone at the Morrow Point Dam, orientation of recovered core by viewing bore wall features and matching the core with the pictures. At the Parker Dam, California, a TV camera was used to examine in-place concrete conditions prior to the installation of instrumentation for a stress analysis study (Logan 1965). The Corps of Engineers District personnel and staff members at WES have used the NX and a larger unit (used in boreholes up to 7-3/4 in. in diameter) with excellent results. The Corps reports are generally informal project reports that do not get published. Excellent results at the Gathright Dam in Virginia were obtained in foundation rock.* Cavities up to 25 ft in diameter were detected in rock at the Maramack Dam near St. Louis, Missouri.**

131. Borehole televiewer. The televiewer contains a continuously rotating piezoelectric transducer which probes the borehole wall with bursts of acoustic energy in a manner similar to sonar. Because the tool is moved vertically up the hole simultaneously with transducer rotation, a narrow, spiral strip of the wall is probed. Vertical velocity is controlled so that the entire borehole wall is logged. The log is oriented electronically by a fluxgate magnetometer rotating with the transducer and sensing magnetic north. The amount of energy reflected by the wall, and thus detected upon return to the transducer, is a function of the physical properties of the surface. A smooth surface will reflect better than a rough surface, a hard surface better than a soft one, and a surface perpendicular to the acoustic beam better than a skewed one (Zemanek et al. 1969).

132. One advantage of the televiewer is that quite small (1/8 to 1/4 in.) vugs and cracks can be identified in good, competent rock; this degree of resolution can be obtained in most concretes. The televiewer may be run in small exploratory borings NX in size. The tool works well in boreholes

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* J. S. Huie. 1982. Personal Communication, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** F. L. Smith. 1982. Personal Communication, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

filled with fresh water, saturated brine, crude oil, or drilling muds; thus, it has an advantage over the television and hard-film cameras.

133. Access to borings may require barge and crane support due to the size and weight of equipment and transportation vehicles. The magnetic orientation is disrupted in concrete that contains reinforcing steel close to the borehole (Stowe 1979). In mass concrete, reinforcing steel would not be a problem.

134. Zemanek (1969) cites numerous examples of application in rock with excellent results. His referenced paper contains photographs of televiewer logs illustrating voids, cracks, and artificially induced fractures. The USGS* and the Corps of Engineers** have used the televiewer extensively in geologic materials. The USGS has used the instrument in concretelike pipe, cemented well holes, and observed short concrete plugs downhole. Resolution in these cases was as good as in sound rock. The Corps of Engineers has used the televiewer in concrete dam piers where nearby reinforcing steel caused interference of the magnetometer. The pictures were distorted due to the magnetic interference. In private conversations with Keys* and Huie and Smith,** it was stated that the televiewer would be as effective at indicating voids and cracks in concrete. A modified version of the televiewer is purported to record excellent pictures in concrete with steel adjacent to the bore wall.

Underwater Inspection

135. Guidance is not available within the Corps of Engineers on procedures and techniques for conducting underwater inspection of structures. Guidance concerning the safety of divers is available through use of the Navy Divers Manual. The following paragraphs give helpful hints on some techniques that may be used and states some of the types of damage that may occur on portions of structures under water. A full treatment of underwater inspection was not within the scope of this study. It is recommended that engineering guidance for underwater inspection of structures be developed in the near future within the Corps of Engineers.

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* S. Keys. 1982. Personal Communication, U. S. Geological Survey, Denver, Colo.

** J. S. Huie and F. L. Smith. 1982. Personal Communication, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

136. The safety of a structure depends on the physical integrity of both the superstructure and substructure. The superstructure generally receives much more attention than those parts of the substructure located below the waterline. It is essential, from a safety standpoint, that the underwater portions of a structure be adequately inspected, maintained, and repaired as necessary.

137. In the authors' opinion, the synthesis by Lamberton et al. (1981) and the work by Brackett et al. (1982) are excellent references on the inspection, maintenance, and repair of substructures below the waterline. Lamberton's synthesis is a survey of practices of a number of state highway agencies, selected railroads, and turnpike and bridge authorities. The report identifies the policies, procedures, and techniques currently used to inspect underwater bridge substructures. The purposes of the synthesis are:

a. To identify problems with various materials and types of substructures.

b. To synthesize and evaluate underwater inspection procedures, equipment, and techniques.

c. To recommend the best procedures for underwater inspection.

d. To suggest areas in need of modification or improvement.

138. Visual examination is the primary method of detecting underwater problems. However, in turbid water, the inspector must use tactile examination to detect flaws, damage, or deterioration. In some cases, ultrasonic thickness gauges, computerized tomography, or television cameras may be required. Samples may be taken to help identify the problem and the extent of damage (Lamberton et al. 1981). Lamberton states that with proper training, equipment, and supervision, an effective job can be accomplished. He states that supervision personnel should be well-trained in structural inspection and have a good understanding of the structural configuration being inspected. Supervisors and divers with experience in construction may be able to determine damage or errors due to construction practices. Perspective schematics can be helpful in aiding diver orientation.

139. Prior to inspecting concrete under water, the various mechanical, physical, and electrochemical processes that could contribute to the deterioration of the concrete should be reviewed. The review should be site specific. Such information will assist the divers in interpreting what is observed.

140. Lamberton (1981) talks about underwater cleaning and some problems that may be encountered in underwater inspection.

> Cleaning marine growth from the underwater portions of the structures is almost always necessary. The extent of the cleaning depends on the amount of growth present and the type of inspection; indiscriminate cleaning should be avoided. Light cleaning can be done with a diver's knife or with hand tools. Tough jobs on concrete and steel require a high-pressure water blaster.
>
> Scour around piers and abutments is one of the problems that can be detected by inspection. Other problems include the deterioration of wood from attacks of marine borers, corrosion of steel, cracks, spalls, and cavities in concrete, and structural damage caused by construction, collision, abrasion, or storm. Structural failure resulting from overload, foundation failure, or maintenance failure may also be revealed during underwater inspection.

141. During the inspection, divers can use black and white or color closed-circuit television cameras and high resolution color still photography. Any measurement device, like a ruler used to indicate dimension of features, should be large enough to be viewed clearly when pictures are taken. The divers need to measure and record loss of structural sections. They should pry and probe around piles and columns and bracing for deterioration, tightness, soundness, missing rivets, bolts, nuts, etc. Narration by the diver during inspection may be accomplished by tape recorder.

142. During an underwater inspection, there are typical areas of construction weakness that should be looked at in detail; loss of protective stone facing, construction joints, joint sealants, evidences of flowing water, embedded fixtures, valves, openings, timbers, and scouring.

143. Wood deteriorates through various fungal processes (rot) and can be destroyed by marine organisms. Lamberton gives techniques for detecting the common marine-borers within U. S. waters. They are the Limnoria, a crustacean whose tunneling is limited to the outer shell of wood, and the Teredo (or shipworm), a mollusk whose larvae end up in the internal structure of the wood member.

144. Corrosion of steel is generally detected by visual inspection. Some of the nondestructive techniques discussed under Interior Condition of Concrete in this report are becoming increasingly useful. "Corrosion is identified by its typical reddish-brown rust-colored appearance and its pitted,

oxidized surfaces, usually showing loose flakes of oxide (Lamberton et al. 1981). The precipitated rust on marine steel members can form a dense coating to protect against additional corrosion; it's recognized by a greenish-black color.

145. Concrete and natural stone deterioration under water is generally recognized by the presence of cracks, spalls, and cavities. The cause of underwater deterioration of concrete is difficult to ascertain. Erosion is probably the easiest deteriorating agent to identify by examining debris, suspended sediment, ice, etc. Chemical substances, including magnesium irons in seawater, react with the calcium in the concrete. This reaction, plus sulfate attack, is accomplished by a substantial expansion and causes cracking and spalling of the concrete. Deterioration due to corrosion of reinforcing steel can occur under water as well as above the waterline; air and moisture must penetrate the concrete cover for corrosion to occur under water. "Several types of marine borers, including the Pholes, bore into concrete. However, this damage occurs infrequently in tropical or semitropical waters and appears to take place only in low-quality concrete" (Chelle's 1951).

Core Drilling

146. Core drilling is the best method of obtaining information on concrete within a structure in areas which otherwise cannot be observed. Core drilling is expensive and should only be considered when sampling and testing of interior concrete is deemed necessary (ACI Committee 207 1979). It can sometimes be tied in with foundation drilling efforts. Samples of concrete may be recovered by sawing or by breaking pieces off.

147. Within the Corps of Engineers, there is no formal guidance as to proper drilling equipment or drilling procedures for obtaining concrete or rock foundation cores. Appendix B, Drill Rigs, in EM 1110-2-1907, "Soil Sampling," (OCE 1972) contains recommended equipment and procedures that are currently followed. Core barrel and drilling bit guidance is contained in the Diamond Core Drill Manufacturers Association Standard literature (DCDMA) (1980).

148. Drill location and depth of coring dictate the type and size of drill equipment necessary. WES has used truck-mounted, skid, and portable rigs on many different structures. Drilling at locks and dams may require marine floating plant and crane support. A crane and small floating plant, an

8- by 20-ft barge with drilling well and skid rig, have been used in areas which are difficult to reach, such as behind a dam. Figures 29 and 30 illustrate drilling behind a dam and on a vertical surface to obtain horizontal cores.

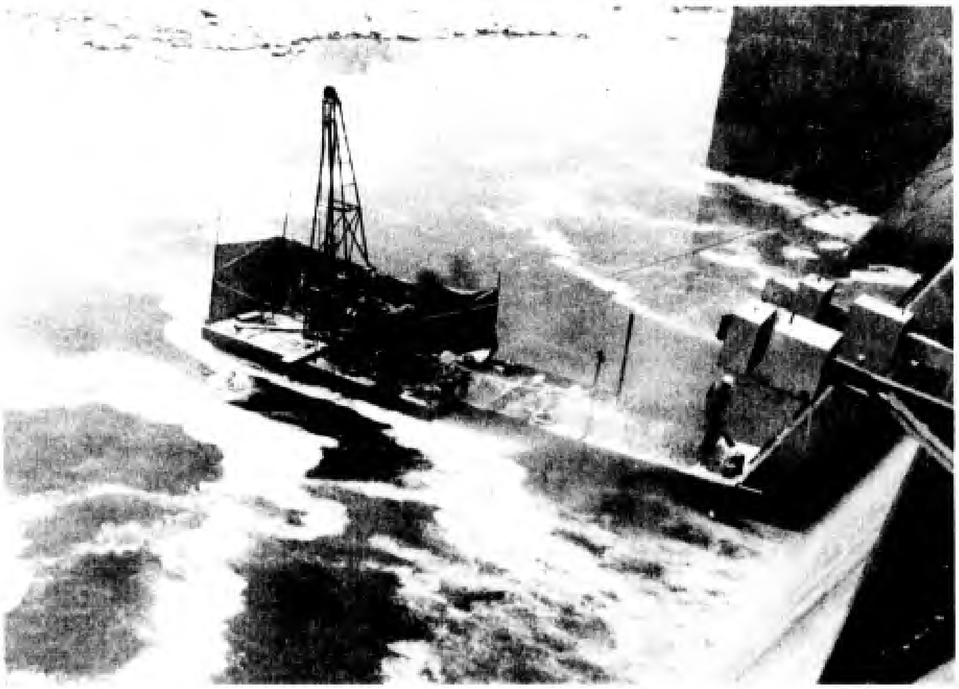

Figure 29. Floating plant with drill equipment behind dam, Brandon Road Lock and Dam, Illinois Waterway

Figure 30. Crane supporting horizontal drilling operation, Starved Rock Lock and Dam, Illinois Waterway

Sampling hardened concrete

149. A practical approach to sampling of concrete is to take random samples from each category of concrete. ASTM C823-75/CRD-C 26-76 states additional requirements for sampling hardened concrete.

> Samples may be taken to exemplify unusual or extreme conditions or features to aid in the identification of causes of distress or failure of concrete, but these samples should be kept apart from samples that are taken to exemplify statistically the properties of the concrete in place. Thus, the samples may be of two types, namely, (a) those that, together, are intended to be representative of the variability of the concrete in place, and (b) those that display specific features of interest but are not intended, individually or collectively, to be representative of any substantial proportion of the concrete in place.
>
> The samples should include portions of both near-surface concrete and concrete at depth, because the concrete may vary substantially with depth in the development of cracking, deterioration of the cement paste, progress of cement-aggregate reactions, and other features.

150. Depth of cores will vary depending upon intended use and type of structure. The minimum depth of sampling concrete in massive structures should be 2 ft in accordance with ASTM C823-75/CRD-C 26-76.

151. The samples should be sufficient in number and size to permit appropriate laboratory examination and testing. For compressive strength, static or dynamic modulus of elasticity, the diameter of the core should be not less than 3.0 times the maximum size of aggregate. For 6-in. maximum size aggregate concrete, 8- or 10-in.-diameter core is generally drilled because of cost, handling, and laboratory loading capabilities.

152. A word of warning should be given against taking NX size core in concrete. When 2- to 6-in. maximum size aggregate concrete is cored, NX size (2-1/8-in. diameter) core will generally be recovered in short pieces or as short pieces and broken core. The reason for breakage occurring is because of the ratio of core diameter to aggregate diameter. When this ratio approaches one, there is simply little mortar bonding the concrete together across the diameter of the core. Thus, the drilling action can easily break the core. When drilling in poor quality concrete with any size core barrel, the material generally comes out as rubble.

153. ASTM C823-75/CRD-C 26-76 should be consulted for detailed information concerning:

a. Recommended sampling method.

b. Sample size (number of samples).

c. Evaluation of test results on the basis of variability.

d. Evaluation of the quality of concrete.

e. Sampling procedures.

154. Samples must be properly identified and oriented with permanent markings on the material when feasible. Location of borings must be accurately described and located on photographs or drawings. Cores should be logged by methods similar to those used for geological subsurface exploration. "Logs should show, in addition to general information on the hole, conditions at the surface, depth of obvious deterioration, fractures and conditions on fractured surfaces, unusual deposits, coloring or staining, distribution and size of voids, locations of observed construction joints, and contact with the foundation or other surfaces" (ACI Committee 207 1979). "The concrete should be wrapped and sealed as may be appropriate to preserve the moisture content representative of the structure at the time of sampling, and should be packed so as to be properly protected from freezing or damage in transit or storage, especially if the concrete is very weak" (ASTM C823-75/CRD-C 26-76). Figure 31 illustrates a typical log for a concrete core recovered during a condition survey.

In situ stress in concrete

155. Some cracking and other distress of concrete in structures can be attributed to excessive stress, design errors, or construction errors, or a combination of the three. Critical residual stresses may develop and cause concrete deterioration which could then compromise the integrity of the structure. The problem is to locate possible areas of high stress concentrations and determine the stress fields. During a condition survey, it may be desirable to ascertain the in situ stress at a given location. The over coring technique provides a method of accomplishing this end and will yield absolute measurements.

156. The over coring technique was originally developed in the study of rock mechanics. However, in the last 10 years it has also been applied to investigate in situ stress in concrete structures (ACI Committee 207 1979). The procedure was developed by the Bureau of Mines and documented in their Information Circular No. 8618 dated 1974 (Hooker and Bickel 1974).

157. A three-component borehole deformation gage (BDG) is designed to

Hole No. L WES L-2-78

DRILLING LOG	DIVISION Chicago District	INSTALLATION Lockport Lock	SHEET 1 OF 1 SHEETS

1. PROJECT Compliance Phase
2. LOCATION (Coordinates or Station) River wall of lock Monolith 57, 15' N/S from D/S face of mono 5' from vert face chamber side of wall
3. DRILLING AGENCY WES
4. HOLE NO. (As shown on drawing title and file number) L WES L-2-78
5. NAME OF DRILLER Henry McGee
6. DIRECTION OF HOLE [X] VERTICAL [ ] INCLINED ______ DEG. FROM VERT.
7. THICKNESS OF Concrete 9.85
8. DEPTH DRILLED INTO ROCK --
9. TOTAL DEPTH OF HOLE 9.85
10. SIZE AND TYPE OF BIT 6 x 7-3/4-in. Diamond
11. DATUM FOR ELEVATION SHOWN (TBM or MSL) MSL
12. MANUFACTURER'S DESIGNATION OF DRILL S & H Skid Rig
13. TOTAL NO. OF OVER-BURDEN SAMPLES TAKEN: DISTURBED -- UNDISTURBED --
14. TOTAL NUMBER CORE BOXES 2
15. ELEVATION GROUND WATER --
16. DATE HOLE: STARTED 2/15/78 COMPLETED 2/20/78
17. ELEVATION TOP OF HOLE 585
18. TOTAL CORE RECOVERY FOR BORING 100 %
19. SIGNATURE OF INSPECTOR Richard L. Stowe

ELEVATION a	DEPTH b	LEGEND c	CLASSIFICATION OF MATERIALS (Description) d	% CORE RECOVERY e	BOX OR SAMPLE NO. f	REMARKS (Drilling time, water loss, depth of weathering, etc., if significant) g
585	0		Finished surface, good condition 0.0 to 0.9' new concrete overlay, crushed agg ~3/4" max size, brn in color. 2-3% entrapped air.	100		WL Run 1.75' Began 9:32 Rec 1.75' End 9:30 Loss -- Time 18 min Gain -- Drl time 18 min Hyd press 240 Water press 50 RPM 100 Drl Action Smooth Water ret 100%
584	1		0.9' begins old concrete, lt brn, river gravel ~3" max size. Subparallel cracking thru agg and matrix.	Run 1		
583	2		MB 0.85' interval of frost damage beneath overlay.	100		Driller's notes lost
582	3		Vertical crack, wht deposit on ~10% surface. Crack probably due to alkali-silica reaction. Goodly number of pockets of wht material easily dug w/ knife blade. Small agg and sand grains affected.			
581	4		Gel, wht soft	Run 2 100		Driller's notes lost
580	5		Concrete has voids up to 1/2" in length & depth; 2 to 3 voids per linear foot. Core Spin	Run 3	Box 1	
579	6		Concrete as above Vertical crack through core 100% surface wht material Construction joint, slightly honeycomb, surface dk brn to dk gry	100		WL Run 4.15' Began 2:45 Rec 4.15' End 3:35 Loss -- Time Gain -- Drl time 50 min Hyd press 240 Water press 50 RPM 100 Drl Action Smooth Water ret 100%
578	7		Concrete as above Crack ends			
577	8		MB		Box 2	Waxing portions of 1st 15', 19'-29', and 38'-48'
576	9					Abbreviations: MB = m chine break brn = brown lt = light agg = aggregate dk = dark gry = gray
575	10		MB End of Boring	Run 4		

ENG FORM 1836 MAR 71 PREVIOUS EDITIONS ARE OBSOLETE. PROJECT HOLE NO.

Figure 31. Typical information included on a drill log for concrete core L WES L-2-78

measure diametral deformation during over coring along three diameters 60 deg apart in a plane perpendicular to the walls of a 1-1/2-in.-diam borehole. The 1-1/2-in. hole is over cored with a 6-in. core barrel; the amount of stress relieved by the over coring is obtained using the measured strains. Additional items required include three strain indicators, orientation and placement tools, calibration jig for the BDG, biaxial chamber used to determine Young's modulus of the retrieved core, drilling equipment, and miscellaneous hand tools. The procedures for conducting over coring tests are presented in method RTH 341-80 (U. S. Army Engineer Waterways Experiment Station 1980). Figure 32 illustrates the borehole stress-relief technique.

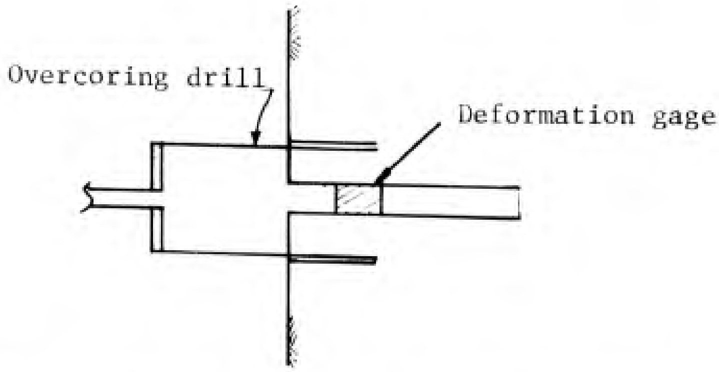

Figure 32. Borehole stress-relief technique

158. When properly applied, the borehole stress-relief method furnishes the three-diametral stress field in mass concrete. The technique can be used at depths to 1000 ft.

159. Accuracy of the results are influenced by many factors including: (a) precise orientation of the gage in the borehole, (b) good seating of the gage points against parent rock, (c) vibration of drill as over coring proceeds, (d) temperature of the drill water, (e) complete waterproofing of the gage, (f) influence of joints and/or voids on stress distribution within the concrete (Wallace, Slebir, and Anderson 1969). More difficulty is experienced in vertical holes than in horizontal holes with core recovery and cable breakage. An example of application is cited in ACI Committee 207 1979:

> The U. S. Bureau of Reclamation used the over coring stress relief method to investigate three thin arch dams located near Phoenix, Arizona.…
>
> Drilling three horizontal holes, which intersected near the center of the structure and at an angle of 22.5 deg with each other, produced accurate determinations of

> in situ maximum and minimum stress conditions. The results further showed that in arch dams, a single drill hole drilled approximately normal to the principal stresses in the vertical-tangential plane was adequate for maximum/minimum stress determinations.
>
> The 6 in. (152 mm) overcore recovered was also tested for triaxial shear, compressive strength, tensile strength, modulus of elasticity, Poisson's ratio, specific gravity, absorption, alkali-aggregate reaction, and used for petrographic examinations.

160. Two other methods of determining in situ stress conditions of rock masses could be used in concrete; they are the photoelastic stressmeter and the flatjacks. The photoelastic stressmeter is used in a similar fashion as the over coring technique. A short hole is drilled, the bottom of the hole smoothed, a photoelastic gage bonded to the bottom of the hole, and then the gage is over cored. The flatjack method involves cutting a slot in the concrete and provides a measure of actual stress in the surface plane. However, this method is restricted to near-surface measurements because of the difficulty of cutting deep flatjack slots.

Instrumentation

161. Supplemental instrumentation may be needed to evaluate questionable behavior detected during a condition survey of a concrete structure. "Measurement of the overall movement of the structure may be needed or of movement between monoliths along joints or displacement at cracks, as well as hydrostatic pressures in joints and cracks and under the structure (uplift)." (Jansen 1980).

162. Some of the more common instrumentation used by the Corps of Engineers on civil works structures will be discussed in the following paragraphs. The use and principle of operation is given for each instrument; where available, the advantages, limitations, and examples of application are also given. The most comprehensive guidance for instrumentation for concrete structures in the Corps is Engineer Manual 1110-2-4300 (OCE 1980). The reader is referred to this manual for examples of additional instruments used to measure the behavior of concrete structures.

163. "EM 1110-2-4300 describes new techniques which have evolved from recent technological advances in electronic instrumentation as well as methods

which have been developed over a long period of time for the preparation, fabrication, protection, and installation of instruments and the collection of data therefrom" (OCE 1980). The manual describes instruments used for the measurement of strain, stress, joint movement, pore pressure, interior concrete temperature, uplift pressure, leakage, structural deflection, head loss, and distance measurement.

164. Most of the information presented in the following paragraphs is taken from EM 1110-2-4300. Some information is taken from Lesson 307 in the Maintenance and Repair of Concrete Structures course taught at WES (Hoot 1981). The instruments are grouped as to use (the job they perform); hence, they are roughly categorized.

Strain and deflection measurements

165. Strain and deflection measurements on existing structures can be made using the vibrating wire strain gage described in EM 1110-2-4300 (OCE 1980). This gage is used in conjunction with one of several mechanical strain gages for external applications.

166. Vibrating wire strain gage. "A vibrating wire strain gage consists of a pretensioned fine steel wire clamped between two end flanges and enclosed in a tube. The end flanges can be welded or bolted to the (free) surface of a structure; for example, across a joint or crack. Forces acting on the structure produce strains which introduce relative movements between the end flanges and thus a change of tension in the steel wire. A coil is mounted in the gage housing, adjacent to the wire, and is electrically coupled to the measuring equipment. A current pulse, generated by the measuring equipment, energizes the coil, thus plucking the wire and causing it to vibrate at a natural frequency determined by the tension in the wire. The vibrating wire, in turn, induces an a-c voltage in the coil with a frequency corresponding to that of the vibrating wire. The frequency of the coil output voltage is sensed by the measuring equipment (OCE 1980). This type of gage gives relative measurements; i.e., subsequent readings are indicative of strains occurring in the concrete after installation of the gage.

167. The main advantages of the vibrating wire meter are: good long-term stability; high sensitivity; cable lengths have no effect; and relative insensitivity to moisture and resulting electrical ground leakage.

168. Mechanical strain gages. These types of gages are used for measuring long-time static strains on structures. Strain is derived from

measuring distance changes between attached reference points on the structure. Reference points are fixed to the structure by bonding contact seats to its surface, or by drilling holes in the structure wall and embedding inserts to support contact seats. Typically, the gage has conical points which are seated in small holes in the reference points when strain measurements are made. A dial attached to the gage indicates the positions of the two reference points from which the strain is obtained before and after the stressed condition. The gage length varies for different instruments and readings of one ten-thousandth of an inch (0.0001 in.) can be made.

169. The main advantages of the mechanical strain gages are: good long-term stability; good resolution to 0.0001 in. linearly over distances up to 1/4 in.; portability and ease in use.

170. One disadvantage is that the attached reference points on the structure are easily knocked off in areas of heavy traffic. The points must be covered or otherwise protected where susceptible to damage.

Expansion and contraction measurements

171. Relative movement across joints or cracks can be measured with the single or multiple position borehole extensometer or a ball-n-box gage. The extensometers can be used for internal or external applications; the ball-n-box gage is used for external applications.

172. Single (SPX) and multiple (MPBX) position borehole extensometers are well suited to measuring movement of cracks that are not surface accessible. The instrument is fitted into a borehole and anchored at a single or multiple locations. These locations can be between cracks in the interior of a structure so that the nature of movements can be monitored. The units measure the relative displacement of the borehole anchors which are mechanically fixed to the wall of the borehole. Each anchor is connected to a tensioning wire which is attached to the sensing head. As the anchors move, they in turn move a mechanism in the sensing head which is connected to an electrical transducer that records the movement of the anchor.

173. The advantages of the SPX or MPBX include small physical size, good sensitivity, remote electronic readout, and suitability for use in drill holes as deep as 1000 ft.

174. An example of application of both single and multiple borehole extensometers is presented for an instrument package set up to monitor structural

movement at the Starved Rock Lock and Dam located on the Illinois Waterway. The following is in large part taken from Anderson's letter report.*

175. The purpose of the instrumentation at Starved Rock was to monitor possible movement of piers 10 and 11. Chicago District personnel believed that movement of one or both piers might be the cause for jamming of the tainter gate located between the piers. There was also speculation that ice formations between pier 11 and the abutment rock might be pushing the pier toward the river. Pier 11 is the left abutment pier.

176. The instruments used to monitor possible movements included 10 SPX, one 3-point MPX and a Slope Indicator Digitilt. Some of the extensometers were placed horizontally through the leftmost concrete pier and into the left abutment, which is a high cliff; maximum depth embedment was 146 ft. Other extensometers were placed vertically through the piers and well into the foundation rock. A number of single point extensometers were placed between piers 10 and 11. The instruments were read by project personnel for a period of one year so that structural movements could be monitored during periods of high and low temperature extremes and various upper and lower pool elevations.

177. Conclusions drawn are: Instrument data were reduced and plotted continuously throughout the year with no significant movements observed. Extensometers 4 and 6 did show movement; however, these extensometers measured distance changes between piers 10 and 11 and were completely exposed to ambient temperatures. Although invar steel tape was used to reduce the effects of temperature fluctuations, the readings closely followed changes in temperatures. Movements observed with extensometers 4 and 6 are believed to be solely temperature effects on the piers. The instruments installed with their measuring tapes encased in boreholes showed little, if any, change. The problem with the operation of the tainter gate between piers 10 and 11 may be due to expansion of the concrete in piers 10 and 11. The outer concrete (1 to 2 ft) in these piers has been deteriorated by cycles of freezing and thawing and by alkali-silica reaction. The Starved Rock dam is undergoing a major rehabilitation; when complete, and if expansion of the concrete pier has caused the problem, the repair of piers 10 and 11 may arrest the problem; i.e., allow the tainter gate to operate freely.

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* R. Anderson. 1982. Letter report in draft form, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

178. Ball-n-box. Relative displacement in three orthogonal directions across a joint or crack can be measured with this instrument. The gage consists of a chrome-plated steel or brass rod with a steel ball attached to the tip of the rod. The rod is attached to the structure adjacent to a crack or joint. A hollow box frame is attached to the structure on the other side of the crack or joint. The three reference faces of the box have been machined orthogonal to each other.

179. A dial gage is used to measure any movement of the steel ball. The difference between the previous and present reading for any of the three reference faces yields the relative movement of the two sides of the crack or joint.

180. The advantages of the ball-n-box gage are its accuracy to 0.001 in. and its three-dimensional measurement capability.

Stress and pressure measurements

181. Stress and pressure measurements can be made with the over coring techniques (see section on Core Drilling), the vibrating wire stress meter, the WES hydrostatic pressure cell, and an open tube piezometer.

182. Vibrating wire stress meter. "The vibrating wire stress meter manufactured by Irad Gage has been designed to monitor stress changes in rock, coal, or concrete under the most adverse environmental conditions. When prestressed into a 1-1/2-in. borehole, the cylindrical gage can sense stress changes of as little as 2 psi. The stress changes act on the gage and alter the period of the resonant frequency of a highly tensioned steel wire clamped diametrically across the gage. Because the stress meter is rigid compared to the surrounding material, conversion of the frequency readings to stress changes does not require accurate knowledge of the media modulus. Calibration charts are supplied with the gages" (OCE 1980).

183. The advantages of the stress meter include: (a) installation in dry or wet holes at depths of up to 100 ft, (b) signal cable lengths up to a mile can be used, and gages for maximum readings to 10,000 psi can be obtained.

184. WES hydrostatic pressure cell. "A hydrostatic pressure cell for measuring pore water pressure in concrete is available from the Operations Branch, Instrumentation Services Division of the Waterways Experiment Station, P. O. Box 631, Vicksburg, Mississippi 39180. Its principle of operation is the amount of electrical resistance generated in a full bridge electrical resistance strain gage circuit bonded to a metal diaphragm that reacts to water

pressure on its face. The metal diaphragm is directly behind a porous stone in the instrument face. The pore pressure deflects the metal diaphragm inducing strain which is proportional to the pore pressure at the face of the meter. Readings are taken with a standard strain indicator such as the Carlson test set or Biddle strain gage indicator" (OCE 1980).

185. Open tube piezometer. "The open tube piezometer is the most common piezometer used in the Corps. It is used to determine water pressure at a given location such as in a particular geologic formation or at the base of a structure. The piezometer measures uplift pressure. It consists of a porous tip at the bottom of a watertight tube. The assembly is usually installed vertically with the porous tip located at the elevation the pressure is to be measured. Water enters at the porous tip and rises up the tube a distance equal to the pressure head at the tip. The water level in the tube is determined by sounding in the tube with a graduated water-sensitive probe. The distance to the water is subtracted from the known elevation of the top of the tube to determine the water level" (OCE 1980).

186. The open tube piezometer has been used extensively throughout the Corps. Many district offices have examples of application for the reader's review.

Leakage measurements

187. Flow rates are usually easy to isolate and measure and can be a very important indicator of structural performance. Observations of leakage from construction joints, lift joints, and cracks provide a means for judging this performance. An increase in flow can signal the opening of joints or cavitation, while a decrease in flow might signal the clogging of drain holes or sealing of openings. The readings reflect indirectly on structural performance and must usually be tempered with other factors such as rainfall and fluctuations in contiguous water levels. The vee-notch weir is widely used to measure flow.

Vee-notch weir

188. Measurement of flow in selected lengths of gutters may be accomplished by inserting vee-notch weirs to measure total cumulative flow above each weir. Individual drains or joints leaking excessively may be gaged separately. In some places where weirs are not suitable, a container of known volume may be used to determine the flow.

Tilting and deflection measurements

189. Tilting and deflection measurements can be obtained using the electrolevel or the inclinometer. These two instruments are more adaptable for use on completed concrete structures while plumblines are generally installed during construction.

190. Electrolevel. "The Electrolevel is a British instrument which is designed to provide a remote-reading facility for measurement and control of small angular movements. It has a spirit level vial with a precision-ground upper surface that is filled with an electrically conducting liquid and has three electrodes of platinum rigidly attached to the inner surface. The liquid is an alcoholic solution that has virtually the same properties as the liquid of a spirit level. The bubble runs on the curved surface which is free from discontinuities over the operating range. Movement of the bubble changes the electrical resistance values between the inner and outer electrodes. By using an alternating current bridge circuit the bubble position can be read with a pointer-type instrument (meter movement)."

191. "The Electrolevel vial provides an electrical signal proportioned to the angular deviation from horizontal over a range of ±30 minutes of arc. When used with a bridge circuit and detector amplifier, indications of tilt of less than 1 sec of arc can be displayed. The Electrolevel heads can be located at distances up to 300 ft from the detector. More detailed information about this tilt measuring device can be obtained from Tellurometer U. S. A., Division of Plessey Incorporated, 87 Marcus Boulevard, Hauppauge, New York 11787" (OCE 1980).

192. Inclinometer. "An inclinometer is a device which measures the deviation from the vertical of a flexible casing installed in a borehole. Deviations can be converted to displacements by trigonometric functions. Successive measurements enable the determination of the depth, magnitude, and rate of lateral movement (tilt). This type of instrument has been found to be useful in obtaining measurements of movement in levee foundations (Kaufman and Weaver 1966), excavated slopes (Burland and Moore 1973), concrete retaining walls (Meese 1970), and concrete dam piers (Anderson 1982)."*

193. "Advantages of the inclinometer (there are about five makes available) include: (a) wide range of application from earth abutments to rigid

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* op. cit.

concrete structures, (b) inclinometer systems are compatible with automatic data processing equipment, and (c) if, during field use, the devices are treated as precision instruments, reliable and repeatable results can be expected" (OCE 1980).

194. Limitations include: (a) reading affected by fine material in the grooves of the casing, (b) instruments can be damaged by jolting such as allowing the probe to hit the bottom of the hole, (c) moisture in the cable connection will cause problems, and (d) problems can arise if routine maintenance such as checking for damage to O-rings, moisture, or dirt is not performed.

195. An example of application of a Digitilt inclinometer is presented in Anderson's letter report;* the Digitilt is made by Slope Indicator Co. of Seattle, Washington. Background information is given in this report under the subheading "Single and Multiple Position Borehole Extensometer."

196. One inclinometer boring was drilled through pier 11 (the abutment pier) to a depth of 110 ft. The bottom of the boring was located 60 ft below the base of pier 11 in what was assumed to be undisturbed bedrock. Readings were taken at 5-ft intervals starting from the bottom of the boring to the top of the pier. After installation, zero readings were taken and subsequent readings were taken more or less weekly for about one year.

197. The Digitilt inclinometer data showed a random deviation of approximately ±0.18 in., which falls within the accuracy of the instrument and the ability of the operator to reproduce the readings. The inclinometer did not detect a discernible movement of pier 11 nor of the foundation material.

Precise measurement systems

198. Where there is a question of movement of a structure or foundation, measurements of lengths, angles, and alignment may provide answers or give some understanding as to the nature of a problem. Optical, laser, and wavelength techniques are available to provide alignment, distance measurement, and minute movement to high degrees of accuracy (OCE 1980).

199. "On dams and similar structures these techniques require setting up permanent and movable monuments as reference points for using the instruments and targets. Permanent marker points are set in the top and the toe of structures, and length, alignment, and deflection readings are made with the precise instruments set up on these monuments. Readings from the electronic

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* op. cit.

instruments that relate to distance measurement are generally accurate to 0.01 ft while alignment measured with the use of micrometer targets are recorded to the nearest 0.001 in., and are usually made at night to avoid troublesome optical distortions due to sunlight and heat radiation" (OCE 1980).

200. The precision alignment instruments, i.e., laser and theodolite, function by establishing a reference line described by the instrument and the measurement of the distance between markers on the structure and the reference line. Other types of alignment measurements used throughout the Corps are triangulation and trilateration.

201. Laser alignment instruments. "The laser system consists of a two-component unit, a transmitter and a receiver. The transmitter developed by the U. S. Army Engineer Topographic Laboratory (OCE 1980) is a continuous-wave helium-neon gas laser, mounted in a yoke, which is similar to the standards of a theodolite, with elevation and azimuth adjustments. The laser mount is attached to a tribrach which has a built-in circular level and optical plummet for centering on a reference point. The laser exciter is a separate unit and can be operated by either 12-v d-c or 115-v a-c current. The laser transmitter is equipped with a 2.0-in. beam expanding telescope to provide the necessary degree of collimation" (OCE 1980).

202. Precise distance measuring. "This type of measuring technique utilizes the measurement of the time that it takes a wave of light to travel from its source to a reflector and back to the source. Since the speed of the light wave can be accurately measured and corrected for various conditions of atmospheric density, the distance from source to target is a function of the time it requires the light beam to travel the course. The instrument used for this measurement is the Electronic Distance Measuring (EDM) instrument. This type of precision measurement is made to accurately determine the distance between a reference monument and an alignment marker. Used in its primary capacity, it will accurately measure distance; however, when it is used in triangulation and trilateration surveys, it will measure deflection of alignment markers" (OCE 1980).

203. Advantages include increased accuracy and decreased time consumption in measurement when compared to the old-fashioned method of taping distances. Resolutions up to 0.001 ft can be obtained with this method of measurement. Some instruments have a range of at least 2 miles. The units work off 12-v batteries which increase portability.

204. A summary tabulation of the instrumentation described in this report is presented in Figure 33 to assist in selecting instruments for evaluating questionable behavior in a concrete structure.

Environmental Influences

205. Environmental factors can be causative or contributory factors in the deterioration of concrete. Studying the environment in relation to concrete condition may afford a better understanding of how to repair the damaged concrete.

206. During construction, environmental effects should be studied in relation to the following:

- a. The concrete and concrete-making materials at the sources and at the construction site.
- b. Identifiable problems of handling, placing, and finishing concrete.
- c. Curing and early protection of concrete.

After construction, the environmental effects on the following should be contemplated:

- a. Differences in thermal exposure to solar heating. The north sides of structural elements generally have less deterioration because the lowest number of daily thermal cycles occur there.
- b. Differences in exposure to moisture, based on orientation of the construction to prevailing winds during times of rainfall or snowfall, and which will be affected by the diurnal thermal cycles.
- c. Differences in the mineral composition of the subgrade so that part of the construction is located on a foundation containing swelling clay or containing unstable sulfides or sulfates.
- d. Differences in the moisture content of the subgrade (Jansen 1980).

207. Environmental influences like freezing and thawing, chemical attack, abrasion, erosion, and cavitation have been covered under "Exterior Condition of Concrete" presented earlier in this report. Other detrimental elements like loading (impact, vibration, traffic) should also be studied if applicable. "The foundation and subgrade material and conditions should also be carefully examined if there is a possibility of their involvement in serviceability of the concrete." (Jansen 1980).

INSTRUMENT	DESCRIPTION	DATA PROVIDED	READOUT	RELIABILITY	INSTALLATION	MAINTENANCE	AVAILABILITY	REMARKS
VIBRATING WIRE STRAIN GAGE	Steel wire between end flanges enclosed in tube	Relative movement across joint or crack	One tech digital	Good long-term stability, high sensitivity	Specialist	Good, insensitive to moisture	Irad Gage, Inc.[1]	End flanges can be bolted or welded to structure, cast in place
MECHANICAL STRAIN GAGE	Mechanical gage with fixed reference point on structure	Relative movement across joint or crack	One tech gage with dial	Good with same operator	Technician	Fair, reference points require protection	Soiltest[1]	Simple system but accurate and repeatable
EXTENSOMETER	Anchors fixed in borehole with tensioned wire attached to sensing head	Relative displacement along borehole, across cracks	One tech digital	Good	Drill crew with specialist	Fair, sensing head needs cover or recessed	Terrametrics[1]	Good for monitoring wall stability, foundation/abutment
BALL-IN-BOX	Metal box with ball on rod assembly	Three-dimensional movement of joint or crack	One tech dial gage	Excellent	Survey crew	Fair, requires protective cover	Missouri River Division Lab	Shows good promise for relatively new gage
VIBRATING WIRE STRESS METER	Tensioned steel wire clamped inside cylinder anchored in borehole	Stress changes in medium	One tech digital	Fair	Drill crew with specialist	Fair, requires protective cover	Irad Gage, Inc.[1]	Maximum readings to 10,000 psi, sensitive to 2-psi stress change
WES HYDROSTATIC PRESSURE CELL	Strain gaged metal diaphragm in a cylinder	Hydrostatic pressure	One tech digital	Excellent	Drill crew with specialist	Good, go-no go system	Waterways Experiment Station	Long-term stability
OPEN TUBE PIEZOMETER	Vertical tube connected to porous tip	Hydrostatic pressure at porous tip	One tech with water-sensitive probe	Excellent	Drill crew	Good, open end of tube (riser) requires protection	Slope Indicator[1]	Recommend porous tip piezometer over other types
VEE-NOTCH WEIR	Vee-notched plate and depth gage	Flow rates	One tech	Excellent	Technician	Good, indoors; fair, outdoors	Local	Sometimes requires collector system
ELECTROLEVEL	Spirit level bubble changes resistance of electrodes monitored by meter	Tilting and deflection	Specialist bridge circuit and amplifier	Good	Survey crew	Fair	Plessey, Inc., Hauppauge, N. Y.	Indications of tilt of less than 1 second of arc
INCLINOMETER	Embedded tube electronic probe and readout box	Depth, magnitude, and rate of lateral movement or tilt	One tech magnetic tape	Good	Drill crew with specialist	Good, should have protective box	Slope Indicator Company[1]	Good instrument with many applications
LASER ALIGNMENT	Laser transmitter and fixed target, receiver	Horizontal and vertical deviations from a base line	Specialist instrument monitor	Good	Survey crew	Good, shield laser from wind gusts	Engineer Topographic Lab. Similar equipment local	Good idea to use two receivers, stability of laser beam affected by ambient atmosphere
PRECISE DISTANCE MEASURING	Source instrument, a reflector prism assembly and reference monuments	Distance and deflection	Specialist source instrument contains readout	Good	Survey crew	Good, atmospheric conditions along length to be measured to be equal	Local	Increase accuracy and decrease time over taping method

1 Not the only source.

Figure 33. Instrumentation summary (from Hoot 1981)

PART IV: LABORATORY INVESTIGATION

General

208. The laboratory investigation is conducted for two reasons: (a) to determine the quality of the concrete by conducting physical, mechanical, and chemical tests, and (b) to determine the cause of the concrete damage/deterioration. Besides using the physical and mechanical test results for quality determinations, the results are available for use in structural stress and stability analysis.

209. After the core is received in the laboratory, the first task is to lay the boxes out and begin a detailed log of the core. This is essential so that no weakness within the core is overlooked. The petrographic analysis of the concrete should be performed by a person qualified by education and experience so that proper interpretation of test results is made. The petrographer should be consulted before samples are taken in the field (ACI Committee 207 1979).

210. Procedural steps in the petrographic examination and features looked for include the following (ACI Committee 207 1979).

> Visual inspection with the unaided eye, a hand lens and a stereoscopic microscope can provide valuable information when applied to original exterior surfaces, surfaces of fractures and voids, surfaces of fresh fractures, and through the cement paste and aggregate. From this examination, the following features can be studied and described:
>
> - Condition of the aggregrate
> - Pronounced cement-aggregate reactions
> - Deterioration of aggregate particles in place
> - Denseness of cement paste
> - Homogeneity of the concrete
> - Occurrence of settlement and bleeding of fresh concrete
> - Depth and extent of carbonation
> - Occurrence and distribution of fractures
> - Characteristics and distribution of voids
> - Presence of contaminating substances
>
> As part of the visual examination, noteworthy portions of the concrete, secondary deposits, or particles of aggregate are separated for more detailed microscopical study or for chemical, x-ray diffraction, or other types of analyses.

211. Petrographic thin sections of the concrete are made and examined for the following features (ACI Committee 207 1979). Detailed instructions on conducting a petrographic examination of hardened concrete can be found in ASTM C858-77/CRD-C 57-78.

- Composition of fine and coarse aggregates
- Evidence of cement-aggregate reaction
- Proportion of unhydrated granules of cement
- Presence of mineral admixtures

The thin sections permit a thorough examination of the concrete texture and structure. Where microscopy techniques do not give enough detailed information, X-ray diffraction, differential thermal analysis, and X-ray emission techniques can be used.

212. The volumetric proportions of aggregate, cement paste, and air voids can be obtained using ASTM C-457. This information can be used to further assess the quality of the concrete and to check specified quantities of materials required to be in the concrete.

213. A chemical analysis of hardened concrete may be conducted for a number of reasons. The most common is for ascertaining the proportions of cement used in the mixture. Improper amounts of cement can easily lead to inferior concrete strength; hence, the concrete may be highly susceptible to deterioration.

214. The following physical and mechanical tests are generally performed on concrete core:

a. Density.

b. Compressive strength.

c. Static modulus of elasticity.

d. Poisson's ratio.

e. Bulk modulus.

f. Pulse velocity.

g. Direct shear strength of concrete bonded to foundation rock.

h. Friction sliding of concrete on foundation rock.

i. Volume change potential by freezing and thawing.*

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* These tests may be useful in predicting the relative rate at which deterioration of concrete in the structure may occur and service life of the structure.

PART V: REPORT

General

215. A formal report shall be submitted to the agency or organization requesting the condition survey. The report should clearly state the condition of the concrete in the structure and appurtenant structure. Any dangerous conditions existing in the concrete structure and evidence of existing or potential problems in the site environs, embankments, foundation, or in electrical, mechanical, or hydraulic features should be reported to appropriate officials of the project immediately. The final report will also contain such information.

Contents of Report

216. A description of the project should include vicinity, locality, and plan view maps, elevations, sections of the structures, and foundation sections, and geologic maps when applicable. "General purpose and operating requirements of the project and safety hazards and economic impacts involved in case of structural failure should be described. Significant structural design criteria upon which evaluation of the concrete was made and analyses, test methods, data and investigations pertinent to the evaluation should be described." (ACI Committee 207 1979). All engineering data reviewed concerning the foundation design, construction, operations, and maintenance should be referenced.

217. The report should also contain a summary of data collected; i.e., existing records and documents; visual inspection of concrete including photographs and sketches which indicate location, extent, and depth of damaged and eroded concrete; analysis of existing instrumentation, inspections, and test records; and results and analysis of new investigations and test data.

218. The report should specify the current adequacy of concrete based on recent design criteria and service conditions. Projections of continued serviceability should also be made. When appropriate, recommendations for conventional or state-of-the-art repair should be given to assure serviceability of the structure. WES has an ongoing research program entitled "Maintenance and Preservation of Concrete Structures" which includes an objective

to develop and evaluate materials and techniques for repair and rehabilitation of Civil Works structures. The reader is encouraged to contact WES for information, bulletins, and reports of work accomplished and planned on the repair of concrete structures.

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APPENDIX A:

GUIDE FOR MAKING A CONDITION SURVEY OF CONCRETE IN SERVICE

(ACI 201.1R-68)

ACI Committee Reports, Guides, Standard Practices, and Commentaries are intended for guidance in designing, planning, executing, or inspecting construction, and in preparing specifications. Reference to these documents shall not be made in the Project Documents. If items found in these documents are desired to be part of the Project Documents, they should be incorporated directly into the Project Documents.

ACI 201.1R-68

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Guide for Making a Condition Survey of Concrete in Service

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This guide provides a system for reporting on the condition of concrete in service. It includes a check list of the many details to be considered in making a report, and provides standard definitions of 40 terms associated with the durability of concrete. Its purpose is to establish a uniform system for evaluating the condition of concrete.

Keywords: buildings; concrete construction; concrete durability; concrete pavements; concretes; corrosion; cracking (fracturing); deterioration; environment; freeze-thaw durability; inspection; joints; popouts; quality control; scaling; serviceability; spalling; strength; surveys (data collection).

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■ A CHECK LIST IS provided for making a survey of the condition of concrete. The list is designed to be used in recording the history of a concrete project from inception through completion and subsequent life of the structure or pavement.

While it probably will be used most often in connection with the survey of concrete that is showing some degree of distress, its application is recommended for all important concrete structures. In any case, records of the materials and construction practices used should be maintained because they are difficult to obtain at a later date.

The committee has attempted to include all pertinent items that might have a bearing on the performance of the concrete. However, those making the survey should not limit their investigation to the items listed, thereby overlooking or ignoring other possible contributing factors. Simply following the guide will not eliminate the need for intelligent observation and the use of sound judgment.

Those performing the survey should be experienced and competent in this field. In addition to verbal descriptions, numerical data obtained by laboratory tests and field measurements should be provided wherever possible. Photographs, including a scale to indicate linear dimensions, are of great value in showing condition of structure.

One of the objects of a condition survey is to provide information that will be of value in the construction of more economical, serviceable structures. The survey may show causes of deterioration or lack of need of expensive materials or construction methods. The check list should be used in conjunction with the following:

1. ACI Committee 311, "Recommended Practice for Concrete Inspection (ACI 311-75)," American Concrete Institute, 1975, 6 pp. Also *ACI Manual of Concrete Practice,* Part 2.

2. ACI Committee 201, "Guide to Durable Concrete"—ACI 201.2R-77, ACI JOURNAL, *Proceedings* V. 74, No. 12, Dec. 1977, pp. 573-609. Also *ACI Manual of Concrete Practice,* Part 1.

CHECK LIST

1. Description of structure or pavement

1.1. Name, location, type, and size

1.2. Owner, project engineer, contractor, when built

1.3. Design

- 1.3.1. Architect and/or engineer
- 1.3.2. Intended use and history of use
- 1.3.3. Special features

1.4. Photographs

- 1.4.1. General view
- 1.4.2. Detailed close-ups of condition of area

1.5. Sketch map — orientation showing sunny and shady walls and well and poorly drained regions

2. Present condition of structure

2.1. Over-all alignment of structure

- 2.1.1. Settlement
- 2.1.2. Deflection
- 2.1.3. Expansion
- 2.1.4. Contraction

2.2. Portions showing distress (beams, columns, pavement, walls, etc. Subjected to strains and pressures)

2.3. Surface condition of concrete

- 2.3.1. General (good, satisfactory, poor, etc.)
- 2.3.2. Cracks
  - 2.3.2.1. Location and frequency
  - 2.3.2.2. Type and size
  - 2.3.2.3. Leaching, stalactites
- 2.3.3. Scaling
  - 2.3.3.1. Area, depth
  - 2.3.3.2. Type (see definition)
- 2.3.4. Spalls and popouts
  - 2.3.4.1. Number, size and depth
  - 2.3.4.2. Type (see definitions)
- 2.3.5. Extent of corrosion or chemical attack
- 2.3.6. Stains
- 2.3.7. Exposed steel
- 2.3.8. Previous patching or other repair

2.4. Interior condition of concrete

- 2.4.1. Strength of cores
- 2.4.2. Density of cores
- 2.4.3. Moisture content (degree of saturation)
- 2.4.4. Evidence of alkali-aggregate or other reaction
- 2.4.5. Bond to aggregate, reinforcing steel, joints
- 2.4.6. Pulse velocity
- 2.4.7. Volume change
- 2.4.8. Air content and distribution

3. Nature of loading and detrimental elements

3.1. Exposure

- 3.1.1. Environment — arid, subtropical, marine, freshwater, industrial, etc.
- 3.1.2. Weather — (July and January mean temperatures, mean annual rainfall and months in which 60 percent of it occurs)
- 3.1.3. Freezing and thawing
- 3.1.4. Wetting and drying
- 3.1.5. Drying under dry atmosphere
- 3.1.6. Chemical attack — sulfates, acids
- 3.1.7. Abrasion, erosion, cavitation
- 3.1.8. Electric currents

3.2. Drainage

- 3.2.1. Flashing
- 3.2.2. Weepholes
- 3.2.3. Contour

3.3. Loading

- 3.3.1. Dead
- 3.3.2. Live
- 3.3.3. Impact

3.3.4. Vibration
3.3.5. Traffic index
3.3.6. Other
3.4. Soils (foundation conditions)
3.4.1. Stability
3.4.2. Expansive soil
3.4.3. Settlement
3.4.4. Restraint

4. Original condition of structure
4.1. Condition of formed and finished surfaces
4.1.1. Smoothness
4.1.2. Air pockets
4.1.3. Sand streaks
4.1.4. Honeycomb
4.1.5. Soft areas
4.2. Early structural defects
4.2.1. Cracking
4.2.1.1. Plastic shrinkage
4.2.1.2. Settlement
4.2.1.3. Cooling
4.2.2. Curling
4.2.3. Structural settlement

5. Materials of construction
5.1. Hydraulic cement
5.1.1. Type and source
5.1.2. Chemical analysis (obtain certified test data if available)
5.1.3. Physical properties
5.2. Aggregates
5.2.1. Coarse
5.2.1.1. Type, source and mineral composition (representative sample available)
5.2.1.2. Quality characteristics
5.2.1.2.1. Percentage of deleterious material
5.2.1.2.2. Percentage of potentially reactive materials
5.2.1.2.3. Coatings, texture, and particle shape
5.2.1.2.4. Gradation, soundness, hardness
5.2.1.2.5. Other properties as specified in ASTM Designation C 33 (C 330 — for lightweight aggregate)
5.2.2. Fine aggregate
5.2.2.1. Type, source, and mineral composition (representative sample available)
5.2.2.2. Quality characteristics
5.2.2.2.1. Percentage of deleterious material
5.2.2.2.2. Percentage of potentially reactive materials
5.2.2.2.3. Coatings, texture and particle shape
5.2.2.2.4. Gradation, soundness and hardness
5.2.2.2.5. Other properties as specified in ASTM Designation C33 (C330 for lightweight aggregate)
5.3. Mixing water
5.3.1. Source and quality
5.4. Air-entraining agents
5.4.1. Type and source
5.4.2. Composition
5.4.3. Amount
5.4.4. Manner of introduction
5.5. Admixtures
5.5.1. Mineral admixture
5.5.1.1. Type and source
5.5.1.2. Physical properties
5.5.1.3. Chemical properties
5.5.2. Chemical admixture
5.5.2.1. Type and source
5.5.2.2. Composition
5.5.2.3. Amount
5.6 Concrete
5.6.1. Mixture proportions
5.6.1.1. Cement content
5.6.1.2. Proportions of each size aggregate
5.6.1.3. Water-cement ratio
5.6.1.4. Water content
5.6.1.5. Chemical admixture
5.6.1.6. Mineral admixture
5.6.1.7. Air-entraining agent
5.6.2. Properties of fresh concrete
5.6.2.1. Slump
5.6.2.2. Percent air
5.6.2.3. Workability
5.6.2.4. Unit weights
5.6.2.5. Temperature
5.6.3. Type
5.6.3.1. Cast-in-place
5.6.3.2. Precast
5.6.3.3. Prestressed
5.6.4. Reinforcement
5.6.4.1. Yield strength
5.6.4.2. Thickness of cover
5.6.4.3. Presence of stirrups
5.6.4.4. Use of welding

6. Construction practices
6.1. Storage and processing of materials
6.1.1. Aggregates
6.1.1.1. Grading
6.1.1.2. Washing
6.1.1.3. Storage
6.1.1.3.1. Stockpiling
6.1.1.3.2. Bins
6.1.2. Cement and admixtures
6.1.2.1. Storage
6.1.2.2. Handling
6.1.3. Reinforcing steel and inserts
6.1.3.1. Storage
6.1.3.2. Placement
6.2. Forming
6.2.1. Type
6.2.2. Bracing
6.2.3. Coating
6.2.4. Insulation

6.3. Concreting operation

6.3.1. Batching plant

6.3.1.1. Type—automatic, manual, etc.

6.3.1.2. Condition of equipment

6.3.1.3. Batching sequence

6.3.2. Mixing

6.3.2.1. Type—central mix, truck mix, job mix, shrink mix, etc.

6.3.2.2. Condition of equipment

6.3.2.3. Mixing time

6.3.3. Method of transporting—trucks, buckets, chutes, pumps, etc.

6.3.4. Placing

6.3.4.1. Methods—conventional, underwater slipform, etc.

6.3.4.2. Equipment—buckets, elephant trunks, vibrators, etc.

6.3.4.3. Weather conditions—time of year, rain, snow, dry wind, temperature, humidity, etc.

6.3.4.4. Site conditions—cut, fill, presence of water, etc.

6.3.4.5. Construction joints

6.3.5. Finishing

6.3.5.1. Type—slabs, floors, pavements, appurtenances

6.3.5.2. Method—hand or machine

6.3.5.3. Equipment—screeds, floats, trowels, straight-edge, belt, etc.

6.3.5.4. Additives, hardeners, water, dust coat, coloring, etc.

6.3.6. Curing Procedures

6.3.6.1. Method—water, covering, curing compounds

6.3.6.2. Duration

6.3.6.3. Efficiency

6.3.7. Form removal (time of removal)

7. Initial physical properties of hardened concrete

7.1. Strength—compressive, flexural, elastic modulus

7.2. Density

7.3. Percentage and distribution of air

7.4. Volume change potential

7.4.1. Shrinkage or contraction

7.4.2. Expansion or swelling

7.4.3. Creep

7.5. Thermal properties

8. Additional items pertaining to pavements

8.1. Structural section (sketch and thickness of pavement layers—base, subbase, etc.)

8.2. Joints

8.2.1. Type, spacing, design

8.2.2. Condition

8.2.3. Filling material

8.2.4. Faulting—(measured in mm)

8.3. Cracks

8.3.1 Type (longitudinal, transverse, corner), size (measured in mm), frequency

8.4. Patching

8.5. Riding quality (as measured by instruments such as the BPR roughometer, the CHLOE profilometer, or profilograph present serviceability index, (PSI), etc.)

8.6. Condition of shoulders and ditches

APPENDIX

DEFINITION OF TERMS ASSOCIATED WITH THE DURABILITY OF CONCRETE

A.1 Cracks: An incomplete separation into one or more parts with or without space between.

A.1.1. Cracks will be classified by direction, width and depth. The following adjectives can be used: longitudinal, transverse, vertical, diagonal, and random. Three width ranges are suggested as follows: fine—generally less than 1 mm; medium—between 1 and 2 mm; wide—over 2 mm (see Fig. A.1.1.a through A.1.1.h).

A.1.2. *Pattern cracking*: Fine openings on concrete surfaces in the form of a pattern; resulting from a decrease in volume of the material near the surface, or increase in volume of the material below the surface, or both (see Fig. A.1.2.a through A.1.2.c).

A.1.3. *Checking*: Development of shallow cracks at closely spaced but irregular intervals on the surface of mortar or concrete (see Fig. A.1.3).

A.1.4. *Hairline cracking*: Small cracks of random pattern in an exposed concrete surface.

A.1.5. *D-cracking*: The progressive formation on a concrete surface of a series of fine cracks at rather close intervals, often of random patterns, but in highway slabs paralleling edges, joints, and cracks and usually curving across slab corners (see Fig. A.I.5.a and A.1.5.b).

A.2. Deterioration: Deterioration is any adverse change of normal mechanical, physical and chemical properties either on the surface or in the whole body of concrete generally through separation of its components.

A.2.1. *Disintegration*: Deterioration into small fragments or particles due to any cause (see Fig. A.2.1).

A.2.2. *Distortion*: Any abnormal deformation of concrete from its original shape (see Fig. A.2.2).

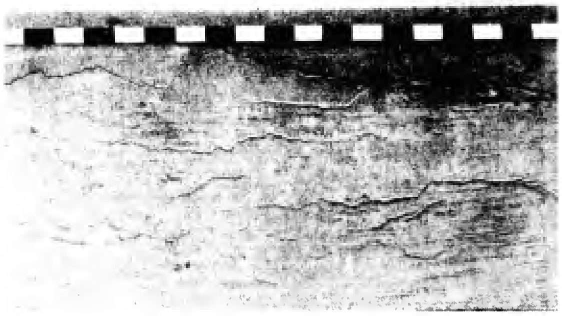

Fig. A.1.1.a—Longitudinal cracks (medium)

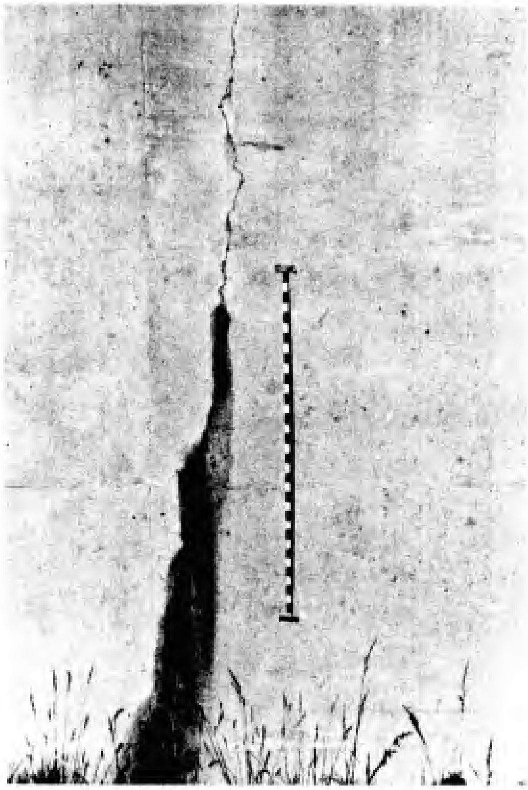

Fig. A.1.1.d—Vertical crack (medium)

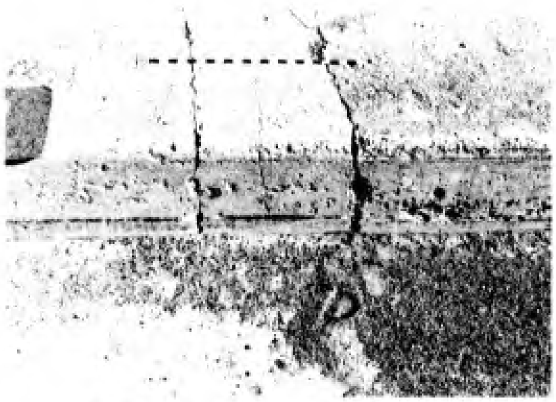

Fig. A.1.1.b—Transverse cracks (wide)

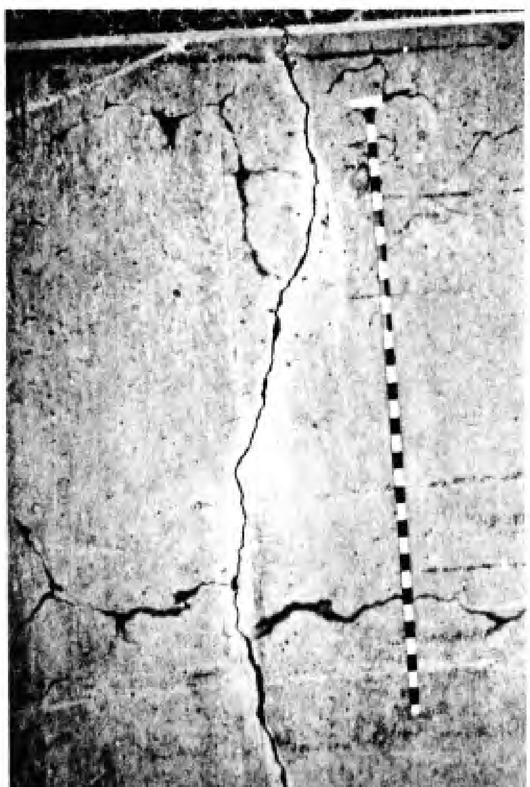

Fig. A.1.1.e—Vertical crack (wide)

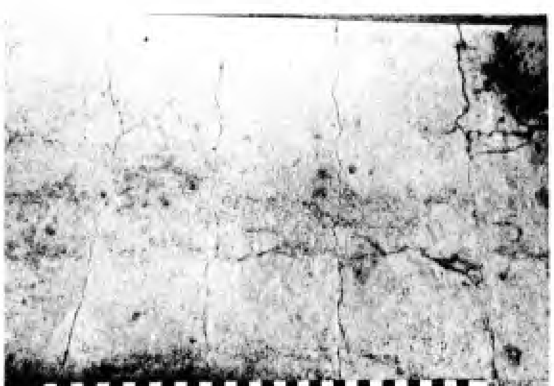

Fig. A.1.1.c—Transverse cracks (fine)

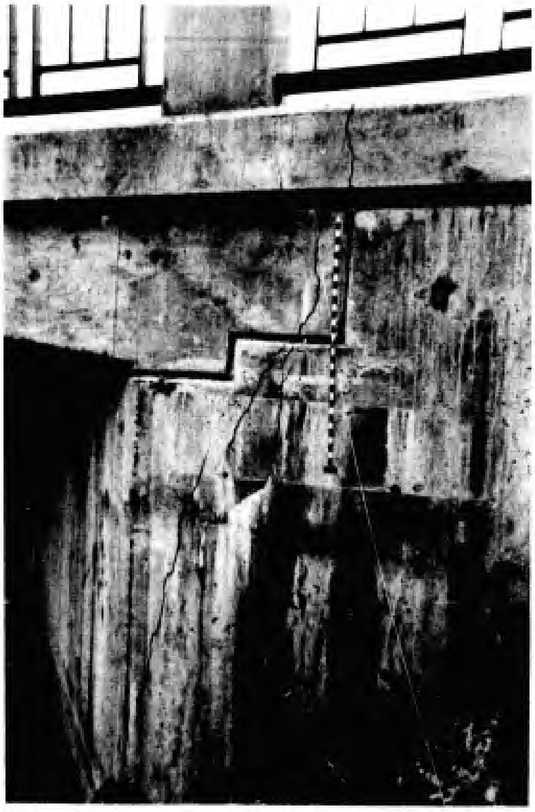

Fig. A.1.1.f—Diagonal cracks (wide)

Fig. A.1.1.g—Random cracks (wide)

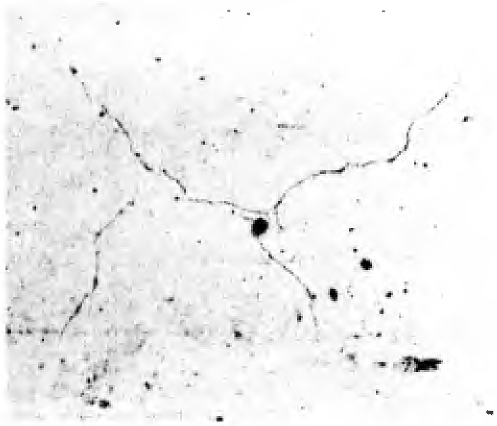

Fig. A.1.1.h—Random cracks (medium)

Fig. A.1.2.a—Pattern cracking (fine)

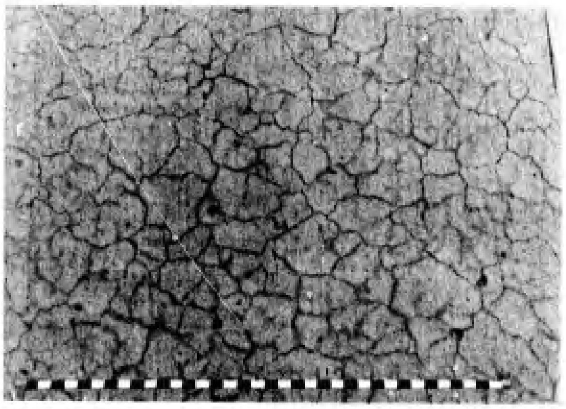

Fig. A.1.2.b—Pattern cracking (medium)

Fig.A.1.2.c—Pattern cracking (wide)

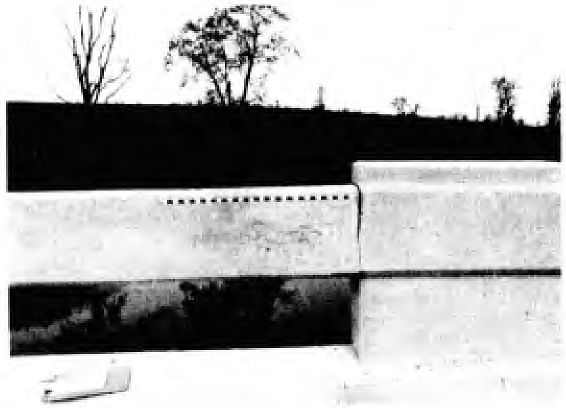

Fig. A.1.5.b—D-cracking (fine)

Fig. A.1.3—Checking (medium)

Fig. A.2.1—Disintegration

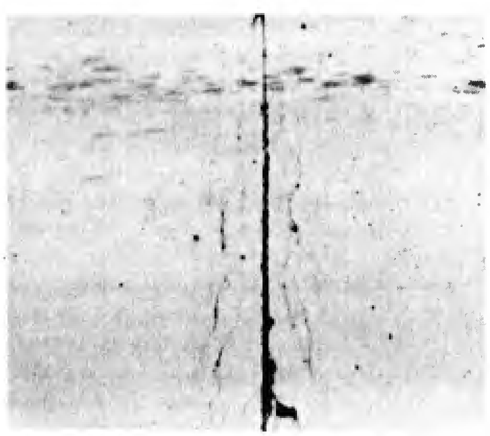

Fig. A.1.5.a—D-cracking (fine)

A.2.3. *Efflorescence*: A deposit of salts, usually white, formed on a surface, the substance having emerged from below the surface.

A.2.4. *Exudation*: A liquid or viscous gel-like material discharged through a pore, crack or opening in the surface (see Fig. A.2.4.a, A.2.4.b, and A.2.5).

A.2.5. *Incrustation*: A crust or coating generally hard formed on the surface of concrete or masonry construction (see Fig. A.2.5)

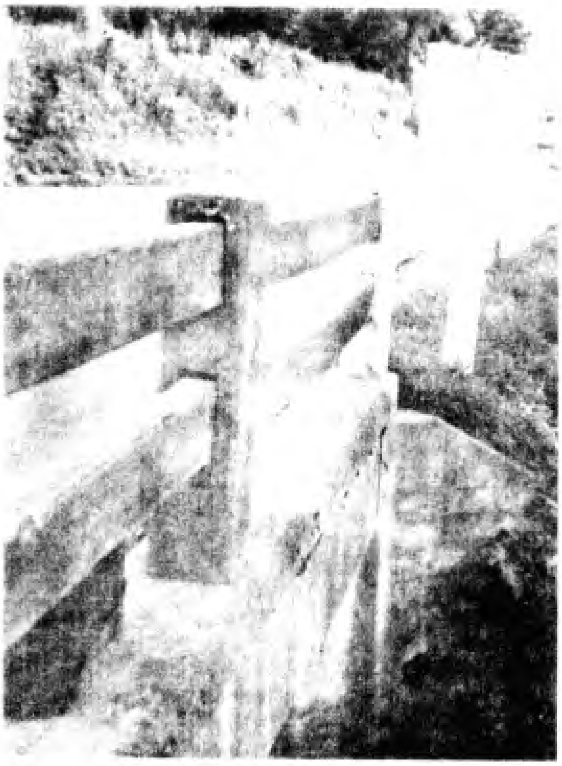

Fig. A.2.2—Distortion

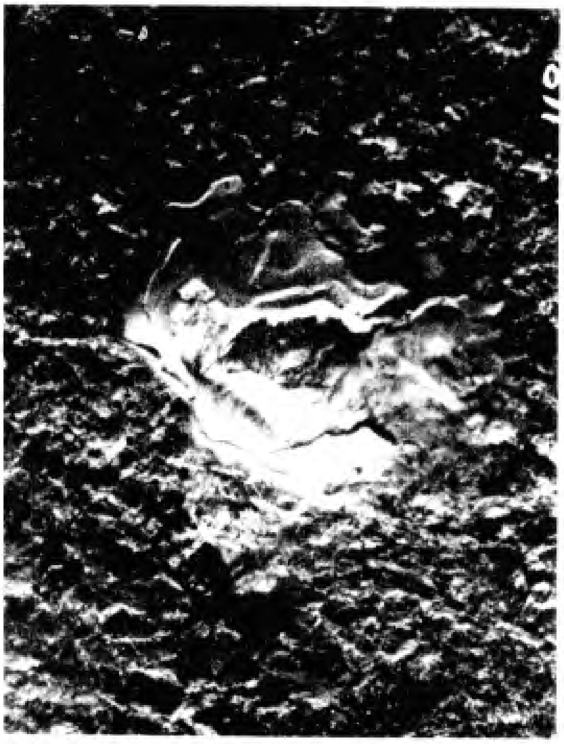

Fig. A.2.4.b—Exudation

Fig. A.2.4.a—Exudation

Fig. A.2.5—Exudation and incrustation

A.2.6. *Pitting*: Development of relatively small cavities in a surface, due to phenomena such as corrosion or cavitation, or, in concrete, localized disintegration.

A.2.7. *Popout*: The breaking away of small portions of a concrete surface due to internal pressure which leaves a shallow, typical conical, depression (see Fig. A.2.7).

A.2.7.1. *Popouts, small:* Popouts leaving holes up to 10 mm in diameter, or the equivalent (see Fig. A.2.7.1).

A.2.7.2. *Popouts, medium:* Popouts leaving holes between 10 and 50 mm in diameter, or equivalent (see Fig. A.2.7.2).

A.2.7.3. *Popouts, large:* Popouts leaving holes greater than 50 mm in diameter, or the equivalent (see Fig. A.2.7.3).

A.2.8. *Erosion*: Deterioration brought about by the abrasive action of fluids or solids in motion (see Fig. A.2.8).

A.2.9. *Scaling*: Local flaking or peeling away of the near surface portion of concrete or mortar.

A.2.9.1. *Peeling*: A process in which thin flakes of mortar are broken away from a concrete surface; such as by deterioration or by adherence of surface mortar to forms as forms are removed (see Fig. A.2.9.1.a and A.2.9.1.b).

Fig. A.2.7—Popout

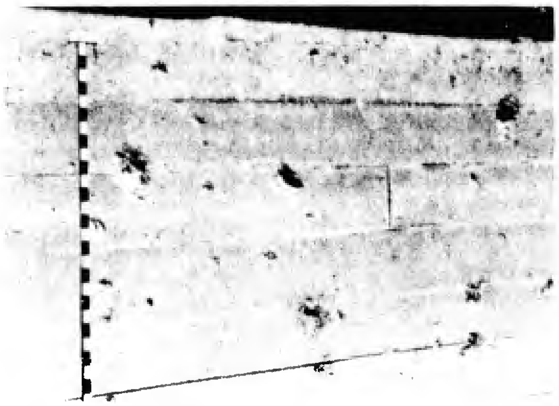

Fig. A.2.7.2—Popouts (medium)

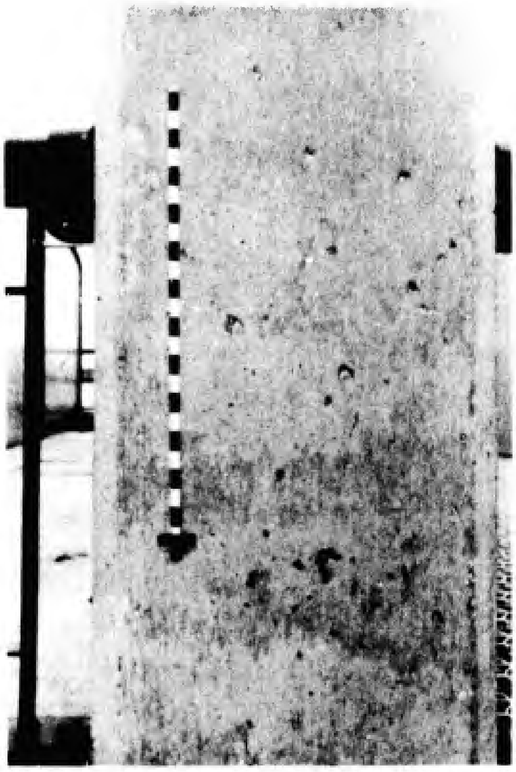

Fig. A.2.7.1—Popouts (small)

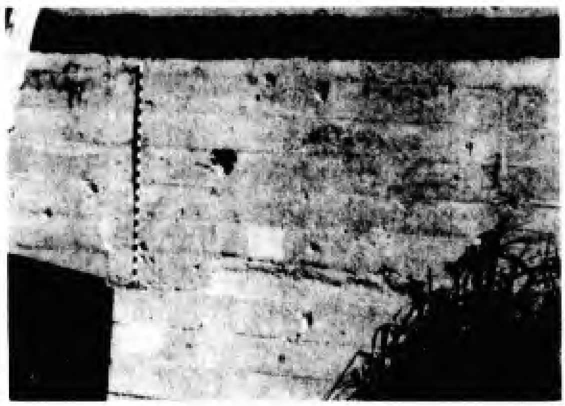

Fig. A.2.7.3—Popouts (large)

A.2.9.2. *Scaling, light*: Loss of surface mortar without exposure of coarse aggregate (see Fig. A.2.9.2.a and A.2.9.2.b).

A.2.9.3. *Scaling, medium:* Loss of surface mortar up to 5 to 10 mm in depth and exposure of coarse aggregate (see Fig. A.2.9.3.a and A.2.9.3.b).

A.2.9.4. *Scaling, severe:* Loss of surface mortar 5 to 10 mm in depth with some loss of mortar surrounding aggregate particles 10 to 20 mm in

Fig. A.2.8—Erosion

depth, so that aggregate is clearly exposed and stands out from the concrete (see Fig. A.2.9.4.a and A.2.9.4.b).

A.2.9.5. *Scaling, very severe:* Loss of coarse aggregate particles as well as surface mortar and mortar surrounding aggregate, generally greater than 20 mm in depth (see Fig. A.2.9.5.a and A.2.9.5.b).

A.2.10. *Spall:* A fragment, usually in the shape of a flake, detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the large mass.

A.2.10.1. *Small spall:* A roughly circular or oval depression generally not greater than 20 mm in depth nor greater than about 150 mm in any dimension, caused by the separation of a portion of the surface concrete (see Fig. A.2.10.1).

A.2.10.2. *Large spall:* May be roughly circular or oval depression, or in some cases an elongated depression over a reinforcing bar, generally 20 mm or more in depth and 150 mm or greater in any dimension, caused by a separation of the surface concrete (see Fig. A.2.10.2).

A.2.11. *Joint spall:* Elongated cavity along a joint (see Fig. A.2.11.a and A.2.11.b).

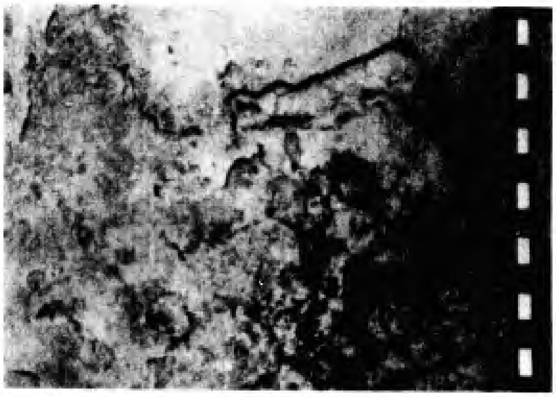

Fig. A.2.9.1.a—Close-up of peeling

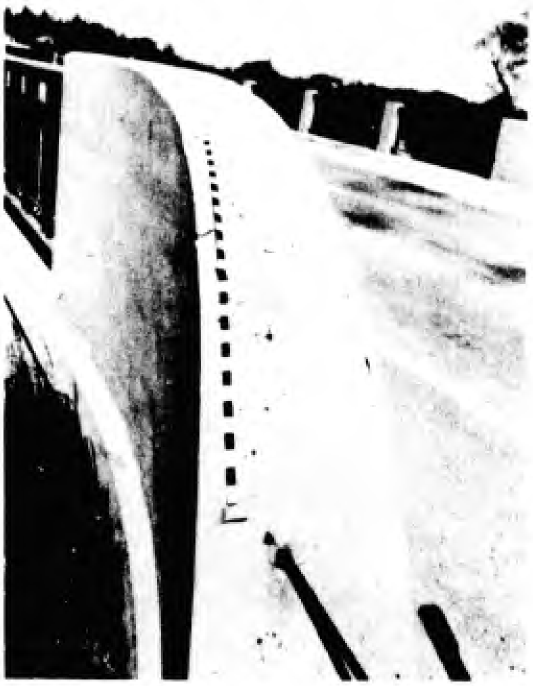

Fig. A.2.9.1.b—Peeling on bridge abutment

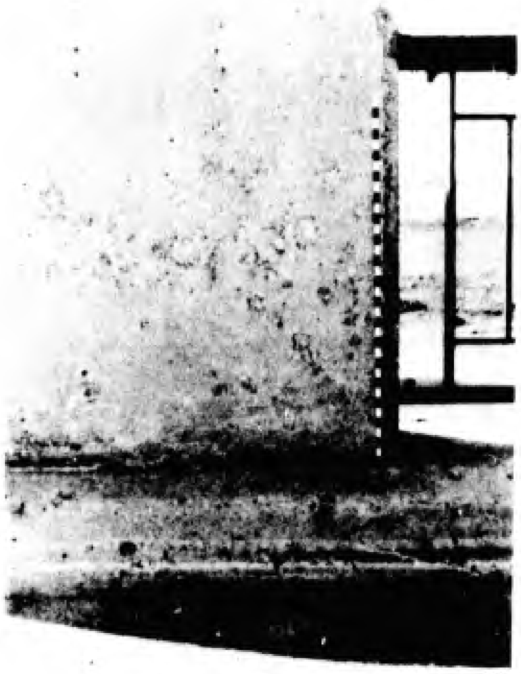

Fig. A.2.9.2.a—Scaling (light)

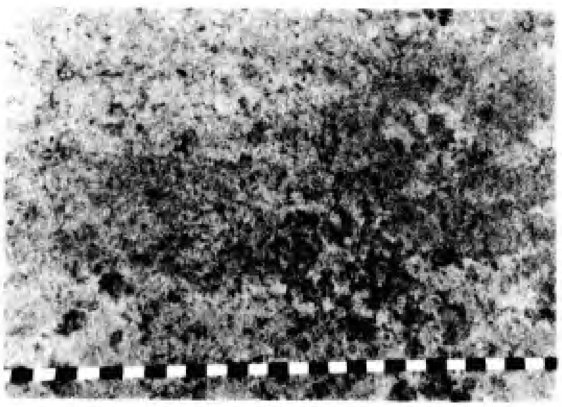

Fig. A.2.9.2.b—Close-up of scaling (light)

A.2.12. *Drummy area:* Area of concrete surface which gives off a hollow sound when struck.

A.2.13. *Stalactite*: A downward pointing formation, hanging from the surface of concrete, shaped like an icicle.

A.2.14. *Stalagmite*: As stalactite, but upward formation.

A.2.15. *Dusting*: The development of a powdered material at the surface of hardened concrete (see Fig. A.2.15).

Fig. A.2.9.3.a—Scaling (medium)

Fig. A.2.9.4.b—Scaling severe

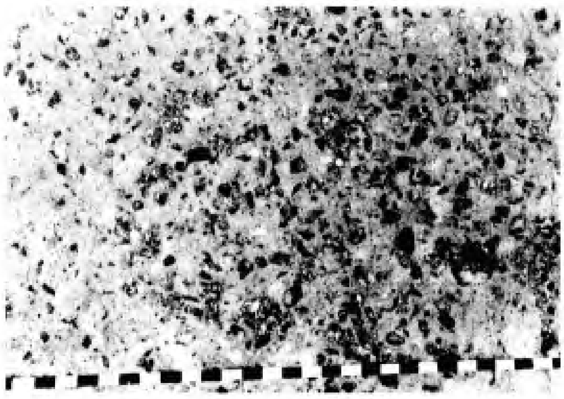

Fig. A.2.9.3.b—Close-up of scaling (medium)

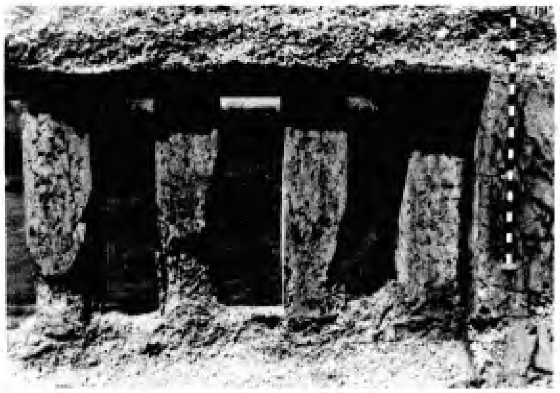

Fig. A.2.9.5.a—Scaling (very severe)

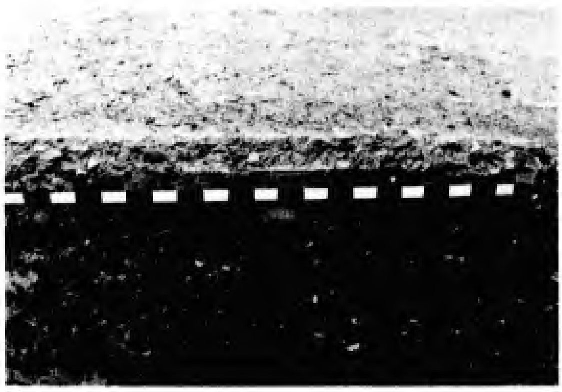

Fig. A.2.9.4.a—Close-up of scaling (severe)

Fig. A.2.9.5.b—Close-up of scaling (very severe)

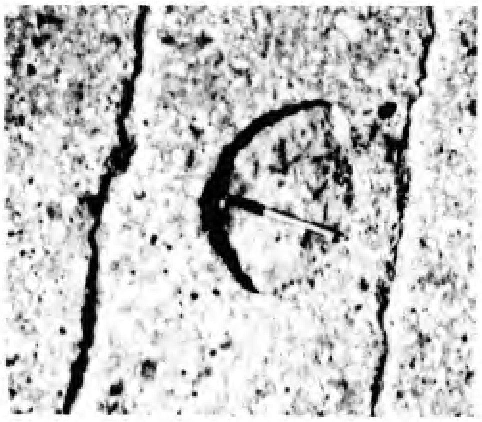

Fig. A.2.10.1—Small spall

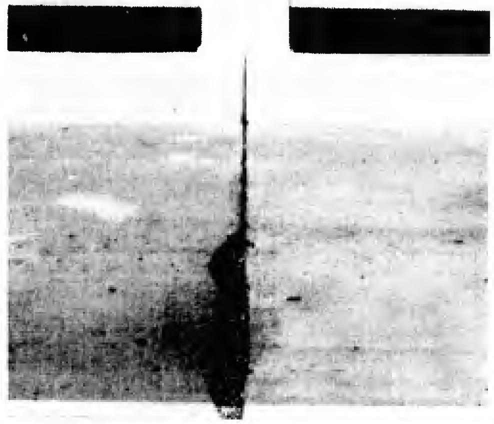

Fig. A.2.11.a—Joint spall

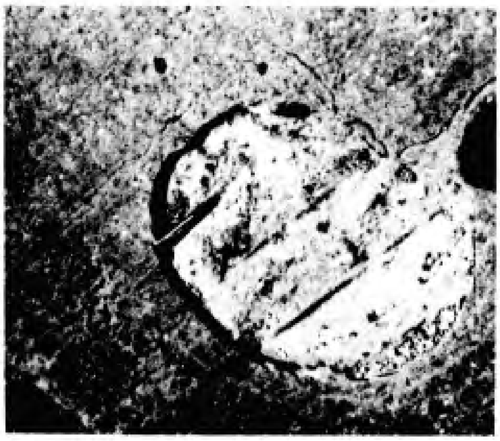

Fig. A.2.10.2—Large spall

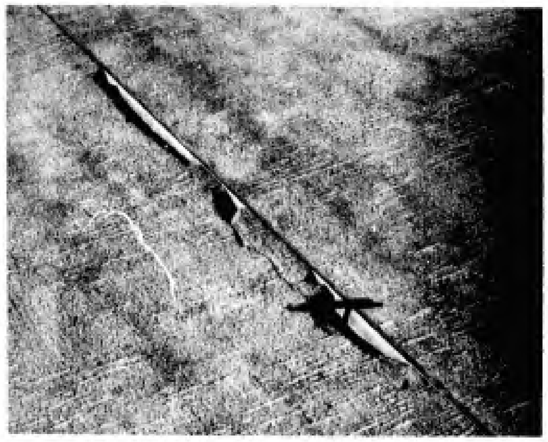

Fig. A.2.11.b—Joint spall

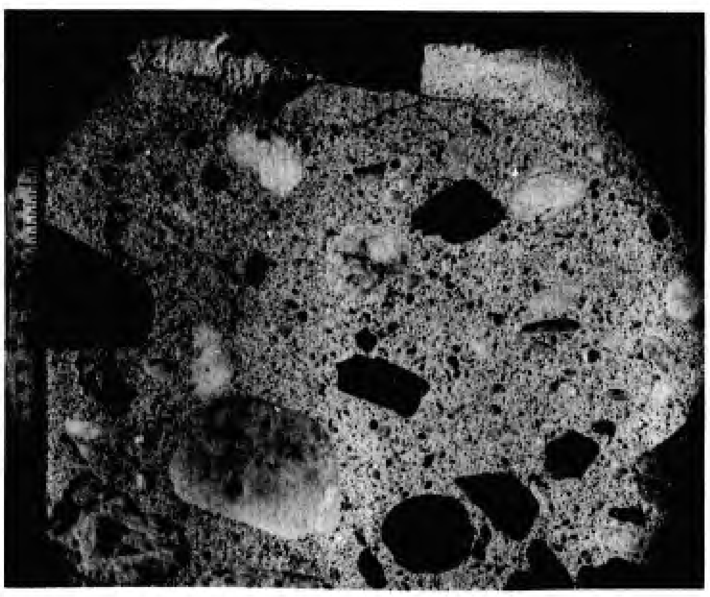

Fig. A.2.15—Dusting; surface at top of ruler is a floor surface of concrete placed *very* wet and which also carbonated; segregation is also evident

A.2.16. *Corrosion*: Disintegration or deterioration of concrete or reinforcement by electrolysis or by chemical attack (see Fig. A.2.16).

A.3. Textural defects:

A.3.1. *Bleeding channels*: Essentially vertical localized open channels caused by heavy bleeding (see Fig. A.3.1).

A.3.2. *Sand Streak*: Streak in surface of formed concrete caused by bleeding (see Fig. A.3.2).

A.3.3. *Water pocket*: Voids along the underside of aggregate particles or reinforcing steel which formed during the bleeding period. Initially filled with bleeding water.

A.3.4. *Stratification*: The separation of overwet or overvibrated concrete into horizontal layers with increasingly lighter material toward the top; water, laitance, mortar, and coarse aggregate will tend to occupy successively lower positions in that order; a layered structure in concrete resulting from placing of successive batches that differ in appearance (see Fig. A.3.4.).

A.3.5. *Honeycomb*: Voids left in concrete due to failure of the mortar to effectively fill the spaces among coarse aggregate particles (see Fig. A.3.5.a and A.3.5.b).

Fig. A.2.16—Corrosion

Fig. A.3.1—Bleeding channels and water pockets of concrete in a caisson; note laitance below particles of coarse aggregate

Fig. A.3.2—Sand streaking on a vertical formed surface

Fig. A.3.4—Stratification

Fig. A.3.5.a—Honeycomb

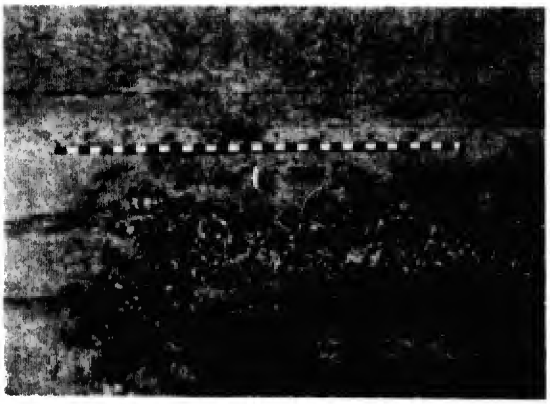

Fig. A.3.5.b—Honeycomb

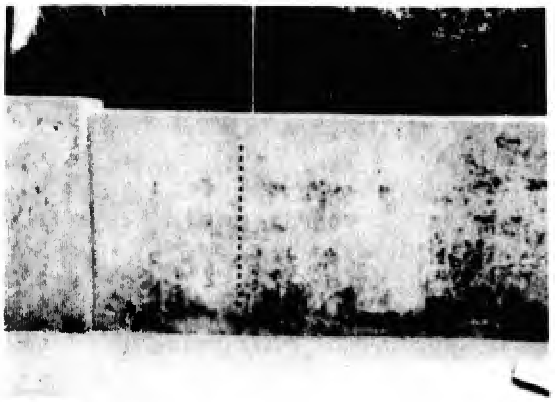

Fig. A.3.8—Discoloration

A.3.6. *Sand Pocket*: Part of concrete containing sand without cement.

A.3.7. *Segregation*: The differential concentration of the components of mixed concrete, resulting in non uniform proportions in the mass.

A.3.8. *Discoloration*: Departure of color from that which is normal or desired (see Fig. A.3.8).

REFERENCES

1. ACI Committee 116, "Cement and Concrete Terminology"—ACI 116R-78, American Concrete Institute, Detroit, 1978, 50 pp., Also, *ACI Manual of Concrete Practice,* Part 1.

2. Committee DB-5, "Standard Nomenclature and Definitions for Use in Pavement Inspection and Maintenance," Highway Research Board, Washington, D.C.

3. *Trilingual Dictionary of Engineering Materials Testing*, RILEM Bulletins 20-25, Paris, 1955.

This report was approved by letter ballot of the committee and reported to ACI headquarters Jan. 5, 1967. At the time of balloting (late 1966), the committee consisted of 22 members, of whom 19 voted affirmatively, 1 negatively, one "conditionally" affirmative, and one not returning his ballot.